bulletin 55

draft model code



Model Code 2010

First complete draft

Volume 1

Model Code 2010

First complete draft

Volume 1

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State-of-Art Report	approved by a Commission
Manual, Guide (to good practice) or Recommendation	approved by the Technical Council of ***fib***
Model Code	approved by the General Assembly of ***fib***

Any publication not having met the above requirements will be clearly identified as a preliminary draft.
This Bulletin 55 is a draft Model Code; it has not yet been approved by the General Assembly of ***fib***.

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Cover image: Svratka River pedestrian bridge, Brno, Czech Republic; one of the winners in the 2010 *fib* Awards for Outstanding Concrete Structures, Civil Engineering Structures Category. Structural concept by Jiri Strasky; architectural concept by Vaclav Hlavacek.

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Preface

In 1978 the first Model Code for Concrete Structures was published. It was a product of cooperation between CEB and FIP, two large international bodies aiming at synthesizing research findings, defining new research directions and producing design recommendations. The first CEB/FIP Model Code had a considerable impact on the national codes in many countries.

The CEB/FIP Model Code 1990, which was the next in the series, was a major step forward to international harmonization of codes. It served as a basis for the Eurocode for Concrete Structures, which is now introduced in most European countries.

Twelve years after the appearance of MC90 it was decided to start again the preparation of a new Model Code. The first meeting, of the MC Preparation Group was held in May 2002 in Delft, The Netherlands. The last, and 15th, meeting was in held in November 2009 in Oslo.

fib Special Activity Group 5 has tried to write a document which is future oriented, taking due account of societal developments. This has led to a code that is basically oriented to life cycle thinking. The content reflects this, following the sequel conceptual design, dimensioning, construction, conservation, and even dismantlement. Reliability plays an important part: various methods are offered to cope with this aspect. The choice depends on the case considered. The materials concrete and steel are described according to the latest state of the art. The design of concrete structures is described for a large number of conditions. This includes static and various types of non-static loading, a large range of temperatures and several environmental conditions. Design criteria are given with relation to reliability, functionality, durability and sustainability, where the last category is in the state of development. Moreover particular techniques are treated, like analysis of structures by numerical analysis and design by testing. The chapter on conservation of structures gives an insight into the degradation of concrete as a function of various types of environmental conditions. The analysis can be carried out with various levels of sophistication. Attention is given to non-traditional types of reinforcement as well, like steel fibres and FRP, which have reached a status of recognition in the previous years.

The Model Code 2010 does not intend to give only operational rules, although operationality has been an important target. The intention is as well to give backgrounds, show trends, and point out where further research is necessary.

The draft, as it is presented here, is open to suggestions for improvement. The draft is offered to the *fib* commissions, who are asked to react with regard to the available text. Here it should be emphasized that the so-called Esquillan principle should be followed. This principle, known from former CEB times, says that criticism is OK, as long as it is accompanied by proposals for improvement. After having taken due notice of comments and suggestions, the final document will be submitted to the *fib* General Assembly for definite approval.

The members of SAG5, the invited experts and the groups which gave essential input, like the JCSS (Joint Committee of Structural Safety) are gratefully acknowledged for their continuous engagement.

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Notations

Meaning of Roman capital letters

A	area
B	(void)
C	torsional moment of inertia
D	fatigue damage factor; diffusion coefficient
E	modulus of elasticity; earthquake action
F	action in general; local loading
G	permanent action; shear modulus
H	horizontal component of a force
I	second moment of a plane area
J	creep function
K	(permeability) coefficient
L	can be used for 'span; length of an element' in place of I
M	bending moment; coefficient of water absorption
N	axial force
O	(void)
P	force
Q	variable action
R	strength (resisting load effect); reaction at a support; resultant
S	load effect (M, N, I', T); static moment of a plane area
T	torsional moment; temperature
U	(void)
V	shear force, volume
W	modulus of inertia
X	reaction or force in general, parallel to x-axis
Y	reaction or force in general, parallel to y-axis
Z	reaction or force in general, parallel to z-axis

NOTE: Roman capital letters can be used to denote types of material, e.g. C for concrete, LC for lightweight concrete, S for steel, Z for cement.

Meaning of Roman lower case letters

a	deflection; distance; acceleration
b	width
c	concrete cover
d	effective height; diameter (see also h)
e	eccentricity
f	strength of a material
g	distributed permanent load; acceleration due to gravity
h	total height or diameter of a section; thickness
i	radius of gyration
j	number of days
k	all coefficients with dimension
l	span; length of an element
m	bending moment per unit length or width; mass; average value of a sample
n	normal (longitudinal, axial) force per unit length or width
o	(void)
p	prestressing
q	distributed variable load r radius
s	spacing; standard deviation of a sample
t	time; torsional moment per unit length or width; thickness of thin elements
u	perimeter

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v	velocity; shear force per unit length or width
w	width of a crack
x	co-ordinate; height of compression zone
Y	co-ordinate; height of rectangular diagram co-ordinate; lever arm

Use of Greek lower case letters

alpha	α	angle; ratio; coefficient
beta	β	angle; ratio; coefficient
gamma	γ	safety factor; density; shear strain (angular strain)
delta	δ	coefficient of variation; coefficient
epsilon	ε	strain
zeta	ζ	coefficient
eta	η	coefficient
theta	θ	rotation
iota	ι	(void)
kappa	κ	(to be avoided as far as possible)
lambda	λ	slenderness ratio; coefficient
mu	μ	relative bending moment; coefficient of friction; mean value of a whole population
nu	ν	relative axial force; Poisson's ratio
xi	ξ	coefficient; ratio
omicron	ο	(void)
pi	π	(mathematical use only)
rho	ρ	geometrical percentage of reinforcement; bulk density
sigma	σ	axial stress; standard deviation of a whole population
tau	τ	shear stress
upsilon	υ	(void)
phi	ϕ	creep coefficient
chi	χ	(to be avoided as far as possible)
psi	ψ	coefficient; ratio
omega	ω	mechanical percentage of reinforcement

Mathematical symbols and special symbols

S	sum
Δ	difference; increment (enlargement)
ø	diameter of a reinforcing bar or of a cable
’	(apostrophe) compression (only in a geometrical or locational sense)
e	base of Naperian logarithms
exp	power of the number e
π	ratio of the circumference of a circle to its diameter
n	number of ...
w/c	water/cement ratio
≯	not greater than: indicates the upper bound in a formula *
≮	not smaller than: indicates the lower bound in a formula *
<	smaller than
>	greater than

*: These symbols placed at the end of an expression indicate that where the result to which it leads is higher (or lower) than the limit given, then the values given should be taken into account and not the result obtained from the formula.

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General subscripts

a	support settlement; additional; accidental load
b	bond; bar; beam
c	concrete; compression; column
d	design value
e	elastic limit of a material
f	forces and other actions; beam flange; bending; friction
g	permanent load
h	horizontal; hook
i	initial
j	number of days
k	characteristic value
l	longitudinal
m	mean value; material; bending moment
n	axial force
o	zero
p	prestressing steel
q	variable load
r	cracking
s	ordinary steel; snow; slab
t	tension;* torsion;* transverse
u	ultimate (limit state)
v	shear; vertical
w	wind; web; wire; wall
x	linear co-ordinate
y	linear co-ordinate
z	linear co-ordinate
1, 2, 3	particular values of quantities
cc	conventional asymptotic value

NOTE: * When confusion is possible between tension and torsion, the subscripts tn (tension) and tr (torsion) should be used.

Subscripts for actions and action effects

a(*A*)	support settlement; accidental action
cc	creep of concrete
cd	delayed elasticity of concrete
cf	delayed plasticity of concrete
cs	shrinkage of concrete
ep	earth pressure eg(E) earthquake; seismic
ex	explosion; blast
eq (*E*)	forces and other actions
g(*G*)	permanent load
im	impact
lp	liquid pressure
m(*M*)	bending moment
n(*N*)	axial force
p(*P*)	prestress
q(*Q*)	variable load
s(*S*)	snow load
t(*T*)	torsion; temperature
v(*V*)	shear
w(*W*)	wind load

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Subscripts obtained by abbreviation

abs	absolute
act	acting
adm	admissible, permissible
cal	calculated, design
crit (or *cr*)	critical
ef	effective
el (or *e*)	elastic
est	estimated
exc	exceptional
est	external
fat	fatigue
inf	inferior
int	internal
lat	lateral
lim	limit
max	maximum
min	minimum
nec	necessary
net	net
nom	nominal
obs	observed
pl	plastic
prov (or *pr*)	provisional (stage of construction), provided
red	reduced
rel	relative, relaxation rep representative req required
res	resisting, resistant
ser	serviceability, service sup superior
tot	total
var	variable

Notation list

Roman lower case letters

$1 / r$	curvature of a section of an element
$1 / r_{(g)}$	curvature due to g
$1 / r_{(g+q)}$	curvature due to g and q
$1 / r_{0\,(g+9)}$	instantaneous (initial) curvature due to g and q
$1 / r_1$	curvature of an uncracked concrete section (state I)
$1 / r_{1\,r}$	curvature in state I under cracking moment
$1 / r_2$	curvature of a cracked concrete section (state II)
$1 / r_{2r}$	curvature in state II under cracking moment
$1 / r_{ts}$	tension stiffening correction for curvature
a	deflection
a_c	elastic deflection (calculated with rigidity $E_c I_e$)
b	breadth of compression zone or flange
b_{red}	reduced breadth of web
b_x	smaller side dimension of a rectangular section
b_y	greater side dimension of a rectangular section
b_w	breadth of web
c	concrete cover, concentration of a substance in a volume element
c_l	column dimension parallel to the eccentricity of the load

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c_2	column dimension perpendicular to the eccentricity of the load
c_{min}	minimum concrete cover
c_{nom}	nominal value of concrete cover (= c_{min} + tolerance)
d	effective depth to main tension reinforcement
d'	effective depth to compression reinforcement
d_{max}	maximum aggregate size
e	load eccentricity
e_0	first order eccentricity (= M_{Sd} / N_{sd})
e_{01}	smaller value of the first order eccentricity at one end of the considered element
e_{02}	greater value of the first order eccentricity at one end of the considered element
e_{tot}	total eccentricity
f_{bd}	design value of bond stress
f_c	cylinder compressive strength of concrete
f_c^*	cylinder compressive strength of concrete under triaxial loading (confined strength), reduced concrete strength due to transverse tension
f_{cc}	cylinder compressive strength of concrete under uniaxial stress
f_{cd}^*	design compressive strength of concrete under triaxial loading (confined strength), reduced design concrete strength due to transverse tension
f_{cd}	design value of f_c
f_{cd1}	average design strength value in an uncracked compression zone
f_{cd2}	average design strength value in a cracked compression zone
$f_{cd,fat}$	design fatigue reference strength of concrete under compression
f_{ck}	characteristic value of f_c
$f_{ck,cf}$	value of f_{ck} of confined concrete
$f_{ck.cube}$	characteristic value of cube compressive strength of concrete
$f_{ck,fat}$	fatigue reference compressive strength
f_{cm}	mean value of compressive strength f_c at an age of 28 days
f_{ct}	axial tensile strength of concrete (determined according to R1LEM CPC 7)
f_{ctd}	design value of f_{ct}
f_{ctk}	characteristic value of f_{ct}
f_{ctm}	mean axial tensile strength
$f_{ct,fl}$	mean flexural tensile strength (at T = 20°C)
$f_{ct,sp}$	mean splitting tensile strength
f_d	design value of strength
$f_{p0,1}$	0,1 % proof stress of prestressing reinforcement
$F_{p0,2}$	0,2% proof stress of prestressing reinforcement
$f_{p0,1k}$	characteristic 0,1% proof stress
$f_{p0,2k}$	characteristic 0,2% proof stress
f_{pt}	tensile strength of prestressing reinforcement
f_{ptd}	design tensile strength of prestressing reinforcement
f_{ptk}	characteristic tensile strength of prestressing reinforcement

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f_{py}	tension yield stress of prestressing reinforcement
$f_{py}d$	design value of tension yield stress of prestressing reinforcement
f_{pyk}	characteristic value of tension yield stress of prestressing reinforcement
f_R	relative (or projected) rib area
f_t	tensile strength of non- prestressing reinforcement
f_{tk}	characteristic value of tensile strength of non- prestressing reinforcement
f_y	tension yield stress of non- prestressing reinforcement
f_{yc}	strength of steel in compression
f_{ycd}	design strength of steel in compression
f_{yd}	design value of tension yield stress of non- prestressing reinforcement
f_{yk}	characteristic value of tension yield stress of non- prestressing reinforcement
g_d	design value of distributed permanent load
h	overall depth of member, total height; notional size of a member ($2\ A_c/u$; u: perimeter in contact with the atmosphere)
h_b	depth of beam
h_f	depth of flange
Δh_w	height of water column
i	radius of gyration
l	design span, effective span, length of an element, thickness of a penetrated section
Δl	measured elongation between two measuring points
l_0	design lap length, effective length (of columns); distance between measuring points
l_b	basic anchorage length
l_{bp}	basic anchorage length of pretensioned reinforcement
l_{bpd}	design anchorage length of pretensioned reinforcement
l_{bpt}	transmission length of pretensioned reinforcement
$l_{b,min}$	minimum anchorage length
$l_{b,net}$	design anchorage length
l_{ch}	characteristic length (fracture parameter)
l_p	development length for prestressing reinforcement
l_{pl}	plastic length (region in which tensile strain is larger than yield strain)
Δl_{pl}	residual elongation after unloading
$l_{p,max}$	length over which the slip between prestressing steel and concrete occurs
$l_{s,max}$	length over which the slip between steel and concrete occurs
l_t	transmission length
m	moment per unit width (out-of-plane loading); mass of substance flowing: degree of hydration
n	number of bars, number *of* load cycles; force per unit width (in-plane-loading)
n_{Ri}	number of cycles leading to failure at stress levels $S_{i,min}$ and $S_{i,max}$, respectively
n_{Si}	number of cycles applied at constant minimum and maximum stress levels $S_{i,min}$ and $S_{i,max}$, respectively
p	local gas pressure

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q	distributed variable load
q_d	design value of distributed variable load
r	radius
s	slip (relative displacement of steel and concrete cross-sections), shear slip (at interfaces); spacing of bars
s_{max}	maximum bar spacing
s_r	distance between cracks; radial spacing of layers of shear reinforcement
$s_{r,m}$	mean spacing between cracks
t	time, age, duration; thickness of thin elements
t_0	age at loading
t_s	concrete age at the beginning of shrinkage or swelling
t_T	effective concrete age
u	length of a perimeter; component of displacement of a point
u_0	length of the periphery of the column or load
u_1	length of the control perimeter for punching
u_{ef}	length of the perimeter of A_{ef}
u_n	length of the control perimeter for punching outside a slab zone with shear reinforcement
v	shear force per unit width (out-of-plane loading), component of displacement of a point
w	crack width; component of displacement of a point
w_c	crack width for $\sigma_{ct} = 0$
w_k	calculated characteristic crack width
w_{lim}	nominal limit value of crack width
x	depth of compression zone, distance
z	internal lever arm

Greek lower case letters

α	coefficient, reduction factor
α_e	modular ratio (E_s / E_c)
$\alpha_{e,p}$	modular ratio (E_p / E_c)
$\alpha_{e,sec}$	secant modular ratio ($E_{s,sec}$ / $E_{c,sec}$)
α_{ST}	coefficient of thermal expansion for steel
α_T	coefficient of thermal expansion in general
β	coefficient characterizing the bond quality of reinforcing bars
$\beta_c(t,t_0)$	coefficient to describe the development of creep with time after loading
γ	safety factor
γ_c	partial safety factor for concrete material properties
$\gamma_{c,fat}$	partial safety factor for concrete material properties under fatigue loading
γ_F	partial safety factor for actions
γ_G	partial safety factor for permanent actions
γ_Q	partial safety factor for variable actions
γ_s	partial safety factor for the material properties of reinforcement and prestressing steel

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$\gamma_{s,fat}$	partial safety factor for the material properties of reinforcement and prestressing steel under fatigue loading
δ_{jj}	node displacement
ε	strain
ε_c	concrete compression strain
ε_c^*	concrete compression strain under triaxial stress
ε_{cm}	average concrete strain within $l_{s,max}$
ε_{c0}	concrete strain at peak stress m compression
$\varepsilon_{cc}(t)$	concrete creep strain at concrete age $t > t_0$
$\varepsilon_{ci}(t_0)$	stress dependent initial strain at the time of stress application
$\varepsilon_{cn}(t)$	total stress independent strain at a concrete age t (= $\varepsilon_{cs}(t) + \varepsilon_{cT}(t,T)$)
$\varepsilon_{cs}(t,t_s)$	total shrinkage or swelling strain at concrete age t (t in days)
$\varepsilon_{c\sigma}(t)$	total stress dependent strain at a concrete age t (= $\varepsilon_{ci}(t_0) + \varepsilon_{cc}(t)$)
ε_{ct}	concrete tensile strain
$\varepsilon_{cT}(t,T)$	thermal strain at a concrete age t
ε_{cu}	ultimate strain of concrete in compression
ε_{d0}	strain of prestressed reinforcement corresponding to P_{d0}
ε_{pu}	total elongation of prestressing reinforcement at maximum load
ε_r	strain at the onset of cracking
ε_s	steel strain
ε_{s1}	steel strain in uncracked concrete
ε_{s2}	steel strain in the crack
ε_{sm}	mean steel strain
$\Delta\varepsilon_{sr}$	increase of steel strain in cracking state
ε_{sr1}	steel strain at the point of zero slip under cracking forces
ε_{sr2}	steel strain in the crack under cracking forces (σ_{ct} reaching f_{ctm})
ε_{sT}	thermal strain of steel
ε_{su}	strain of non-prestressing reinforcement at maximum load
$\Delta\varepsilon_{ts}$	increase of strain by the effect of tension stiffening
ε_u	total elongation of reinforcing steel at maximum load
ε_{uk}	characteristic total elongation of reinforcing steel at maximum load
ε_{yd}	design yield strain of non - prestressing reinforcement (= f_{yd} / E_s)
ε_v	transverse contraction
ζ	ratio of bond strength of prestressing steel and high-bond reinforcing steel
η	viscosity of gas
θ	angle between web compression and the axis of a member; rotation
θ_f	angle between inclined compression in a flange and the axis of the member
λ	slenderness ratio (= l_0 / i)
μ	coefficient of friction, relative bending moment
ν	relative axial force
ν_c	Poisson's ratio of concrete
ν_s	Poisson's ratio of steel
ν_{sd}	relative design axial force (= $N_{Sd} / A_c f_{cd}$)
ρ	ratio of (longitudinal) tension reinforcement (= A_s/bd)
$\rho_{s,ef}$	effective reinforcement ratio (= $A_s/A_{c,ef}$)
ρ_t	relaxation after t hours
ρ_w	ratio of web reinforcement (= A_{sw}/b_{ws} sinα)
σ	stress

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σ_1, σ_2, σ_3	principal stresses
σ_c	concrete compression stress
σ_{cd}	design concrete compression stress
σ_{ct}	concrete tensile stress
$\sigma_{c,ef}$	compression stress of confined concrete
$\sigma_{c,max}$	maximum compressive stress
$\sigma_{c,min}$	minimum compressive stress
$\sigma_{p0}(x)$	initial stress in prestressing reinforcement at a distance x from anchorage device
$\sigma_{p0,max}$	maximum tensile force in prestressing reinforcement at tensioning
σ_{pcs}	tendon stress due to prestress after all losses (due to creep and shrinkage)
σ_{pd}	tendon stress under design load
$\Delta\sigma_{Rsk}(n)$	stress range relevant to n cycles obtained from a characteristic fatigue strength function
σ_s	steel stress
σ_{s2}	steel stress in the crack
σ_{sE}	steel stress at the point of zero slip
σ_{sr2}	steel stress in the crack under crack loading (σ_{ct} reaching f_{ctm})
$\Delta\sigma_{Ss}$	steel stress range under the acting loads
τ_b	local bond stress
τ_{bm}	mean bond stress
$\tau_{fu,d}$	ultimate design shear friction capacity
τ_{max}	maximum value of bond stress
τ_{Rd}	resistance to shear stress (design value)
τ_{Sd}	applied shear stress (design value)
$\psi(t,t_0)$	relaxation coefficient
ω	mechanical reinforcement ratio
ω_{sw}	mechanical ratio of stirrup reinforcement
ω_v	volumetric ratio of confining reinforcement
ω_w	volumetric mechanical ratio of confining reinforcement
ω_{wd}	design volumetric mechanical ratio of confining reinforcement

Roman capital letters

A	total area of a section or part of a section (enclosed within the outer circumference)
A_1	section area in state I (taking into account the reinforcement)
A_c	area of concrete cross section or concrete compression chord
$A_{c,ef}$	effective area of concrete in tension
A_{core}	effectively confined area of cross-section in compression
A_{ef}	area enclosed by the centre-lines of a shell resisting torsion
A_p	area of prestressing reinforcement
A_s	area of reinforcement
$A_{s'}$	area of compressed reinforcement
A_{sh}	area of hoop reinforcement for torsion
A_{sl}	area of longitudinal reinforcement
A_{st}	area of transverse reinforcement
A_{sw}	area of shear reinforcement
$A_{s,cal}$	calculated area of reinforcement required by design
$A_{s,ef}$	area of reinforcement provided
$A_{s,min}$	minimum reinforcement area
D	fatigue damage, diffusion coefficient

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D_{lim}	limiting fatigue damage
E	modulus of elasticity
E_c	reduced modulus of elasticity for concrete
$E_c(t_0)$	modulus of elasticity at the time of loading t_0
E_{ci}	tangent modulus of elasticity at a stress σ_i (at T = 20°C)
$E_{c,sec}$	secant modulus of elasticity at failure for uniaxial compression ($E_{c,sec}=f_{cm} / \lvert\varepsilon_{c0}\rvert$)
E_P	modulus of elasticity of prestressing steel
E_s	modulus of elasticity of steel
$E_{s,sec}$	secant modulus of elasticity of steel
F	force, applied load or load effect
F_b	bond force transmitted along the transmission length
F_c	strut force (compression force)
F_d	design value of action
F_{pt}	tensile load of prestressed reinforcement
$F_{p0,1}$	characteristic 0,1 % proof -load
$F_{Sd,ef}$	effective concentric load (punching load enhanced to allow for the effects of moments)
F_t	tie force (tension force)
F_{ud}	ultimate dowel force
G	permanent action
G_F	fracture energy of concrete
G_{F0}	base value of fracture energy (depending on maximum aggregate size)
G_{inf}	favourable part of permanent action
G_{sup}	unfavourable part of permanent action
H	horizontal force, horizontal component of a force
I	second moment of area
I_1	second moment of area in state I (including the reinforcement)
I_2	second moment of area in state II (including the reinforcement)
I_c	second moment of area of the uncracked concrete cross-section (state I)
$J(t,t_0)$	creep function or creep compliance representing the total stress dependent strain per unit stress
K_g	coefficient of gas permeability
K_w	coefficient of water permeability
L	span, length of an element
M	bending moment; maturity of concrete
M_r	cracking moment
M_{Rd}	design value of resistant moment
M_{Sd}	design value of applied moment
M_u	ultimate moment
M_y	yielding moment
N	axial force, number of cycles to failure (fatigue loading)
N_r	axial cracking force
N_{Rd}	design value of resistance to axial force
N_{Sd}	design value of applied axial force
P_{d0}	design value of prestressing force (initial force)
$P_{k,inf}$	lower characteristic value of prestressing force
$P_{k,sup}$	upper characteristic value of prestressing force
P_m	mean value of prestressing force

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Q	variable single action; volume of a transported substance (gas or liquid)
R	resistance (strength); bending radius; universal gas constant
R_d	design resistance
RH	ambient relative humidity
RH_0	100% relative humidity
S	load effect (M, N, V, T); absorption coefficieni
ΔS_{cd}	stress range under fatigue loading
$S_{cd,max}$	design value of maximum compressive stress level (fatigue loading)
$S_{cd,min}$	design value of minimum compressive stress level (fatigue loading)
$S_{c,max}$	maximum compressive stress level (fatigue loading)
$S_{c,min}$	minimum compressive stress level (fatigue loading)
S_d	design load effect (M, N, V, T)
T	temperature, torsional moment
ΔT	temperature change
T_{Rd}	design value of resistance to torsional moment
T_{Sd}	design value of applied torsional moment
$T_{Sd,eff}$	effective design value of applied torsional moment
V	shear force; volume of gas or liquid
V_{Rd}	design value of resistance to shear force
V_{Sd}	design value of applied shear force
V_u	ultimate shear force
W_1	section modulus in state I (including the reinforcement)
W_2	section modulus in state II (including the reinforcement)
W_c	section modulus of the uncracked concrete cross-section (state I)
$W_{c,cf}$	volume of confined concrete
W_e	external work
W_i	internal work
$W_{s,trans}$	volume of closed stirrups or cross-ties

Others

ø	nominal diameter of steel bar
$ø_n$	equivalent diameter of bundles containing n bars
$ø_p$	diameter of prestressing steel (for bundles equivalent diameter)
$\phi(t,t_0)$	creep coefficient
ϕ_0	notional creep coefficient
Θ_{pl}	plastic rotation capacity
ΣU	total perimeter of rebars

Statistical symbols

Roman lower case letters

$f_x(x)$	probability density function (of normal distribution)
$f_r(r)$	probability density function (of log-normal distribution)
$f_R(r)$	probability density function of resistance
$f_S(s)$	probability density function of action
k	normalised variable or fractile factor
m_x	mean (same meaning as $\overline{x}$)
m_R	mean of resistance

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m_S	mean of action
p_f	failure probability
$\breve{x}$	median
$\hat{x}$	modal value
$\overline{x}$	mean (same meaning as m_x)
x_p	p-%-fractile

Greek lower case letters:

α	sensitivity factor
β	reliability index
γ	(partial) safety factor
σ_x^2	scattering or variance
σ_x	standard deviation
σ_R	standard deviation of resistance
σ_S	standard deviation of action

Roman capital letters:

$F_r(r)$	probability distribution function (of log-normal distribution)
$F_x(x)$	probability distribution function (of normal distribution)
R	resistance
S	action
V_x	coefficient of variation
Z	safety zone (difference of R and S)

Others

$\Phi(k)$	normalized function

The Model Code for Concrete Structures was an initiative taken by CEB and FIP in the time that there were hardly international codes. Since in those former days CEB and FIP were organisations aiming at synthesis of international research and experience, it was regarded as an important step forward to convert this knowledge and experience into practical documents for design, so that national code commissions could take advantage of it. The first code-like recommendations in 1964 and 1970 were used in this way. The Model Code 1978 contributed as well to international harmonization. The Model Code 1990 confirmed the use according to the initial intention, by serving as an important basis for the most recent version of Eurocode 2.

The format of this *fib* Model Code follows the earlier CEB-FIP tradition:

- on the right-hand side the main provisions are presented in the logical sequence of topics. Structural requirements are stated, followed by the relevant design criteria, i.e. appropriate engineering models and/or design rules: their application is intended to satisfy the relevant structural requirement.
- on the left-hand side explanations are given, with specific diagrams, alternative simplified rules, short justifications of options found on the right-hand side and references to other sources. Those other sources can be *fib* Bulletins, CEB-FIP Bulletins or other documents.

Assuring a defined service life means:

- use of materials with satisfactory properties;
- definition of appropriate limit states for durability;
- verification with regard to the probability of exceeding those limit states;
- a good conceptual design;
- appropriate construction methods;
- a plan for life cycle management.

1 Scope

1.1 Aim of the Model Code

The Model Code for Concrete Structures is a document that is meant to serve as a basis for future codes. Where existing operational codes are legal documents, based on mature knowledge, the Model Code also takes into account new developments with regard to concrete structures, the structural material concrete and new ideas with regard to requirements to be formulated for structures to achieve optimum behaviour according to new insights and ideas. In this Model Code, those new ideas do not only refer to classical demands with regard to safety and serviceability, but also take account of the increasing significance of design criteria for durability and sustainability.

With respect to those who will be involved in updating existing codes or develop new codes for concrete structures, the Model Code should be a source of information. Where a normal operational code gives predominantly sets of application rules which should be transparent enough to be applied by professional designers and in the mean time be accurate enough to be economical, the Model Code intends to give, additionally, sufficient background information.

Nevertheless the Model Code is meant to be an operational document also for normal design situations and structures.

1.2 Main innovating aspects

Compared to Model Code 1990, a number of new innovative aspects have been introduced. The most important is the introduction of “time” as a design parameter. “Time” is related to the awareness that a structure should not only be sufficiently safe and serviceable, but should as well have sufficient resistance against deterioration. In this respect not only the intrinsic resistance at the start of the service life of the structure should satisfy the relevant design criteria, but a strategy for maintenance should also have been developed.

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Verification of the limit states shall be realised by a probability-based method. This Model Code recommends for verification of the limit states to use one of the following safety formats:

- probabilistic safety format;
- partial safety factor format;
- global resistance format;
- deem-to-satisfy-approach;
- design by avoidance.

Material properties are described for a large range of loads (static and dynamic, short and long term). Moreover the most relevant material properties with regard to durability are treated, like the resistance against transport of liquids and gases through hardened concrete by diffusion, capillary suction and leaching. For fibre concrete testing methods are defined to derive the properties relevant for design.

The separate treatment of interface characteristics is especially for the design of hybrid systems. Hybrid systems can be not only composite steel – concrete structures, but also structural systems arising from refurbishment or upgrading of existing structures (additional concrete layers or strengthening by externally glued materials).

Another new tendency is the provision of a basis for performance-based design. This requires not only the definition of appropriate limit states, but also solid knowledge of material properties, a suitable definition of loads or combination of loads and methods of verification.

An extensive state-of-the–art with regard to material properties for structural concrete is given. This includes constitutive relations for concrete up to a strength class C120, and properties of reinforcing and prestressing steel, including prestressing systems. Special attention is given to the application of fibre concrete for structural applications. Moreover the application of non-metallic reinforcement is dealt with.

Special attention is given to interface characteristics. This concerns bond of embedded and external reinforcement, new concrete to old concrete and concrete to structural steel.

Design rules for serviceability and ultimate limit states are given for a wide range of conditions, like static and non-static loading (fatigue, impact and explosion, earthquake), high and low temperatures (fire and cryogenic). Furthermore limit states for durability are given. Design for sustainability will be an important task in future. Here initial ideas are given. Design for robustness is also given due attention.

Special attention is also devoted to verification assisted by numerical simulations and verification assisted by testing.

Since the quality of construction is of large significance for the durability of the structures, adequate attention is given to a number of important construction aspects.

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An important chapter within the scope of design for service life is Conservation. This chapter deals with conservation strategies and conservation management, condition survey and conditions assessment, decision making, interventions and recording.

To complete the idea of life cycle design, a final short chapter on dismantlement, removal and recycling is included. In the future this aspect deserves better integration within life cycle design.

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2 Terminology

2.1 Definitions

This section defines the various technical terms that appear in this Model Code. Definitions are based on the sources listed in section 2.2.

Acceptance: The process of handing over the structure or a part of it to the stakeholders (i.e. owners, users, contractors, society) .[6]

Accidental action: Action with low probability of occurrence, usually of short duration and of considerable effect compared to the variable action.[6, 8]

Accidental design situation: Design situation taking into account accidental conditions for the structure or its component under consideration.[6]

Accompanying action: An action accompanying the loading action.[6]

Action effect: Response of a structure to actions.[6]

Actions: Mechanical loadings and other physical, chemical and biological actions on the structure, resulting from the execution and the use of the structure as well as from the environmental influences.[6]

Adverse state: State where the performance criterion is not met.

Aesthetics of structures: Aspects of the appearance of a structure perceived in terms of visual aesthetic considerations.

Examples of the *action effects* are stresses, stress resultants, reactions, deformations, displacements, as well as other effects, depending on the type of structure.[6]

Aesthetics of structures is usually associated with the visual sense and, to some extent, the senses of sound and texture, as well as with the perception of the recognised associations and the context.

Although any person's response to the aesthetics of a structure will be unique to that individual, many aesthetic principles can be identified and used by the creator of the structure to achieve specific aesthetic effects. Effects relevant for structures include for instance repetition, symmetry/asymmetry, rhythm, perspective, proportion, harmony, contrast, pattern, ornamentation, texture, colour, granularity, the interaction of sunlight and shadows.

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In order to derive an *analytical model* use is made of basic relationships such as equilibrium conditions, constitutive relationships and kinematic conditions.

Availability refers to the probability that a structure is actually available for use at the period of time when it is supposed to be available.[10]

The *birth certificate* should provide specific details on parameters important to the durability and service life of the structure concerned (e.g. cover to reinforcement, concrete permeability, environmental conditions, quality of workmanship achieved, etc.) and the basis on which future knowledge of through-life performance should be recorded.[1]

The framework laid down in the *birth certificate* should provide a means of comparing actual behaviour / performance with that anticipated at the time of design of the structure.[1]

The *birth certificate* should facilitate ongoing (through-life) evaluation of the service life which is likely to be achieved by the structure.[1]

Analytical model: Mathematical relationship between the forces exerted on the structure or a structural element and its response to those forces (e.g. deformations, displacements or internal forces).

Assessment: see *Condition assessment*.

Availability: The ability of a structure to operate satisfactorily at any point in time, excluding times when the structure is under repair.[10]

Basic variable: Part of a specified set of variables representing physical quantities, which characterise actions and environmental influences, geometrical quantities, and material properties.[5]

Basis of design: Technical description of the implementation of the service criteria agreement.[6]

Bearing: Device to transfer mainly compressive vertical force for supporting an element.

Biological actions: The aggression of biological organisms (bacteria, insects, fungi, algae) affecting and influencing the structure or its components.

Birth certificate: A document, report or technical file (depending on the size and complexity of the structure concerned) containing engineering information formally defining the form and the condition of the structure after construction.[1]

Capacity design: Method of seismic design with appropriately defined areas of plastic deformations exhibiting adequate ductility, together with other areas of the structure that are provided with increased ultimate resistance to ensure elastic behaviour.[6]

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Collapse may be a sudden occurrence, giving limited warning of the impending calamity.[1]

Characteristic value of a material property: The value of a material property (e.g. structural material or soil) having an a priori specified probability of not being attained in the supply produced within the scope of the relevant material standard.[9] The characteristic value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product. A nominal value is used as the characteristic value in some circumstances.[4]

Characteristic value of a geometrical property: Value usually corresponding to the dimensions specified in the design.[9] Where relevant, characteristic values of geometrical quantities may correspond to some prescribed fractiles of the statistical distribution. [4]

Characteristic value of an action: Principal representative value of an action.[9]

Chemical actions: The reactive transport of chemicals (e.g. salts, acids, alkaline substances and organic compounds) affecting and influencing the structure or its components.

Collapse: Catastrophic physical disruption, giving-way or breakdown of elements or components of a structure, to such an extent that the structure is unable to perform its intended load-bearing function.[1]

Commissioning: Start of planned use.[6]

Composite element: an element made of both precast and in-situ concrete, with or without metallic connectors.

Conception: Identifying, developing and assessing different design alternatives.

Conceptual design: All the activities and developments leading from the design criteria to a suitable structural solution.

Condition assessment: A process of reviewing information gathered about the current condition of a structure or its components, its service environment and general circumstances, whereby its adequacy for future service may be established against specified performance requirements for a defined set of loadings and / or environmental circumstances.

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The term *'condition assessment'* may be used more commonly in connection with damaged or deteriorated structures.[1]

Condition evaluation would generally consider the requirement for any subsequent intervention which may be needed to meet the specified performance requirements.

A wide range of parameters might be included within *condition survey*, with data being obtained by activities such as visual inspection and various forms of testing. *Condition survey* would also seek to gain an understanding of the (previous) circumstances which had lead to the development of that state, together with the associated mechanisms causing damage or deterioration.

Conservation activities may involve restoring the current condition of a structure to a satisfactory state, or include preventative measures which aim to ensure that the future condition of a structure remains within satisfactory bounds, or improvements to meet revised performance requirements. For this, consideration may need to be given to the effects of potential future deterioration.

Condition control: The overall through-life process for conserving the condition of a structure involving condition survey, condition assessment, condition evaluation, decision-making and the execution of any necessary interventions; performed as a part of the conservation process.

Condition evaluation: Similar to *Condition assessment*, but may be applied more specifically in respect of suitability against a particular criterion, such as a specified loading.[1]

Condition survey: A process whereby information is acquired relating to the current condition of the structure with regard to its appearance, functionality and / or ability to meet specified performance requirements with the aim of recognizing important limitations, defects and deterioration.

Configuration: Creation of an aesthetic expression by means of spatial arrangement, shaping and choice of structural materials.[1]

Connection: The whole of D-regions affected by force transfer between two or more elements.

Conservation: Activities and measures taken which seek to ensure that the condition of a structure remains within satisfactory bounds to meet the performance requirements for a defined time; that is in respect of structural safety, serviceability and sustainability requirements, which may include considerations such as aesthetics.

Conservation plan: The overall plan for controlling and conserving the condition of a structure; i.e. condition survey, condition assessment, condition evaluation, decision-making and the execution of any necessary interventions.

Construction: see *Construction process*

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For comparison, see definition of *Structural materials*.

The *construction* is deemed to include any necessary preparatory works (e.g. excavation, landfill, etcetera) and finishing works required to be carried out at a particular site or location to facilitate the creation of the desired entity (e.g. bridge, etc).[1]

Construction products are either construction materials or various components, elements and assemblies made of construction materials, which are use during construction.

Cumulative knowledge of through-life performance concerns the evolution of certain properties or parameters relevant to the durability of the structure, the magnitude and number of structural loadings (especially if fatigue effects are of potential concern), data on the characteristics of the environment(s) affecting the structure, etc.[1]

Construction documents: Contract documents, construction programmes, minutes of meetings and records of construction inspections, together with the daily record of work carried out.[6]

Construction inspection plan: Specifying the type, extent, execution and timing of construction inspections, including information on quality requirements and admissible deviations as well as resolving questions of responsibilities and information flow.[6]

Construction inspections: Checking whether the design specifications are implemented correctly during exection.[6]

Construction materials: Structural and non-structural materials used in a construction process.

Construction: The overall physical process whereby construction products are brought together to create a building, an item of infrastructure or other man-made facility or asset.[1]

Construction product: Any product that is manufactured for permanent installation in a building, an item of infrastructure or other man-made facility or asset.

Construction work: Carrying out the construction according to contract.[6]

Construction works documents: Documents specific to construction works.[6]

Control measurement: Measurement to monitor selected physical quantities (e.g. geometrical characteristics or structural deformations).[6]

Cumulative knowledge of through-life performance: Information on the performance of a structure, based on systematic gathering and evaluation of data during the service life.[1]

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Defects may be in-built or may be the result of deterioration or damage.[2]

Design of structures (process) may be subdivided into *conceptual design*, *structural analysis* and *dimensioning*.

In the context of performance-based design, sets of *performance requirements* are used as input into the *design of structures*. Therefore performance-based *design of structures* shall be preceded by the *conceptual design* including *requirements development phase* (which may be preceded by a *feasibility study* of the project).

Damage: Physical disruption or change in the condition of a structure or its components, brought about by external actions and influences, such that some aspect of either the current or future performance of the structure or its components will be impaired.[1]

Decommissioning: Discontinuation or interruption of use.[6]

Degradation: Worsening of condition with time - see deterioration.[1]

Defects: A specific *deficiency* or inadequacy in the structure or its components which affects their ability to perform according to their intended function at the required level, either now or at some future time.[1]

Deficiency: Lack of something, possibly arising as a result of an error in design or construction, which affects the ability of the structure to perform according to its intended function, either now or in the future.[1]

Deformation capacity: (Elastic and/or plastic) deformation of a structure or a structural component reached at failure or at other defined state of loading.

Demolition: The process of dismantling and removal of existing structures.[1]

Design: Developing a suitable solution, taking due account of functional, environmental and economical requirements.

Design alternatives: Feasible alternatives to solve the design assignment.

Design boundary conditions: Space, time, legal, financial, structural, material-, execution- and service-related conditions for design.[6]

Design criteria: see *Performance criteria*.

Design of structures: Process of developing a suitable solution, taking due account of safety, functionality and sustainability of a structure.

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The *design situations* considered shall include all foreseeable conditions that can occur during execution and use. The design shall demonstrate that for the identified design situation the relevant limit states are not exceeded.[4]

The *design value of a geometrical property* is generally a *nominal value*.

Where relevant, the *design value of a geometrical property* may be equal to the characteristic value and correspond to some prescribed fractile of the statistical distribution. However, it may be treated differently in cases where the limit state under consideration is very sensitive to the value of the geometrical property.[5]

Alternatively, the *design value of a geometrical property* can be established on a statistical basis, with a value corresponding to a more appropriate fractile (e.g. rarer value) than applies to the characteristic value.[5]

Typically *deterioration* of the structure or its components will be driven by chemical, mechanical or physical processes or agents, or combinations thereof.[1]

Design service life: see *Specified (design) service life*

Design situations: Sets of defined actions and physical conditions representing the real situation expected during a specified time interval, for which the design is performed.

Design value of a geometrical property: Specified minimum or maximum value of geometrical dimension, which should not be exceeded.

Design value of an action: Value obtained by multiplying the representative value by the partial safety factor.[4]

Design value of material or product property: Value obtained by dividing the characteristic value by a partial safety factor, or, in special circumstances, by direct determination.[4]

Desired state: State where the performance criterion is met.

Destruction: Damage to a structure that is of such severity that repair is not a practical or viable option.

Detailing: Determining the dimensions of structural components in local areas of the structure and specifying the structural details.

Deterioration: Worsening of condition with time, or a progressive reduction in the ability of a structure or its components to perform according to their intended functional specifications.[1]

Deterioration mechanism: (Scientifically describable) process of the cause and development of deterioration.[6]

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The term *'diagnosis'* is typically applied to forms of deterioration and degradation or other mechanisms causing an alteration in the expected or desired behaviour of the structure or its components.[1]

Dimensioning is usually performed in combination with numerical verifications.[6]

In the context of performance-based *design of structures*, *durability* refers to the fulfilment of the performance requirements within the framework of the planned use and the foreseeable actions, without unforeseen expenditure on maintenance and repair.[6]

Environmental influences need to be taken in to account during planning of service life, design and construction of a particular structure or asset.[1]

Environmental influences may need to be considered at different scales ranging from macro level (affecting the overall structure), meso level (affecting an individual element or component) down to micro level (localised influences).[1]

Diagnosis: Identification of the cause or explanation of the mechanism(s) by which a phenomenon affects the behaviour or the condition of a structure or its components based on an investigation of signs and indications exhibited therein.[1]

Dimensioning: Determining the dimensions, the structural materials (including their properties) and the detailing of a structure on the basis of structural and execution-related considerations.[6]

Dimensioning criteria: see *Design criteria*

Dimensioning situations: see *Design situations*

Dimensioning value: see *Design value*

Disintegration: Severe physical damage and disruption of a structure or its components which results in its (localised) break-up into fragments, with the possibility of gross impairment of their functional capability.[1]

Dismantlement: Planned demolition of a structure with separation of the structural members and structural materials, fulfilling disposal requirements.[6]

Ductility: Plastic deformation capacity characterised by irreversible deformations and energy dissipation, usually related to the limit of the elastic behaviour.[6]

Durability: The capability of structures, products or materials of continuing to be useful after an extended period of time and usage.[10]

Economy: Moderate use of financial means and natural resources in relation to the whole period of design, execution and service.[6]

Environmental influences: Physical, chemical and biological actions resulting from the atmospheric conditions or characteristics of the surroundings to the structure (loadings associated with wind or wave effects belong to mechanical loadings).

Estimate: Estimated mean value of a quantity.[6]

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In the context of Limit State Design, *failure* is reached when the criteria of the limit state under consideration are not met.

In the context of performance-based design, a *feasibility study* may be carried out before starting the r*equirements development phase* and the *design of structure.*

Soil can be built on (e.g. foundations to structures), built in (e.g. tunnels, culverts, basements), built with (e.g. roads, runways, embankments, dams) or supported (e.g. retaining walls, quays).

Often the term *'ingress'* is associated with the entry of substances which cause deterioration (e.g. chlorides into reinforced or prestressed concrete, sulphates and carbon-dioxide (CO_2) into concretes, etc.).[1]

Examination: Condition survey and evaluation including recommendation of remedial measures occasioned by special circumstances.[6]

Execution: All the activities and measures involved in the physical creation of a structure, including preparation for construction.[6]

Failure: The state where the performance level of a structure or a structural element is inadequate.

Fatigue resistance: Ultimate resistance under frequently repeated actions.[6]

Feasibility study: Preliminary analysis of all possible solutions to a problem and a recommendation on the best solution. A *feasibility study* is undertaken to ascertain the likelihood of the project's success.

Fixed action: Action with fixed distribution over the structure or structural member; everywhere the magnitude and the direction follow clearly from the information at a point.[6]

Free action: An action whose distribution over the structure is not fixed.[6]

Geometrical properties: Planned dimensions and unwanted imperfections of a structure.[6]

Soil: Subsurface material (e.g. sand, silt, clay, gravel, boulders or rock) in the area under or adjacent to a structure.

Hazard: An occurrence which has the potential to cause deterioration, damage, harm or loss.[1]

Hazard scenario: Critical situation characterised by a leading hazard and defined circumstances.

Ingress: The entry of substances into structural and / or non-structural components of a structure.[1]

Inspection: A primarily visual examination, often at close range, of a structure or its components with the objective of gathering information about their form, current condition, service environment and general circumstances.[1]

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Integration: Adaptation of a structure to the natural and manmade environment.[6]

Intervention: A general term relating to an action or series of activities taken to modify or preserve the future performance of a structure or its components.

Interventions may be undertaken as a preventative intervention (applying some form of treatment / taking action to ensure that the condition of a structure remains within satisfactory bounds / that an unsatisfactory performance condition is not reached) or as a reactive intervention (taking action after damage has become visible e.g. cracking or spalling of concrete).[1]

Interventions may be planned or unplanned. Planned interventions tend to be classified as a maintenance intervention. Un-planned interventions tend to be classified as a repair intervention.

Interventions might be instigated for the purposes of, for example, repair, rehabilitation, remediation of the structure concerned.[1]

Inventory may be established to assist in the management of the structures.[1]

Inventory: Detailed list or register of items or elements, possibly classified by type, function or some other principal attributes.[1]

The process of inquiry might employ sampling, testing and various other means of gathering information about the structure, as well as theoretical studies to evaluate the importance of the findings in terms of the performance of the structure.[1]

Investigation: The process of inquiry into the cause or mechanism associated with some form of deterioration or degradation of the structure and the evaluation of its significance in terms of its current and future performance. The term may also be employed during the assessment of defects and deficiencies.[1]

Irreversible serviceability limit states: serviceability limit states where some consequences of actions exceeding the specified service requirements will remain when the actions are removed.[4]

Leading action: Main action in a load case.[6]

Leading hazard: Main hazard in a hazard scenario.[6]

Limit state represents the transition between the *desired state* and the *adverse state* (*failure*).

Limit state: State beyond which the structure no longer satisfies the relevant performance criteria.[4]

Load: see *Mechanical loading*

Load case: A physically compatible combination of simultaneously occurring actions considered for a specific verification.[6]

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Maintenance activities involve recurrent or continuous measures which enable the structure to fulfil the requirements for reliability.[2]

The term 'maintenance' is commonly applied in the context of building fabric components with a limited life, components associated with water management and rainwater run-off, items where regular intervention is required to maintain their effective operation etc. The term *'maintenance'* is commonly applied ancillary items such gutters, drains, sealants, movement joints, bearings, etc.

Maintainability refers to the probability that an item will be restored to specified conditions within a given period of time when maintenance action is performed in accordance with prescribed procedures and resources.[10]

Management of structures often involves conflicting requirements and objectives, which invariably requires compromise and judgement about the action to be taken or not taken due to limitations in the available resources.[1]

Structural *monitoring* typically involves gathering information by a range of possible techniques and procedures to aid the management of an individual structure or class of structures. It often involves the automatic recording of performance data for the structure and possibly some degree of associated data processing. Strictly this does not need to be so, there being a variety of means of gathering appropriate data.[1]

Maintenance: A set of planned (usually periodic) activities performed during the service life of the structure intended to either prevent or correct the effects of minor deterioration, degradation or mechanical wear of the structure or its components in order to keep their future serviceability at the level anticipated by the designer.[1]

Maintenance plan: Instructions for the maintenance specific to the structure considered.[6]

Maintainability: The ability of a structure to meet service objectives with a minimum expenditure of maintenance effort under service conditions in which maintenance and repair are performed.[10]

Management (of structures): Processes and procedures adopted for the maintenance, inspection, testing, assessment and repair or other remedial action of structures in order to provide effective control against (pre-determined) criteria to ensure the continued safe service of individual structures or wider groupings of structures and related assets.[1]

Material: Metal, non-metallic inorganic or organic material with useful technical properties.[6]

Mechanical loading: (External) pressure, force or imposed displacement to which the structure or its components are subjected.

Method of construction: Manner in which the construction is carried out.[6]

Modification: Making changes to a structure for the purpose of adapting it to new requirements.[6]

Monitoring: To keep watch over, recording progress and changes in materials and / or structural properties with time; possibly also controlling the functioning or working of an associated entity or process (e.g. warning alarms based upon parameters such as applied load, element deflection or some aspect of structural response).[1]

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Monitoring is taken to involve similar activities as survey, but with measurements being undertaken on an ongoing and possibly quasi-continuous basis. Monitoring could involve installed instrumentation. If so, this will introduce different forms of measurement and data gathering to that used for a survey. In some circumstances, these activities might possibly include various forms of local / global response measurement or testing.

The *nominal value* of a material or a product property is normally used as a characteristic value and established from an appropriate document such as a standard.[5]

The uncertainties in material properties are dealt with by the partial safety factor for a material property. The uncertainties of the (resistance) models (including geometric deviations associated with, if these are not modelled explicitly) are dealt with by the partial safety factor for (resistance) model. The uncertainties in actions are dealt with by the partial safety factors for loads and environmental actions.

Monitoring plan: Instructions for the monitoring specific to the structure.[6]

Nominal value: Value fixed on a non-statistical basis, for instance on acquired experience or on physical conditions, or a planned prescribed value.[4, 6]

Objective of protection: Qualitative and quantitative specification of the requirements of a structure for the case of accidental occurrences and conditions.[6]

Observation: Examining the serviceability by simple and regular checks.[6]

Observational method: Possible procedure in the case of insufficiently reliable basic information for the design, execution and use of a structure, involving certain acceptable risks, a prediction of behaviour, and the specification of associated limit values together with corresponding monitoring and safety measures.[6]

Operational instruction: Instructions for the owners and users on the handling and operation of the technical equipment.[6]

Overall stability: State of stable equilibrium for the whole structure as a rigid body.[6]

Partial damage limit state (*PDLS*): a limit state between the SLS and ULS.[3]

Partial safety factor: A factor employed to deal with the uncertainties in the model variable.

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In many instances the term *'penetration'* is used interchangeably with the term *'ingress'*, but it may also be used in the context of evaluating the depth to which a deleterious agent has penetrated the component concerned (e.g. chlorides have penetrated to the depth of the reinforcing steel).[1]

The term *'penetration'* may also be associated with the introduction of agents which will help extend the service life of the structure (e.g. the introduction of resins or corrosion inhibitors into concrete, etc).[1]

In the context of Limit State Design, *performance criteria* are the limit values that describe for each limit state the conditions to be fulfilled.[5]

A *performance indicator* is associated with and gives meaning to the *performance criteria* used to define the *performance requirements* for a design, an actual, a potential or an intended intervention option.[1]

Performance requirements are established by means *performance criteria* and associated *performance indicators* and constraints related to *service life* and *reliability*.

Performance requirements refer to the fulfilment of the essential demands of the stakeholders (i.e. owners, users, contractors, society) during the intended life time of structures or structural elements.[1]

Passive state / Passivity: The state in which, by virtue of a protective oxide film, steel does not spontaneously corrode.[2]

Penetration: The entry of substances, especially moisture, into structural and / or non-structural components of the fabric of a building or structure.[1]

Performance: The behaviour of a structure or a structural element as a consequence of actions to which it is subjected or which it generates.

Performance aspect: Aspect of the behaviour of a structure or a structural element for a specific action to which it is subjected or which it generates.

Performance criteria: Quantitative limits, associated to a *performance indicator*, defining the border between desired and adverse behaviour.

Performance indicator: A measurable / testable parameter (i.e. characteristic of materials and structures) describing quantitatively a performance aspect.

Performance level: Qualification of a structure or a structural element, which is established by verifying its behaviour against the performance requirements. A satisfactory performance level is reached when a structure or a structural element has demonstrated a sufficient behaviour to meet the performance requirements. In the opposite case, the performance level of a structure or a structural element is considered to be unsatisfactory.

Performance requirement: A condition for a design, an actual, a potential or an intended intervention option, that the performance criterion shall be met during the service life with appropriate reliability and in a sustainable way.

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Sets of *performance requirements* are used as input into the performance-based design of structures.

Physical actions are usually caused by change of humidity or temperature (e.g. shrinkage, creep, fire exposure, heating and cooling, freeze–thaw, salt weathering) or movement of agents of wind, water, solid, ice (e.g. water erosion, wind erosion).

Products that are commonly fabricated by precasting include beams and joists, slab units, wall panels, columns, and utility items such as pipes and ducts.[10]

The situation may include circumstances where the performance requirements have changed over time or where the planned service life has been extended. It is implied that the treatment or action will be taken prior to deterioration and damage becoming apparent / visible on the structure; e.g. cracking or spalling of concrete.

In the context of the Model Code, the *(Owner's) Professional Team* means those engaged or commissioned by the *stakeholders* to advise and assist through the appropriate provision of technical and related services. Some, possibly all, of the individuals may reside within the entity or organisation owning the facility concerned.[1]

Protection involves an action or series of actions undertaken to seek to defend a structure from the effects of further or future deterioration by providing a physical or chemical barrier to aggressive species (e.g. chloride

Permanent action: Action almost constant or monotonically approaching a limiting value during a reference period.[6]

Persistent design situation: Design situation which is decisive during a period of the same order of magnitude as the design service life.[6]

Physical actions: Physical phenomena other than mechanical loadings (e.g. hydro-thermal processes, weathering, erosion processes) affecting and influencing the structure or its components.

Precast concrete: Concrete that is cast into forms and cured at a location other than its final position in a structure.[10]

Precast element: element manufactured in compliance with a specific product standard in a factory - or in a location other than its final position in the structure - protected from weather conditions.

Precast structure: a structure made of precast elements.

Preparation for construction: Invitation to tender, tendering, evaluation of tenders, conclusion of contract for materials and work, as well as preparation of construction work.[6]

Preventative intervention: A pro-active conservation activity concerned with applying some form of treatment or taking action prior to a change in a material property (e.g. such as that caused by the influence of carbonation or chlorides) adversely affecting the ability of the structure, or parts thereof, to meet the required performance levels because of deterioration.

(Owner's) Professional Team: A group of persons, generally from one or more organisations, who together are skilled in the various technical aspects and processes required for the design, construction and maintenance of buildings, works and other facilities of public or commercial utility.[1]

Protection: A measure which prevents or reduces the development of defects.[2]

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ions) or other deleterious environmental agents and loadings upon the in-service performance and durability of a structure. Typically this will often be provided by surface coatings, impregnation treatments, overlays, membranes, electro-chemical treatments, enclosure or surface wrappings applied to the concrete structure, elements or parts thereof.[1]

Typically, the *prudent estimate* is concerned with soil properties.

Typically, *recalculation* is concerned with in-service performance assessment and structural load capacity in particular. The process may utilise similar steps and procedures to design but fundamentally differs from this by seeking to take into account the actual form and condition of the structure as found, including deterioration. This will often include a more realistic consideration of the actual loading regimes, rather than the idealised values used in design. The recalculation process may be used to predict future structural performance taking into account the influence of ongoing deterioration processes and any remediation actions.[1]

Prudent estimate: A value which compared to the estimate is provided with an adequate margin to meet the required reliability.[6]

Reactive intervention: A re-active conservation activity undertaken after deterioration and / or damage has become apparent / visible (e.g. cracking or spalling of concrete) such that, because of the deterioration, this has adversely affected the ability of the structure, or parts thereof, to meet the required performance levels (which may include consideration of issues such as aesthetics).

Re-birth certificate: A document, report or technical file similar to the birth certificate for a structure, but related to the information and circumstances associated with a project for the repair / remediation / refurbishment of the structure or a part thereof to extend its anticipated service life.[1]

Rebuild: To create a new structure or structural component to replace the original damaged, defective or deteriorated entity after its destruction or demolition, without restriction upon the materials or methods employed.[1]

Recalculation: A process of analytical examination using mathematical models or simplified representations of a structure or elements thereof to make an estimate of its performance.[1]

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Generally, *reconstruction* is concerned with meeting specific objectives such as strength or future durability requirements.[1]

The aim of *rehabilitation* is in principle similar to the aim of reconstruction, but possibly with greater emphasis upon the serviceability requirements associated with the existing or proposed revised usage of the structure.[1]

In some instances, the *rehabilitation* may not be intended to bring the structure or its components back to the original level of serviceability or durability. The work may sometimes be intended simply to reduce the rate of deterioration or degradation, without significantly enhancing the current level of serviceability.[1]

In the context of performance-based *design of structures*, *reliability* refers to the ability of a structure or a structural member to fulfil the performance requirements during the service life for which it has been designed[5] at a required failure probability level corresponding to a specified reference period.

Possible *remedial interventions* are widely ranging and may involve structural strengthening through to preventative measures, such as applying surface coatings to provide a barrier to the ingress of deleterious environmental agents (e.g. chloride ions). The situation may include circumstances where the performance requirements have changed over time or where the planned service life has been extended.

Reconstruction: To restore or reinstate all or part of a structure or component which is in a changed, defective or deteriorated state to its original or higher level of performance, without restriction upon the methods or materials employed.[1]

Record of construction: Collection of construction works documents updated according to the state of the execution.[6]

Reference period: Chosen period of time that is used as a basis for assessing statistically variable actions, and possibly for accidental actions.[4]

Rehabilitation: Intervention to restore the performance of a structure or its component parts that are in a changed, defective, degraded or deteriorated state to the original level of performance, generally without restriction upon the materials or methods employed.[1]

Reliability: Ability of a structure or a structural member to perform its intended function satisfactorily (from the viewpoint of the customer) for its intended life under specified environmental and operating conditions.[10] Reliability is usually expressed in probabilistic terms.[5]

Reliability differentiation: Measures intended for socio-economic optimisation of the resources to be used to build structures, taking into account all expected consequences of failures and the cost of the structures.[4]

Remediation: see *Remedial intervention*

Remedial intervention: A conservation activity undertaken after a change in a material property (e.g. such as that caused by the influence of carbonation or chlorides) has adversely affected the ability of the structure, or parts thereof, to meet the required performance levels because of deterioration.

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The term *'remodelling'* is often employed where changes principally involve appearance, rather than alteration of the structural components.[1]

In some instances, the *repair* may not be intended to bring the structure or its components so treated back to its original level of serviceability or durability. The work may sometimes be intended simply to reduce the rate of deterioration or degradation, without significantly enhancing the current level of performance.[2]

Replacement may include improvements and strengthening, but does not usually involve a change in function.[1]

The *required service life* is the basis for determining the *specified (design) service life* (for new structures) and the *specified (design) residual service life* (for existing structures).

The *requirements development phase* may be subdivided into gathering the requirements from *stakeholders*, checking for consistency and completeness, definition (writing down descriptive requirements), and specification (creating an initial bridge between requirements and design). The *requirements development phase* may have been preceded by a *feasibility study* of the project.

While *stakeholders* usually believe that they know which performance of a structure they request, it may require skill and experience in structural engineering to recognize incomplete, ambiguous or contradictory requirements.

Remodelling: Changes or alterations to a structure to meet revised functions, performance requirements, usage or occupancy.[1]

Removal: Removing parts from a structure.[1]

Renewal: To reinstate the performance of a damaged or deteriorated component or structure using original methods and materials.[1]

Repair: Intervention taken to reinstate to an acceptable level the current and future performance of a structure or its components which are either defective, deteriorated, degraded or damaged in some way so their performance level is below that anticipated by the designer; generally without restriction upon the materials or methods employed.

Representative value of an action: The value of an action used for the verification of a limit state. A representative value may be the characteristic value, the combination value, the frequent value and the quasi-permanent value, but it may also be an other value of an action.[4, 9]

Replacement: Action to provide substitute new components for ones which have experienced deterioration, damage, degradation or mechanical wear.[1]

Required service life: The demand stated by the stakeholders (i.e. owners, users, contractors, society) for the period in which the required performance shall be achieved.

Requirements development phase: Phase of extracting and describing performance requirements for a structure.

Resistance: capacity of a member or component, or a cross-section of a member or component of a structure, to withstand actions.[5]

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The *residual service life* is related to the *required service life*, as given by the *stakeholders* (i.e. owners, users, contractors, society) of the structure and to the other implications of service criteria agreement, e.g. with regard to structural analysis, maintenance and quality management.

Robustness is an indication of the ability of a structural system to mobilise alternative load paths around an area of local damage. It is related to the strength and form of the structural system, particularly the degree of redundancy (number of potential alternative load paths) within the structural system.[1]

In the context of performance-based *design of structures*, *safety* is one of the basic performance requirements. For comparison, see the definition of *structural safety*.

For comparison see the definition of *Required service life*, *Specified (design) service life*, *Residual service life.*

CEN documents are using the term *working life* where this Model Code is applying the term *service life.*

Serviceability may be evaluated under various headings and consideration would normally be given to a number of issues affecting either the whole structure, or parts thereof. The issues would typically include various limit state cases (e.g. deflection, vibration, thermal movements, appearance, etc.).[1] In the context of performance-based *design of structures*, *serviceability* is one of the basic performance requirements.

Residual service life: The demand for the remaining period in which the required performance shall be achieved, used in the re-design of existing structures.

Restoration: Intervention to bring the structure or its component parts back to their original condition not only with regard to function and performance level anticipated by the designer, but also with regard to aesthetic appearance and possibly other (historical) considerations.[1]

Risk: The combination of the likelihood of occurrence of a particular hazard and its consequences.[1]

Robustness: The ability of a structure subject to accidental or exceptional loadings to sustain local damage to some structural components without experiencing a disproportionate degree of overall distress or collapse.[1]

Safety: Ability of a structure or structural element to ensure that no harm would come to the users and the people in the vicinity of the structure under any (combination of) expected actions.[8]

Safety criterion: performance criterion for the ultimate limit state (ULS).

Service life: The period in which the required performance of a structure or structural element is achieved, when it is used for its intended purpose and under the expected conditions of use. [1, 5]

Serviceability: Ability of a structure or structural element to perform adequately for normal use under all (combinations of) actions expected during service life.[9]

Serviceability limit: Specified limit of serviceability.[6]

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Serviceability limit state (*SLS*): State that corresponds to conditions beyond which specified service requirements for a structure or structural member are no longer met.[4]

Serviceability criterion: Performance criterion for a serviceability limit state (SLS).[4]

Service criteria: Requirements for the behaviour of a structure resulting from the planned use.[6]

Service criteria agreement: Description of the utilisation and protection aims of the stakeholders (i.e. owners, users, contractors, society) as well as the basic conditions and regulations for the design, execution and use of the structure.[6]

Service instructions: Instructions for the owner and the operator on the use of the construction works.[6]

Service situations: Physical circumstances and conditions during the design service life.[6]

Specified (design) service life: The demand for the period in which the required performance shall be achieved, used in the design of new structures.

The s*pecified (design) service life* is related to the *required service life*, as given by the *stakeholders* (i.e. owners, users, contractors, society) and to the other implications of service criteria agreement e.g. with regard to structural analysis, maintenance and quality management.

As a rule, the key *stakeholders* would be the founders, the owners, the residents, the users, the neighbours (if construction creates a nuisance for them), the contractor, the design and constructing team, the tenancy management team and the maintenance team. Other *stakeholders* may be the government and society.

Stakeholder: Person or organization that has a legitimate participation in a project.

Strengthening: An intervention made to increase the strength (load resistance / load capacity) and / or possibly the stiffness of a structure or its components, and / or to improve overall structural stability and / or the overall robustness of the structure to a performance level above that anticipated by the designer.

Structural integrity: The ability of structural components to act together as a competent single entity.[1]

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Construction materials used primarily for decoration, insulation, or other than structural purposes are not included in group of *structural materials*.[10]

Structural safety is usually related to the ratio between the actions that would cause collapse or other similar forms of structural failure and the actions that are imposed upon it in service.[10]

Survey is taken to mean the range of activities used to evaluate conformity with the design data for actions and / or material and / or product properties used in the service life design (SLD) on a periodic basis during the service life of the structure. Survey activities would be expected to include a visual inspection undertaken in conjunction with various forms of localised condition testing and measurement (e.g. measurement of depth of cover to reinforcement).

The term *'survey'* may be applied to the inspection of a number of similar structures / components to obtain an overview. The term *'survey'* is also used to describe the formal record of inspections, measurements and other related

Structural analysis: Determination of action effects by means of a structural model, if necessary in steps, using different analytical models for the structures as a whole, individual members and local effects.[6]

Structural design concept: The basic idea underlying the structural design.[6]

Structural materials: Construction materials which, because of their ability to withstand actions, are considered in the design of a structure.[10]

Structural member: Physically distinguishable part of a structure.[6]

Structural model: Result of delimiting and idealising the structural system.[6]

Structural safety: Ability of a structure and its members to guarantee the overall stability as well as an adequate ultimate bearing resistance, corresponding to the assumed actions and the required reliability for the specified reference period.[6]

Structural system: Arrangement of interacting structural members offering a potential solution to provide bearing resistance to a specified combination of actions.

Structure: Product of human design, intended to fulfil societal functions with adequate reliability with regard to safety, serviceability and sustainability, for a defined period of time.

Substrate: The surface layer in which a protection or repair material is applied or is to be applied.[1]

Survey: The process, often involving visual examination or utilising various forms of sampling and testing, aiming at collecting information about the shape and current condition of a structure or its components.[1]

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information which describes the form and current condition of a structure and its components.[1]

Various types of *testing* are recognised, their classification being primarily based on the amount of damage or interference caused to the structure. The main divisions are:

- non-destructive testing (NDT), which does not cause damage to the structure by the test procedure (e.g. testing with cover meter, radar, acoustic emission, load testing in the elastic range, etc.),
- destructive testing, which may cause damage to the structure or marking of the surface finishes (e.g. pull-out tests, material sampling, load testing beyond the elastic range, etc.).[1]

Generally, the *ultimate limit state* (*ULS*) corresponds to the maximum load-carrying resistance of a structure or structural member.[4]

Up-grading (*retrofitting*) relates particularly to the strengthening of structures as a means of minimising damage during specified loading events.

Sustainability: Ability of a structure or structural element to contribute positively to the fulfilment of the present needs of humankind with respect to nature, society, economy and wellbeing, without compromising the ability of future generations to meet their needs in a similar manner.

Technical report: Explanatory report on design work.[6]

Tender documents: Text of the planned contract for materials and work, special conditions, bill of quantities or work description, plans and general conditions.[6]

Testing: Procedure aiming at obtaining information about the current condition or performance of a structure or its components.[1]

Tie: tensile continuous elements acting across the structure, horizontally and/or vertically.

Transient design situation: Design situation which is decisive during a much shorter period than the design service life.[6]

Ultimate limit state (*ULS*): State associated with collapse or with other similar forms of structural failure.[4]

Ultimate resistance: Limit of resistance.[6]

Up-grading (*retrofitting*): Intervention to enhance the functionality or form of a structure or its components so as to improve some aspect of future performance above that defined / achieved during design and construction; typically undertaken to achieve an improved (higher) load resistance against specified loads / actions.

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Use: Utilisation of a structure as described in the service criteria agreement and in the basis of design.[6]

Variable action: Action which is not permanently acting, not constant or not monotonically changing during a reference period.[6]

Verification: Confirmation of the fulfilment of a performance requirement.

2.2 References

The definitions given in section 2.1 are based on the following sources:

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3 Basic principles

3.1 General

The performance of a structure or a structural component refers to its behaviour as a consequence of actions to which it is subjected or which it generates.

Structures and structural members shall be designed, constructed and maintained in such a way that they adequately and in an economically reasonable way perform during construction, service life and dismantlement.

In general:

- structures and structural members shall remain fit for the use for which they have been designed,
- structures and structural members shall withstand extreme and/or frequently repeated actions and environmental influences liable to occur during their construction and anticipated use and shall not be damaged by accidental and/or exceptional events, to an extent disproportionate to the triggering event,
- structures and structural members shall be able to contribute positively to the needs of humankind with regard to nature, society, economy and well-being.

Accordingly, three categories of performance have to be addressed:

- serviceability, i.e. ability of a structure or structural members to perform, with appropriate levels of reliability, adequately for normal use under all (combinations of) actions expected during service life,
- Structural, safety i.e. ability of a structure and its structural members to guarantee the overall stability, adequate deformability and ultimate bearing resistance, corresponding to the assumed actions (both extreme and/or frequently repeated actions and accidental and/or exceptional events) with appropriate levels of reliability for the specified reference periods. The structural safety shall be analyzed for all possible damage states and exposure events relevant for the design situation under consideration.

Robustness is a specific aspect of structural safety that refers to the ability of a system subject to accidental or exceptional loadings (such as fire, explosions, impact or consequences of human errors) to sustain local damage to some structural components without experiencing a disproportionate degree of overall distress or collapse.

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In ISO 15392 (Sustainability in Building Construction – General Principles) sustainability is defined as state in which components of the ecosystem and their functions are maintained for the present and future generations.

Performance requirements shall be satisfied in a well-balanced manner throughout the life cycle of the structure.

In the context of Limit State Design, the term "failure" means failing to fulfil the criteria of the limit state under consideration.

– Sustainability, i.e. ability of a material, structure or structural members to contribute positively to the fulfilment of the present needs of humankind with respect to nature, society and humans, without compromising the ability of future generations to meet their needs in a similar manner.

3.2 Performance-based design and assessment

3.2.1 General approach

Using a performance-based approach, a structure or a structural component is designed to perform in a required manner during their entire life cycle. In case of existing structures, by a performance-based approach it is assessed whether the actual performance of an existing structure or a structural members and their performance during the residual life satisfy the demands of the stakeholders.

Performance is evaluated by verifying the behaviour of a structure or a structural component against the specified performance requirements.

An adequate performance is reached when a structure or a structural component has demonstrated satisfactory behaviour to meet the performance requirements. In the opposite case, the performance of a structure or a structural component is considered to be inadequate.

In this document, the state where the performance of a structure or a structural component is inadequate is referred to as failure.

The performance-based design of a new structure or a structural component is completed when it has been shown that the performance requirements are satisfied for all relevant aspects of performance related to serviceability, structural safety and sustainability.

The performance-based assessment of an existing structure or a structural component is completed when it has been identified whether all relevant performance requirements are satisfied or not. In the latter case the performance of a structure or a structural component is qualified as inadequate (failure).

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As a rule, the key stakeholders would be the founders, the owners, the residents, the users, the neighbours (if construction creates a nuisance for them), the contractor, the design and construction team, the tenancy management and maintenance team. Other stakeholders may be the government and the society.

While stakeholders usually believe they know what performance they want for a structure, it may require skill and experience in structural engineering to recognize incomplete, ambiguous or contradictory demands.

Specifying performance requirements and associated constraints of service life and reliability creates an initial bridge between the needs of the stakeholders and the design or the assessment. Sets of specified performance requirements are used as input into the performance-based design or assessment of structures.

Refinement of the specification of performance requirements depends on the complexity of the project under consideration.

The specified (design) service life and residual service life shall be defined according to the required service life and of the other implications of service criteria agreement e.g. with regard to structural analysis, maintenance and quality management.

3.2.2 Basis for verification

The stakeholders shall give demands for performance of a structure or a structural component and its required service life.

Those demands reflect the role(s) that a structure or a structural element should play under the intended conditions of construction, service and dismantlement.

For each aspect of performance that is relevant for a structure or a structural component under consideration, the performance requirements must be specified. Demands of the stakeholders are the basis for specifying the performance requirements. Accordingly, the performance requirements refer to the fulfilment of the essential demands of the stakeholders.

Performance requirements are established by means of the performance criteria and the associated constraints related to service life and reliability. The performance requirements are satisfied if all relevant performance criteria are met during the service life at the required reliability level.

Performance criteria are quantitative limits defining the border between the desired and the adverse behaviour, relevant for the specific aspect of performance.

Constraints related to service life are given by means of a specified (design) service life (relevant for the design of new structures) or a residual service life (relevant for the re-design of existing structures). The specified (design) service life and the residual service life refer to the period in which the required performance shall be achieved for structures to be designed and for existing structures, respectively.

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The target reliability level shall be adopted to suit the use of the structure, in dependence of the type of structure or a structural component and the situation considered in design.

An example of the set of performance requirements, specified on the basis of performance criteria and associated constraints for different performance categories is given in the table below. For further information, see for example EN 1990, Annexes B and C.

Table 3.2-1: Example of performance requirements for design of a new structure

Performance category	**Performance criteria**	**Constraints**
Serviceability	Deformation limit Crack width limit Vibration limit, etc.	Specified (design) service life: 50 year Target reliability level: $\beta = 1.5$
Structural safety	Stress limit Capacity limit Progressive collapse limit, etc.	Specified (design) service life: 50 year Target reliability level: $\beta = 3.8$
Sustainability	Emission limits Impact on society Aesthetics, etc.	

Constraints related to reliability are specified by means of a target reliability level. A target reliability level refers to an acceptable failure probability corresponding to a specified reference period, which is required to assure the performance of a structure or structural component for which it has been designed. The target reliability level for structures to be designed and for existing structures may adequately be expressed in terms of the target reliability index β_t or target probability of failure P_{ft}.

The particular choice of performance requirements used in the design depends on the situation that is being modelled.

Considerations regarding the choice of the performance criteria and the associated constraints are found in subclause 3.3.1 (performance requirements with regard to serviceability and structural safety), in subclause 3.3.2 (service life), in subclause 3.3.3 (reliability) and in subclause 3.3.4 (performance requirements with regard to sustainability).

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The limit states either refer to the entire structure, to structural members or to local regions of the members.

In practical design, most of the limit states refer to simplified models for describing the exposure and the structural response. However, limit states may also be introduced which are not directly related to any losses/damages but which are introduced, for example in order to account for several actual limit states simultaneously.

In very particular cases a limit between the serviceability limit states and the ultimate limit states may be defined, a so-called “partial damage limit state” (e.g. in case of earthquake damage of plant structures “partial damage

3.3 Performance requirements for serviceability and structural safety

In this Model Code, the concept of Limit State Design is applied to carry out performance-based design (or re-design) for serviceability and safety.

In the context of the performance-based Limit State Design for safety and serviceability, the structural performance of a whole structure or part of it shall be described with reference to a specified set of limit states, which separate desired states of the structure from adverse states.

Limit states are states beyond which the performance requirements are no longer satisfied.

Conceptually, limit states correspond to a discrete representation of the structural response under specified exposure to which specific losses/ damages can be associated.

Limit states shall be related to design situations. They may relate to persistent situations during the service life of the works, transient situations during the execution of the construction works (stage of construction and/or assembling or repair), extreme actions and environmental influences, unintended use or accidents.

Design principles regarding the performance-based Limit State Design for safety and serviceability are given in chapter 7.

3.3.1 Performance criteria for serviceability and structural safety

In the context of performance-based Limit State Design, performance criteria for serviceability and structural safety are specified by :

- serviceability limit states criteria (see subclause 3.3.1.1),
- ultimate limit states criteria (see subclause 3.3.1.2),
- robustness criteria (see subclause 3.3.1.2).

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limit state" is associated to the safe shutdown of the plant). For more details reference is made to the section 3.11 of CEB Bulletin 191 and to the JCSS Probabilistic Model Code.

In the cases of irreversible local damage or irreversible unacceptable deformations, the exceedance of a serviceability limit state is irreversible and the first time that this occurs causes inadequate serviceability of the structure, i.e. failure. Some repair may be necessary for the structure to be fit-for-use.

In other cases (like temporary local damage by for instance wide cracks, temporary large deformations or vibrations) the exceedance of a serviceability limit state may be reversible and then failure occurs:

- the first time that the serviceability limit state is exceeded, if no exceedance is considered as acceptable;
- if exceedance is acceptable but the time during which the structure is in the undesired state is longer than specified;
- if exceedance is acceptable but the number of times that the serviceability limit state is exceeded is larger than specified, if a combination of the criteria mentioned previously occurs.

Frequently exceeding the serviceability limit states may affect the efficient use of a structure, its components (tanks, pipes, canals) or its appearance. In many cases, the risk of damage is indirectly excluded by ultimate limit state verifications or by detailing.

Generally, operational limit state is reached when:

- the facility has suffered practically no structural or non-structural damage and can continue serving its original intention with little disruption of use for repairs, supported either by undamaged lifelines or by back-up systems and
- any repair that is necessary can be deferred to the future without disruption of normal use.

3.3.1.1 Serviceability limit states

Serviceability limit states correspond to the states beyond which specified demands for a structure or a structural component related to its normal use or function are no longer met.

The serviceability limit states address fitness-for-use of a structure.

Accordingly, the serviceability limit states that should be considered can be described as:

- operational limit states,

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Generally, immediate use limit state is reached when:

- the structure itself is very lightly damaged (i.e. localised yielding of reinforcement, cracking or local spalling of concrete, without residual drifts or other permanent structural deformations),
- the normal use of the facility is temporarily but safely interrupted (in the case of an industrial plant, after a safe shutdown) and can be restored as soon as utility systems are back in operation;
- risk to life is negligible,
- the structure retains fully its earlier strength and stiffness and its ability to withstand loading,
- the (minor) damage of non-structural components and systems can be easily and economically repaired at a later stage.

The serviceability limit state criteria may refer to, for example:

- unacceptable deformations or deflections which impair the functionality of the structures or their in-fills, cause damage to non-structural components, cause discomfort to people, affect the appearance of structural or non-structural components or the functioning of equipment (the conditions to be fulfilled with regard to limiting the deformation are associated with the type of building or the civil engineering structure and are often, for the sake of simplification, substituted by rough approximations);
- excessive vibrations which limit functional effectiveness of the structures, affect non-structural components, impair the user's comfort or the functioning of equipment (although such limit states may be characterized by the magnitude of the vibrations, they are commonly indirectly covered by limiting the fundamental period of vibrations of the structure or some of its structural components, in comparison to the expected period of the vibrations);
- local damage (e.g. cracking, slip in connections) which does not affect structural safety but may affect the efficiency or appearance of structural or non-structural components;
- immediate use limit states.

The corresponding serviceability limit state criteria are related to:

- functionality of the structure related to its normal use,
- comfort of using the structure.

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– local or global degradation due to environmental actions (e.g. depassivation of reinforcement, weathering) which may affect the efficiency or appearance of structural or non-structural components;
– lack of tightness or defective sealing that restrict the functionality or impair the user comfort.

The limit values that define the serviceability limit state criteria differ, depending on whether it concerns operational limit or immediate use limit. Design principles regarding the formulation of performance criteria for the analysis of the serviceability limit states are given in chapter 4.

The procedures for the verification of the serviceability limit states are given in clause 7.6 (for RC and PC structures) and in clause 7.7 (for FRC structures).

3.3.1.2 Ultimate limit states

Ultimate limit states are limit states associated with the various modes of structural collapse or stages close to structural collapse, which for practical purposes are also considered as ultimate limit states.

The ultimate limit states address:
– life safety,
– protection of the structure and environment,
– protection of operations.

Accordingly, the serviceability limit states that should be considered can be described as:
– life-safety limit states,

The effect of exceeding an ultimate limit state is almost always irreversible and causes failure the first time it occurs.

Generally, life-safety limit state is reached when:
– the structure does not collapse, even partly, retaining its integrity and residual load capacity although it is significantly damaged,
– the structure may provide sufficient safety for temporary but not normal use; however it retains its full load capacity and sufficient residual strength and stiffness to protect life for the period until repair,
– secondary or non-structural components may be seriously damaged, but do not obstruct emergency use or cause life-threatening injuries by falling,

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– reparability is economically questionable and demolition may be preferable.

Generally, near-collapse limit state is reached when:

– there is substantial life safety during the loading (but not full, as there may be life-threatening injury due to falling hazards),
– the structure suffers heavy damage and is at the verge of collapse, with low residual strength and stiffness but with sufficient elements still supporting the quasi-permanent loads,
– most non-structural components (e.g. partition walls in buildings) collapse,
– the structure is unsafe even for emergency use and would probably not survive loading,
– it may not be technically feasible or economically sensible to reinstate the structure to its pre-earthquake condition.

The ultimate limit states which may require consideration include:

– attainment of the maximum resistance of structures, structural members and sections (regions) e.g.:
  – attainment of the maximum resistance by material failure, excessive deformations or settlement;
  – attainment of the maximum resistance resulting from the loss of resistance capacity caused by fire;
  – attainment of the maximum resistance of resulting from the loss of resistance caused by degradation due to environmental actions (e.g. corrosion of reinforcement, corrosion induced cracking and spalling, ASR);
  – attainment of the maximum resistance caused by impact or explosion;
  – reduction of residual resistance below a certain limit due to an earthquake;
  – permanent deformations exceeding a certain limit after an earthquake.
– near-collapse limit states.

The corresponding ultimate limit states criteria are related to:

– resistance of critical regions,
– fatigue,
– stability.

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– rupture of structural members caused by fatigue under essentially repetitive loading or other time-dependent effects;
– loss of stability of the structure or any part of it, including supports and foundations, for example:
  – sudden change of the assumed structural system to a new system (e.g. transformation into a kinematic mechanism or snap through);
  – buckling of slender structures or structural members, in which second order effects are important;
  – loss of equilibrium of the structure or of a part of the structure, considered as a rigid body (e.g. overturning);
  – loss of equilibrium caused by impact or explosion;
  – sliding beyond a certain limit or overturning due to an earthquake.

Through its robustness, the structural system must continue to provide the function for which it was created, modified or preserved, without being damaged to an extent disproportionate to the original cause.

The limit values that define the serviceability limit state criteria vary, depending on whether it concerns life-safety limit or near-collapse limit.

Design principles regarding the formulation of performance criteria for ultimate limit states analysis are given in chapter 4.

The procedures for verification of the ultimate limit states are given in clause 7.3 (for predominantly static loading of RC and PC structures), clause 7.4 (for non-static loading of RC and PC structures,) and in clause 7.6 (for FRC structures).

3.3.1.3 Robustness

Robustness is related to survival of function of the structural system during events such as accidental loading or consequences of human errors.

Robustness of the structural system addresses:

– life safety,
– property and environment protection,
– protection of operations.

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The limit states which may require consideration are related to:

- disproportionate failure of a large part of the structure or the whole structure caused by an accidental load or failure of a structural component (e.g. loads, explosion, loads by extremely high soil water, loads due to extreme loading such as fire, impact, explosion or earthquake), such as:
  - system collapse,
  - life-threatening component collapse.

Some specific aspects of verification of robustness in case of extreme loading are addressed in clause 7.4.

Accordingly, the robustness criteria are related to:

- resistance of the structural system,
- special functions (e.g. shelter from climatic phenomena, containment of substances, providing fortification, security, shade, etc.).

The general principles and the procedures for the verification of robustness are given in clause 7.9.

3.3.2 Service life

3.3.2.1 Specified service life and residual service life

For structures to be designed the specified (design) service life defines the period in which the design has to show that the performance of the structure is maintained at a level compatible with the fulfilment of the performance requirements.

For the main dimensioning and for reliability verifications, the service life is for practical purposes expressed in terms of a reference period t_R. The specified (design) service life is however not synonymous to the reference period. Misinterpretations may lead to errors, see the explanations in subclause 3.3.3.1 (Table 3.3-3 to 3.3-6).

The residual service life of an existing structure may be shorter than the specified (design) service life intended for a structure in a structural design. In such a case it may be necessary to upgrade the structure.

In the assessment of existing structures, the demand for the remaining period in which the required performance shall be achieved is referred to as the residual service life.

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Some examples of the specified (design) service life for different types of structures are given in the table below.

Table 3.3-1: Example of specified (design) service life for the design of a new structure, according to ISO 2394

Type of structure	**Specified (design) service life**
Temporary structure	1 to 5 years
Replaceable components of structures e.g. gantry girders, bearings	25 years
Buildings and other common structures of average importance	50 years
Structures of greater importance e.g. monumental buildings, large bridges, other special or important structures	100 years or more

A differentiation between replaceable and non-replaceable components of the structure may be considered when choosing the specified (design) service life for the structure and its components.

Limit states associated to the time-dependent material degradation are for example initiation of reinforcement corrosion, cover concrete cracking and spalling due to corrosion, etc.

The specified (design) service life and the residual service life are related to the required service life as given by the stakeholders and to other implications of service criteria agreement e.g. with regard to structural analysis, maintenance and quality management. The required service life should be given by the owner in consideration of the interests of other stakeholders (i.e. users, contractors, society).

If the performance requirements are satisfied during the specified (design) service life (in case of structures to be designed) or during the residual service life (in case of existing structures), a structure is considered to be sufficiently durable.

The nominal/formal end of the service life is reached when the performance criteria are not anymore met at the required reliability level.

3.3.2.2 Verification of service life

The performance verification shall be conducted with proper consideration of the change of performance over time (e.g. due to degradation or time effects). Currently, this proper consideration of the chronological change of performance is not fully possible.

Therefore, with regard to the verification of performance requirements for safety and serviceability a staggered approach is taken. Verification of limit states associated with safety and serviceability is performed without

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Due consideration is needed to decide if limit states related to a change of performance due to material degradation shall be regarded as serviceability limit states (which may be a failure to achieve some performance, such as good appearance of the structure) or as safety limit states (which may be a failure such as falling of spalling concrete, which may diminish the resistance or may be harmful to people around the structure).

Further considerations for the choice of the level of reliability are found in chapter 7 of the JCCS Probabilistic Model Code.

The costs involved when upgrading the performance of existing structures (e.g. increasing their safety) are usually high compared to the costs of improving the same performance by a structural design in case of a new structure. Upgrading existing structures may go along with relocation of occupants and disruption of activities or influence heritage values, which does not play a role in case of the design of new structures. Finally, considering a change of performance over time due to degradation. In parallel, verification of limit states associated to the time-dependent material degradation is performed by means of service life verification.

Accordingly, the service life verification is performed as a justification of the assumption of time-independence of the structural performance, which is made when verifying safety and serviceability according to the procedures described in the clauses 7.3 (verification of structural safety for predominantly static loading of RC and PC structures), 7.4 (verification of structural safety for non-static loading of RC and PC structures), 7.6 (verification of the serviceability for RC and PC structures) and in clause 7.7 (verification of safety and serviceability for FRC structures).

Service life verification demonstrates that during the specified (design) service life (new structures) or the residual service life (existing structures) the performance will not be deteriorated due to degradation.

Design principles and the procedures for service life design are given respectively in chapter 4 and clause 7.8.

3.3.3 Reliability

3.3.3.1 Target reliability level

The choice of the target level of reliability should take into account the possible consequences of failure in terms of risk to life or injury, the potential economic losses and the degree of societal inconvenience. The choice of the target level of reliability also takes into account the amount of expense and effort required to reduce the risk of failure.

Because of large differences in the outcome of such considerations, due attention should be given to differentiating the reliability level of structures to be designed and of existing structures.

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sustainability requirements (e.g. recycling and re-use, reduction of waste) can be usually better satisfied in the case of the design of new structures.

The relationship between P_f and β-values is given in Table 3.3-2.

Table 3.3-2: β-values related to the failure probability P_f, according to EN 1990:2002.

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-6}
β	1.28	2.32	3.09	3.72	4.75

Reliability management shall be supported by suitable databases of different types of structures, their performance over time taking into account various degradation processes. Therefore, data have to be collected in order to quantify risk and, hence, decide on the target reliability values.

The principles of probabilistic structural limit state design with a possibility for differentiating the reliability level are described in the JCSS Probabilistic Model Code.

It is noted that (design) service life and target β value are two independent requirements on structural performance. For example, the same β value may be required for structures with different (design) service lives and vice versa (see ISO 2394).

However, the target reliability sometimes is presented not for the (design) service life but as an equivalent value for different (e.g. one year) reference period t_R. In Table 3.3.3 the EN 1990 values are given for a 50 years reference period, which is supposed to be the standard (design) service life.

Reliability requirements for structures to be designed and for existing structures may adequately be expressed in terms of the reliability index β:

$$\beta = -\Phi^{-1}(P_f) \qquad (3.3\text{-}1)$$

where

$\Phi(\ )$ is the standard normal probability distribution function

P_f is the failure probability corresponding to a specified reference period.

In order to make the right choice for the target β values, the reference period, the consequences of failure and the cost of safety measures shall be analysed for the specific case considered.

The maximum acceptable failure probability depends on the type of the limit state and considered consequences of failure for relevant construction work.

A differentiation of the reliability level for different consequences of failure and the cost of safety measures may be done on the basis of well-founded analysis. If such analysis is omitted, in this Model Code it is recommended to apply target reliability indices for structures to be designed, as given in Table 3.3-5.

Normally, the specified (design) service life is considered as the reference period for a structure to be designed for serviceability and fatigue, while the residual service life determined at the assessment is often considered as the reference period for an existing structure.

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These target β-values are equivalent to the values in Table 3.3-4, which are given for a reference period t_R of 1 year. Note that in both Table 3.3.-3 and 3.3.-4 the (design) service life is equal to 50 years. Similar arguments hold for Tables 3.3-5 and 3.3-6.

Table 3.3-3: Target β-values related to a reference period of 50 years (examples), according to EN 1990.

Relative costs of safety measures	**Consequences of failure** small	some	moderate	great
High	0	1.5	2.3	3.1
Moderate	1.3	2.3	3.1	3.8
Low	2.3	3.1	3.8	4.3

Table 3.3-4: Target β-values related to a reference period of 1 year (examples), according to EN 1990.

Relative costs of safety measures	**Consequences of failure** small	some	moderate	great
High	2.3	3.0	3.5	4.1
Moderate	2.9	3.5	4.1	4.7
Low	3.5	4.1	4.7	5.1

The target reliability indices given in Table 3.3-5 for serviceability limit states verification correspond approximately to the values recommended in ISO 2394 for *small consequences of failure* and *moderate relative costs of safety measures*. The target reliability indices given in Table 3.3-5 for ultimate limit states verification correspond to those recommended in ISO 2394 for, respectively, *some*, *moderate* and *great consequences of failure* and *low relative costs of safety measures*.

Table 3.3-5: Recommended target reliability indices β for structures to be designed, related to the specified reference periods.

Limit states	**Target reliability index β**	**Reference period**
Serviceability		
reversible	0.0	Service Life
irreversible	1.5	50 years
irreversible	3.0	1 year
Ultimate		
low consequence of failure	3.1	50 years
	4.1	1 year
medium consequence of failure	3.8	50 years
	4.7	1 year
high consequence of failure	4.3	50 years
	5.1	1 year

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The target reliability level for the existing structures may be chosen lower than for new structures, because for existing structures the costs of achieving a higher reliability level are usually high compared to structures under design.

For more details reference is made to ISO 13822 "Bases for design of structures – Assessment of existing structures" and ISO 2394 "General principles on reliability for structures"

For more details, reference is made to the JCSS Probabilistic Model Code.

The β values given in Table 3.3-5 may also be used for assessment of existing structures, however differentiation of the target reliability level for the new structures and for the existing structures may need to be considered.

Decision for choosing different target reliability level for existing structures may be taken only on the basis of well founded analysis of consequences of failure and the cost of safety measures for specific case. Some suggestions for the reliability index for existing structures are given in Table 3.3-6 for the specified reference periods.

Table 3.3-6: Suggested range of target reliability indices β for existing structures, related to the specified reference periods.

Limit states	**Target reliability index β**	**Reference period**
Serviceability	1.5	Residual Service Life
Ultimate	in the range of 3.1 - 3.8*	50 years
	in the range of 3.4 - 4.1*	15 years
	in the range of 4.1 - 4.7*	1 year

* depending on costs of safety measures for upgrading the existing structure

The requirements for the reliability of the components of the system shall depend upon the system characteristics. The target reliability indices given in Table 3.3-5 and Table 3.3-6 relate to the structural system or in approximation to the dominant failure mode or structural component dominating system failure. Therefore, structures with multiple, equally important failure modes should be designed for a higher level of reliability per component than recommended in this Model Code.

The target reliability indices given in Table 3.3-5 and Table 3.3-6 are valid for ductile structural components for which a collapse is preceded by some kind of warning, which allows measures to be taken to avoid severe consequences. Therefore by explicit requirements or by appropriate detailing it shall be assured that brittle failure does not occur. A structural component

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which would be likely to collapse suddenly without warning should be designed for a higher level of reliability than recommended in this Model Code for ductile structural components.

To satisfy performance requirements at the target reliability levels as recommended in Table 3.3-5 and Table 3.3-6, one normally proceeds from the safety concepts, explained in chapter 4.

In this Model Code the partial factor method is calibrated in such a way that when applying the values of partial factors given in clause 4.5 the following reliability requirements are satisfied with a sufficient margin during a defined period of 50 years:

$\beta = 1.5$ in case of serviceability limit states verification,

$\beta = 3.1$ in case of fatigue verification,

$\beta = 3.8$ in case of ultimate limit states verification,

For other β values (e.g. applied in assessment of existing structures), the partial factor format, explained in clause 4.5 can also be applied. However, reconsideration of the partial factors and characteristic values of the fundamental basic variables as given in subclauses 4.5.2 and 4.5.3 may be required, following from the consideration of actual uncertainties regarding actions, resistances, geometry, structural modelling and the determination of action effects. This is further discussed in subclauses 4.5.2.3 and 4.5.3.2.

The target reliability index $\beta = 1.5$ corresponds to the value given in Table 3.3-5 for ultimate limit state verification in case of irreversible failure and reference period of 50 years.

The target reliability index $\beta = 3.1$ corresponds to the value given in Table 3.3-5 for ultimate limit state verification in case of low consequence of failure and reference period of 50 years. Depending on particular consequences of fatigue failure and possibility for inspection and repair in case considered, higher or lower values for β in case of fatigue verification may be appropriate.

The target reliability index $\beta = 3.8$ corresponds to the value given in Table 3.3-5 for ultimate limit state verification in case of medium consequence of failure and reference period of 50 years.

It is noted that Eurocode EN 1990, Annex B gives also partial factors to loads corresponding to β-values for other consequences classes.

The fully probabilistic design method as described in clause 4.4 may be used for any β value.

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Component reliability is the reliability of one single structural component which has one dominating failure mode.

System reliability is the reliability of a structural system composed of a number of components or the reliability of a single component which has several failure modes of nearly equal importance.

A probabilistic approach provides a better platform from which system behaviour can be explored and utilised. For more details reference is made to the JCSS Probabilistic Model Code.

The true nature of global environmental problems is a result of socio-economic systems that came about following the explosion of industrialization due to the Industrial Revolution, in which mass production, mass consumption and mass disposal have flourished. Such systems have caused

3.3.3.2 Component reliability and system reliability

Structural analysis methods, as described in this Model Code, are primarily concerned with component behaviour with respect to one dominant failure mode. Each limit state equation is, in most cases, related to a single mode of failure of a single component.

However, individual components may also be susceptible to a number of possible failure modes. Therefore, in design the susceptibility of the individual components to a number of possible failure modes shall be checked where relevant, by checking of a number of limit state equations.

Besides, most structures are an assembly of structural components. System behaviour is usually the most serious consequence of component failure. Therefore, likelihood of system failure following an initial component failure should be assessed in relation to robustness with respect to accidental events, redundancy (alternative load paths), and complexity of the structure (multiple failure modes). Accordingly, system analysis shall be carried out as a part of the design.

In particular, it is necessary to determine the system characteristics in relation to robustness with respect to accidental and/or exceptional events (see subclause 7.4.12).

The system analysis requires considerable innovation and initiative from the engineer. In general, the system behaviour of structures can be quantified in terms of limit state design by deterministic approach (e.g. progressive collapse analysis) or by a probabilistic approach.

3.4 Performance requirements for sustainability

3.4.1 General

The purpose of design for sustainability is to reduce impacts on the environment, society, and the economy by evaluating and verifying the performance of concrete, concrete components, or structures.

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the destruction of ecological system due to the use of land, natural resource and energy depletion, as well as water pollution, the emission and diffusion of hazardous substances and greenhouse gases, waste excretions, etc. Mankind has realized that these impacts exceed the allowable limit.

As a fundamental scheme in social economic activities, therefore, a paradigm shift to sustainable development has become significant. The concept of sustainable development was proposed in Brundtland Report in 1987. Sustainable development was defined as "development which meets the needs of the present without compromising the ability of future generations to meet their own needs." The report described three fundamental aspects: environmental protection, economic growth and social equality. After the publication of this report, a keyword "Sustainable Development" became firmly established as the final target worldwide.

In general, a concrete structure shall be designed so that it can satisfy the performance requirements regarding serviceability, safety and sustainability in a well-balanced manner throughout its design service life.

Economic aspects should be satisfied during the first stage as the most fundamental requirement or it may change depending on the other factors.

Fulfilment of sustainability requirements for a structure presumes that all aspects of design, construction, use, conservation, demolition as well as recycling and disposal that are relevant for the environment and society are taken into account.

The economic aspects of sustainability are not dealt with as a performance requirement in this Model Code.

Accordingly, the performance requirements for sustainability are related to:

- impact on the environment, which is defined as the influence of the activities, from the design to disposal, on the environment,
- impact on society, which is defined as the influence of the activities from the design to disposal, on society,
- aesthetics.

Performance requirements, which are necessary for the verification of sustainability, are determined by a decision maker on the basis of legislative regulations, particular intents of stakeholders (e.g. specifiers or owners), or international agreements, etc.

Performance requirements related to sustainability are formulated in subclause 3.4.2 (impact on environment), subclause 3.4.3 (impact on society) and subclause 3.4.4 (aesthetics).

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Rational evaluation of the sustainability of a structure can be realised by means of life cycle assessment, including cost and risk and other reasonable methods. In general, such assessment of a structure shall consider:

- environmental and social aspects of design, construction, use, recycling and disposal and costs, etc., arising from them,
- risks and consequences of failure of the structure during its service life and costs of insurance covering these risks,
- costs of inspections, maintenance, planned partial renewal and repair,
- costs of operation and administration.

However, cost and risk are not considered in this Model Code as part of the performance requirements of a structure.

The relevant impact categories include:

- urban air pollution,
- hazardous substances,
- destruction of the ozone layer,
- global warming,
- eco-toxicity,
- acidification,
- eutrophication,
- photochemical oxidants,
- land use,
- waste material,
- resource consumption.

The recommended verification methods are given in clause 7.7.

3.4.2 Performance requirements for environmental impacts

A structure shall be designed in such a way that the impact on environment is appropriately taken into consideration in the life cycle.

Performance requirements for environmental impacts shall address, depending on the objects of protection, the following issues:

- impact on human health,
- impact on social property,
- impact on biodiversity,
- impact on primary productivity.

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However, it is generally difficult to set up an appropriate indicator by an end-point approach such as performance requirements. Therefore, inventory items, such as CO_2, NOx, SOx, wastes, etc., will be used as performance indicators.

More detail information on the environmental aspects of concrete and concrete structures is available from *fib* bulletins 18, 21, 23, 28 and 47.

For sustainable development on Earth, fundamentally we have to prevent global warming, which is thought to be caused by greenhouse gases, such as CO_2. In particular, the Kyoto Protocol to the United Nations Framework Convention on Climate Change specifies targets with regard to the limitation of emissions of greenhouse gases. In particular the aggregate anthropogenic carbon dioxide equivalent emissions of the greenhouse gases shall not exceed the assigned emission limitation and reduction commitments, with a view to reducing the overall emissions of such gases by at least 5% below the 1990 levels in the commitment period 2008 to 2012. However, it is becoming important to reduce CO_2 even more drastically, such as 50-80%.

Accordingly, performance requirements environmental impacts can refer to:

- selection of materials,
- structural design,
- execution methods,
- use,
- maintenance procedures,
- demolition and waste disposal,
- recycling procedures,
- energy and resource consumption,
- required limits with regard to CO_2 emissions, water pollution, soil contamination, dust, noise, vibration, chemical substances.

The procedures for verification of environmental impacts are given in subclause 7.7.1.

3.4.3 Performance requirements for impact on society

A structure shall be designed in such a way that the impact on society is appropriately taken into consideration in the life cycle.

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The assessment of impact on society addresses the intended and unintended social effects, both positive and negative, of the project and any social change processes caused by the project.

Performance requirements for impact on society shall be set by using appropriate indicators.

The procedures for verification of social impact are given in 7.7.2

3.4.4 Performance requirements for aesthetics

A structure shall be designed in such a way that it has a pleasing aesthetic appearance, with appropriate integration into its surroundings.

When a structure is designed, there are several aspects to be considered. One of the most important aspects in design is "safety." The aesthetics are also considered to be part of the structure's value. On the other hand, it has been also pointed out that the aesthetics of a structure include an element of subjective judgement. In civil engineering structures a structure with a logical and simple flow of forces may be considered beautiful. In case of buildings, the intention of a designer may be emphasized in an extreme shape.

A beautiful structure can only be achieved if in addition to efficient functioning the aesthetics are developed from the beginning as an essential part of the global structural concept. Owners and engineers have a responsibility and duty to contribute to the aesthetic aspect of a structure, at a reasonable cost.

For a more detailed discussion, reference is made to *fib* Bulletin 9.

Structures are built in various sites, which include urban areas and the natural environment. Therefore, the harmony between a structure and its environment should also be taken into consideration for the creation of overall beauty.

Performance requirements for aesthetics address:

- visual appearance of the structure,
- harmony of a structure and its environment.

Performance requirements for aesthetics can refer to:

- choice of shape and composition,
- selection of colours, textures and materials,
- integration into the surroundings.

The recommended verification methods for aesthetics are given in 7.7.3.

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The through-life management of a structure, as part of the service life design and conservation processes, is discussed in chapter 9.

The word economic may need to be interpreted in the widest socio-economic sense. This may include not only the direct building costs, but also costs of exploitation, maintenance and repair, costs of decommissioning, user costs and environmental impact should be taken into account as appropriate.

Optimisation involves making trade-offs between competing objectives. Interactions and interdependencies between factors such as cost, profits, risk and quality need to be considered. Accordingly the process of making LCM evaluations should be approached with caution. As an optimisation problem, the goal of LCM has been to minimise the expected costs on a net present value basis; but increasingly the expectation is that this should be done in conjunction with minimising adverse environmental and social impacts.

In contemporary engineering practice a practical approach is to minimise the costs associated with achieving the required performance (i.e. to meet relevant performance criteria during the service life at the required reliability level) whilst achieving an appropriate (minimum) quality requirement.

3.5 Life Cycle Management

3.5.1 General

Life Cycle Management (LCM) is the overall strategy that is to be used in managing a structure through its development and service life, with the aim of improving its efficiency from a business / engineering point of view, ensuring that it meets the associated performance requirements defined at the time of design or as may be subsequently modified during the service life of the structure.

LCM is a way of facilitating choices between various design, construction and conservation options on the basis of economics, sustainability and / or other criteria.

In general, LCM seeks to optimise the balance between factors such as cost, profits, risk and quality, durability, sustainability, etc. The LCM process seeks to consider these items in a coherent and integrated way in the process of design, construction, use and conservation of a structure.

A fully integrated approach to LCM is complex and requires realistic Life Cycle Cost (LCC) calculations assuming appropriate service lives for the various elements and components making up the structure.

In this Model Code, quality measures and quality requirements are given in subclause 3.5.2 on Quality Management. Specific methods of achieving required performance of structures at different phases of the life cycle are given in chapter 8 for design , in chapter 9 for construction, in chapter 10 for conservation and in chapter 11 for dismantlement, recycle and reuse.

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Quality Management (QM) is a comprehensive approach to help all parties involved in design, construction, use and dismantlement / demolition of the structure to ensure appropriately high standards of quality and service are achieved while systematically seeking to reduce costs and impacts associated with through-life care and conservation of the structure.

Communication between parties involved in the development of the project is vital. It is important that the client remain engaged in the process even after his basic needs have been defined. It is essential to monitor progress and communicate with the client throughout the whole project development.

Communication needs to take place throughout the whole process from project inception to its life-end. The iterative nature of the design process needs to be recognised. For most of the individual phases of the project communication procedures are generally formalized. But at interfaces communication should get special attention. This is especially the case at the start of the design phase where realistic, feasible and clear requirements and criteria need to be agreed between the client and the designer. This is often a iterative process where the designer should support the client by providing feed-back on how various starting points may affect economical and technical feasibility of the scheme, its sustainability and to advise upon alternatives.

3.5.2 Quality Management

3.5.2.1 General

Quality Management (QM) is a lifecycle process for ensuring that concrete structures achieve the required quality and performance.

The main principle of LCQM is to address quality issues at their root cause. In order to establish adequate quality in the finished structure, quality issues need to be addressed at an early stage in the overall design and construction process.

QM enables quality improvement through quality planning, that comprise quality assurance and quality control issues, at all stages of the project: design (see subclause 3.5.3), construction (see subclause 3.5.4), conservation (see subclause 3.5.5) and dismantlement (see subclause 3.5.6).

To make QM effective, there must be a clear and unambiguous understanding between the owner and the designer about the performance requirements and criteria, along with the strategies to be applied in the design, construction, conservation and dismantlement / demolition phases (including the maintenance strategy of the project).

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Appropriate planning allows the parties involved to ensure alignment between project and quality goals. For proper quality planning, it is necessary to determine quality goals and quality metrics, and to use an agreed set of criteria and a standard methodology for defining the desired levels of quality.

ISO 10005:2005 "Quality management – Guidelines for quality plans" gives further advice on the development, acceptance, application and revision of quality plans.

Requirements for quality assurance and quality control may be defined in terms of parameters such as Design Supervision Levels, Execution Classes and Condition Control Levels. A systematic approach using these concepts is given in the *fib* Model Code for SLD and also in the CEN Eurocodes for structural design. Minimum levels for the quality assurance and quality control may be defined in national legislation of some countries.

Reviews are an important aspect of Quality Assurance and Quality Control, and therefore of the general management of the overall design and construction process. Reviews should be planned in advance and their timing should be linked to decisive milestones within the overall schedule of activity. It is desirable that the first review is undertaken shortly after completion of the basis of design phase / at the start of the design, in order to have the basis of the design reviewed and, as such, confirmed.

A typical contents list of a PQP is as follows:

- general: description of the project, description of the assignment, quality objectives in general, distribution and revisions of the PQP, abbreviations;
- financial: contract data, change procedure, cost control, invoicing, project evaluation;
- risk management: risk inventory, risk mitigation and management, safety and health plan;
- organisational: project organisation, sub-consultants/contractors, interface management, communication procedures (reporting, meetings);

3.5.2.2 Project Quality Plan

Quality planning is required to give structure to the measures, to assure coherence between the various disciplines and stages of development and to allow quantitative management of quality. For quality planning, a Project Quality Plan (PQP) is widely used and often required.

Project Quality Plan (PQP) shall define the tasks and responsibilities of all parties involved, adequate control and checking procedures and the organization and filing of adequate documentation of the building process and its results.

PQP shall comprise quality assurance and quality control issues.

The PQP shall address or refer to:

- objectives and criteria applicable to the project,
- organizational structure,
- technical and organizational working methods and procedures,
- lines of communication,
- tasks and responsibilities,
- quality management measures applicable to the outsourcing / subcontracting of activities,

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- time schedule: planning schedule, milestones, document planning, review and audit planning schedule;
- information management: document control, acceptance procedures, change management, filing, as-built documentation, secrecy agreements;
- process quality: overview of applicable procedures, progress reporting, non conformities, audits, customer satisfaction, project evaluation;
- product quality: functional requirements, boundary conditions, basic data and criteria, codes and practices, verification plan, design validation plan, design and drafting tools.

Checklists may be useful for implementation of a Project Quality Plan. Examples are given in CEB Bulletin 194.

For standard schemes and handled by a single source company with a certified Company Quality Plan, a simple reference can be made to such a Plan for most of the items to be addressed in the PQP. For more complicated schemes and/or schemes handled by a combination of partners, the PQP will generally be project-specific. In such cases the ISO 9000 series of codes may be a useful support.

There is a crucial interaction with the skills of the individuals involved. Although subjective, requirements for skills and qualifications need to be assessed. Where these are deficient, training and education measures should be instigated or more appropriate staff assigned to the project, or a combination of these measures implemented.

Whilst the ISO 9000 series of standards is accepted world-wide as the model approach for quality management, with the focus in contemporary standards upon the concept of the "continual improvement" of an organisation's management system in order to improve overall performance and customer satisfaction, sole reliance on this concept can present various difficulties in respect of the construction of concrete structures on site. In this context there is a need to prevent the occurrence of nonconformities in the "one-off" circumstances associated with the site placement of concrete in a particular structure or component, especially where these may impact upon the structural capacity, performance or durability of the finished entity. Thus

- key personnel involved,
- handling of non-conformities.

The extent of a PQP may differ: depending on the nature and size of the project, type of contract and parties involved, each development phase should have a plan or the plan may cover a number of phases. Coherence and transfer of information and / or instructions between phases is critical. For non standard and/or complicated projects, a project specific risk analysis should be conducted to define the issues to be addressed specifically in the PQP.

Quality cannot be assured by procedures and an organisational structure only. Therefore, the methods of improving quality practices need to be introduced into the process for potential benefits to be realized.

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there needs to be a focus in the practices and procedures for assuring quality upon preventive measures that minimise the risk of nonconformities occurring. This is compatible with a risk-based approach and related methodology to quality management.

For more information upon pre-construction planning, the role of the project specification and of quality management during execution of concrete structures, reference may be made to Annexes F and G of *fib* Bulletin 44: "Concrete structure management - Guide to ownership and good practice", *fib* 2008.

The Life Cycle File should be initiated during the design phase and populated with the first set of the relevant information / documents. Later phases further complete the Life Cycle File. The Life Cycle File also serves as an interface document managing the collection and transfer of information from one phase of the project into the next.

During the design stage, the Life Cycle File will be populated with functional requirements, basic data and boundary conditions, selected engineering approach and applied models, engineering results and applicable criteria, specific instructions for construction (specifications for workmanship and materials, assumed or mandatory construction sequence), risk file, results of tests, certificates, etc.

3.5.2.3 Life Cycle File

To allow effective and efficient QM, the project quality status / progress should be documented. Therefore, development of the Life Cycle File should be integrated with QM activities.

The Life Cycle File is a living document, which continues to be developed throughout the entire Life Cycle of the project. Thus data on the quality metrics for the Life Cycle File are collected throughout the lifecycle, through comprehensive verification and validation processes, including process audits, peer reviews, analysis and testing, as appropriate.

In the Life Cycle File information to manage the project throughout the service life should be available. Therefore, the Life Cycle File should contain all relevant data, such as relevant engineering documents, engineering instructions, specifications, test results and certificates, e.g. built documentation, maintenance strategy, factual maintenance data and the decommissioning strategy of the scheme.

The Life Cycle File shall be populated with information extracted from the following documents:

– Design File, see subclause 3.5.3.2,

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At the beginning of construction, the Life Cycle File will be populated with requirements for execution of the works and the condition control during the service life of the structure.

At the end of construction, the Life Cycle File will be populated with as-built information from construction and associated tests. As such, the document will allow owners to develop an optimised maintenance strategy and will provide the factual information needed to develop future modifications of this.

The Birth Certificate Document (BCD) is a component of the overall Life Cycle File documentation. It contains details about the as-built condition of the structure (see 3.6.2.4). The BCD should correspond to the information included in the Design File.

During the service life, actual maintenance and findings must also be included in the Life Cycle File.

After dismantlement of the structure, essential information from the Dismantlement Document shall be included in the Life Cycle File.

The design process provides a way whereby the initial desire of an owner to get a specific performance realised is interpreted and then developed into the detailed information required by the contractor to actually build the project. An iterative process is employed to take the initial starting points / outline of the owner requirements through to detailed specifications and drawings. Through a series of cycles the plan takes shape, its contents become defined and then refined. The cycles form different stages which create specific outputs that support the owner's decision making process. Without an iterative design process that engages effectively with the owner's decision making process, there may be a risk that substantial re-working of the design may be required at a later stage.

– "As-Built Documentation": Birth Certificate Document, see subclause 3.5.4.2,

– Service-Life File, see subclause 3.5.5.2.

– Dismantlement Document, see 3.5.6.2.

3.5.3 Quality Management in Design

3.5.3.1 Objectives

To enhance the effectiveness and efficiency of the design process it is generally split into a number of phases. These must be formulated in a way that is compatible with the decision process employed by the owner.

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Although there are various ways in which progress through the design stages can be organised, clients decision models are generally based on go / no-go milestones, with a requirement for an associated increase in the accuracy of the prediction of the project budget required. Generally, the engineering input is gradually similarly increased through the various stages of design development. The greater the confidence that the project is correctly formulated and is likely to proceed; the greater is the justification for more detailed design effort. It gives an effective model of how to phase the design process. Desired accuracy levels (plus and minus) will typically be about 30% in the scouting phase, about 20% at the basis of design stage, about 10% at project specification stage, about 5% at the final design / detailed design stage. Whilst these values have typically related to project costing, they could be equally applicable to factors such as environmental impact and the evaluation sustainability parameters.

Generally, following design stages can de distinguished:

- Briefing phase, see 3.5.3.3,
- Scouting phase, see 3.5.3.4,
- Basis of Design phase, see 3.5.3.5,
- Project Specification phase, see 3.5.3.6,
- Final Design phase, see 3.5.3.7,
- Detailed Design phase, see 3.5.3.8.

3.5.3.2 Design File

The Design File of the project shall be initiated at the Briefing phase. Upon completion of the Detailed Design phase, all relevant documents from the design shall be included in the Design File. The Design File shall contain the following documents:

- Client or Owner's Brief, see 3.5.3.3,
- Scouting Report, see 3.5.3.4,
- Service Criteria Agreement, see 3.5.3.5,
- Project Specification Document, see 3.5.3.6,
- Final design report, see 3.5.3.7,
- Calculations report, technical report and design drawings, see 3.5.3.8.

Upon completion of the design, the Design Files shall be included in the Life Cycle File and handed over to owner for further development in the next stage of the project.

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More specific performance goals are more closely and better defined, which increases their effectiveness. Goals that are not clear and measurable may be open to different interpretations, which is likely to limit their effectiveness.

Setting realistic performance goals involves engineering and economic analysis to determine what is possible and how much it will cost.

It is desirable for the performance requirements of the structure to be established by the owner in consultation with the stakeholders and in conjunction with the project team / (owner's) professional team.

The project team / (owner's) professional team.is a group of persons who are skilled in the various technical aspects and processes required for the design, construction and maintenance of structures. This group will include the designer, who is more generally referred to elsewhere in this Model Code.

The stakeholders shall not withdraw from the interaction / communication process once his basic needs have been established. It is important to monitor progress and communicate with the owner during all stages of project. Communication needs to take place throughout the whole project process, from project inception to its life-end.

In many instances the brief is an evolving document. In the Briefing Phase the brief does not provide all the answers, but it should pose questions and challenges for the designers. The discussion and clarification of the final client's / owner's requirements comes during the Scouting Phase, see 3.5.3.4.

Key issues to consider when developing an initial brief include:

- type of structure and its location (decided after examination of other means of achieving the general objectives – a process which is undertaken before deciding to build),
- planned function(s) of the structure and its components,

3.5.3.3 Briefing Phase

Objectives

When applying a performance-based approach, general performance goals shall be developed during the initial stage of design or assessment. General objective statements shall be used to define the global performance requirements for all performance categories.

The stakeholders shall define the desired performance of the structure. Minimum performance requirements, such as those specified in applicable national standards, should not be violated.

Client or Owner's Brief

The client's or owner's requirements shall be written down in a formal document called the (initial) client / owner's brief.

The client / owner's brief addresses the relevant needs and aims of the project, resources to be provided by the client / owner, the details of the project and any appropriate design requirements. It sets a framework within which all subsequent briefing (when needed) and design can take place.

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- requirements for appearance /aesthetic (initially and throughout the life of the structure),
- requirements for usable space, dimensions, services and fittings,
- the period of service, what constitutes end of service life and the requirements for the structure at the end of this period,
- need of future changes of use (e.g. to increase flexibility and minimise the risk of obsolescence),
- time, budget and/or quality limitations.

It is common practice to limit the design effort expended because the feasibility of the project will usually be uncertain at this stage. The objective of making an initial estimate of the overall project cost with limited staff input (and hence incurred cost) will normally require suitably experienced personnel to develop an outline project concept and to make judgements about potential cost, sustainability impacts etc. At this stage the target accuracy for the estimate of overall project cost might typical be +/- 30%. However, this requirement could also be applied to other factors such as environmental impact and the evaluation sustainability parameters.

One approach which is commonly adopted is to review relevant former schemes, adapting them to the specific circumstances and requirements of the new project. To do so effectively with limited staff effort, the designer needs to be well experienced and to understand the general cost drivers associated with the new and previous project concepts. The goal is to identify project specific, decisive points of attention / cost drivers that need to be considered in detail during the next phase of the development of the design.

Goals in the initial brief need to be prioritised into 'must haves' and 'desirables' in order to guide the project team and help them make compromises when the need arises (e.g. prioritising of time, cost and quality).

3.5.3.4 Scouting Phase

Objectives

The Scouting Phase is an initial (basic / simplified) feasibility evaluation of the project / scheme. Generally it will be based on an outline project concept established from the global performance requirements defined in the Briefing Phase.

To support the owner's decision making process, it will be usually be necessary to prepare an indicative budget.

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At this stage the target accuracy for the estimate of overall project cost might typical be +/- 20%. However, this requirement could also be applied to other factors such as environmental impact and the evaluation sustainability parameters.

Examples of basic data include:

- geotechnical data,
- metocean data,
- topographical and bathymetrical data,
- climatological data,
- environmental data (earthquake, hurricanes, the aggressity of the service environment),
- material properties.

Operational/ maintenance requirements may comprise,

- the use of de-icing salts,

Document

The output will be in the form of a Scouting Phase Evaluation Report upon the feasibility of the project / scheme, with global functional requirements, outline concept and budget estimate.

3.5.3.5 Basis of Design Phase

Objectives

During this phase the functional requirements, basic data and design criteria will be developed and the service criteria agreed. A conceptual design will also be developed to support a more accurate budget estimate. Quite some effort is required at this stage as the Basis of Design should be agreed, fixed and frozen upon completion of this stage. An essential part of this phase is the Service Criteria Agreement.

Service Criteria Agreement

The service criteria shall be clearly specified in the Service Criteria Agreement, which shall comprise:

The Service Criteria Agreement shall comprise:

- general aims for the use of the construction works,
- basic relevant data, including third party interactions,
- operational and maintenance requirements,

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– replacement strategy of components subjected to wear,
– flexibility in terms of space requirements, future extension or load bearing capacity.

The objectives and the degree of protection shall be determined on the basis of a risk evaluation.

Fixing the performance criteria for serviceability and safety shall follow consideration of:

– the hazards, together with means by which the hazards might be avoided, reduced, mitigated, controlled, managed or resisted,
– the type and consequences of deterioration and failure,
– the resistance and mitigation mechanisms.

Fixing the specified (design) service life for which the structures are to be designed and the residual service life for existing structures shall follow consideration of factors such as:

– the required service life of a structure, as given by the owner and / or stakeholders,
– what constitutes the end of service life in individual parts of the structure
– a need for differentiation of service life for individual parts of the structure (e.g. depending on factors such as their replaceability),
– the implications of other service criteria e.g. with regard to structural analysis, maintenance and quality management.

Fixing the target reliability level shall follow consideration of factors such as:

– the type and consequences of failure;
– special requirements of the stakeholders,
– objectives for consideration of, protection against and the treatment of special risks,
– loadings and loading combinations,
– codes and regulatory requirements.

In particular, the specification in the service criteria shall address:

– performance criteria for serviceability and safety, see subclause 3.3.1,
– service life constraints, see subclause 3.3.2,
– reliability constraints, see subclause 3.3.3.

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– the amount of acceptable damage;
– the importance of the structure in dealing with a catastrophe following an accidental event;
– the expenditure to reduce the risk;
– the possibilities of monitoring, maintenance and repair as well as the corresponding expenditure,
– a need for differentiation of target reliability level depending on the limit state and reference period, either for the whole structure or its structural components.
– possible hazard scenarios shall be considered and evaluated, and suitable measures shall be specified in order to keep the hazards under control or to limit them to an acceptable extent

The following principles may be applied to mitigate the hazards:

– elimination, prevention or hazard reduction,
– controls or alarm systems,
– choice of structural systems which are less susceptible to the hazards under consideration,
– choice of structural systems which can tolerate local damage as well as the loss of a structural member or a whole part of the structure without failing totally,
– choice of structural systems which do not fail without prior warning,
– limiting the spread of fire by the provision of fire compartments,
– choice of suitable structural materials that, if well maintained, will not substantially degenerate during the required service life,
– accepting a shorter service life for structural components, which may be replaced one or more times during the specified service life,
– appropriate structural analysis and dimensioning,
– careful detailing
– dimensioning the structure in a manner that allows for / compensates for deterioration during the specified service life,

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- choice of an appropriate execution method,
- execution carried out as planned and with the necessary care,
- planning and applying suitable protective and mitigating systems,
- appropriate monitoring and conservation, including inspections at fixed or condition-dependent intervals, and necessary preventative intervention or remedial activities.

Fixing the performance criteria for sustainability shall follow consideration of factors such as:

- the importance of the structure to the global, regional and local environments,
- the required achievements with respect to sustainability, as given by the owner and / or stakeholders,
- the type and consequences of not meeting the required achievements with respect to sustainability,
- the flexibility to allow future extensions and/ or modifications of the functional requirements,
- the expenditure to reduce the risk of not meeting the required achievements with respect to sustainability,
- a need to differentiate the required achievements with respect to sustainability for individual parts of the structure (e.g. depending on factors such as their replaceability)

Well-defined performance requirements allow evaluation of the achievement of performance goals throughout the design, execution, operation and dismantlement / demolition of the structure

At this stage the target accuracy for the estimate of overall project cost might typical be +/- 10%. However, this requirement could also be applied to other factors such as environmental impact and the evaluation sustainability parameters.

- performance requirements for sustainability, see clause 3.4.

Progress toward the performance requirements should be traceable.

3.5.3.6 Project Specification Phase

Objectives

With the basis of design as the starting point, the design will be developed first into a preliminary design. Specifications for workmanship, materials and detailed design will then be developed. Significant effort is generally required at this stage.

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The output of this stage can serve as the technical part of the invitation to bid for a Design/Construct contract.

Factors influencing the constructability / economic feasibility of the project may include:

- accessibility of the site,
- bearing capacity of the subsoil at site in respect to anticipated construction equipment loadings,
- lifting capacity at the site,
- minimum/maximum size of structural components,
- clearances to battery limits,
- quality, availability and reusability of construction materials,
- restrictions regarding the design and construction times, and the budget limitations,
- legal aspects (laws, ordinances, directives),

At this stage alternative structural concepts will generally be developed and evaluated against each other. Numerous aspects should be included in this judgement, potentially including the following:

- robustness of the concept
- constructability of the concept,
- the planning schedule for the concept
- economy of the project / overall Life Cycle Cost and its achievements with respect to sustainability parameters,
- feasibility of future extensions,
- reliability of the concept as a whole and critical components especially,
- maintenance and repair considerations,
- dismantling of the structure / demolition aspects.

In order to develop the structural concept issues such as the following need to be taken into account:

- the service criteria agreement,
- constructability/economic feasibility of the scheme,

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- construction methods, transport and assembly techniques,
- equipment and facilities for monitoring and maintenance measures,
- maintaining the use of traffic structures and lifelines (pipelines, etc.),
- demolition approach at the end of useful life,
- Life Cycle Cost considerations.

The following deviations should be considered:

- deviation from the assumed values of the actions,
- deviations from the planned values of the ultimate resistances of the structure or the soil,
- eccentricities due to construction tolerances, imperfections in the dimensions of structural members,

A structure can be designed for flexibility, anticipating on possible future changes of its function.

A clear statement shall be given indicating which data are fixed and frozen, which data needs further development, which data have been assumed and what assumptions have been made.

- the critical actions and action effects, as well as the sensitivity of the concept to deviations from the anticipated values,

- the foreseeable service situations shall be considered and evaluated, and appropriate measures taken to ensure serviceability,
- aspects of sustainability in agreement with the requirements of the owner, stakeholders or governing authorities.

Project Specification Document

The Project Specification Document needs to include information such as the following:

- the chosen structural system,
- the specified (design) service life,
- the service conditions considered,
- the hazard scenarios considered,
- the requirements for structural safety, serviceability, robustness and sustainability, together with the measures needed to achieve them, including attribution of responsibilities, processes, controls and corrective mechanisms,
- a reliability qualification statement for the data used for design,

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In the context of partial safety factor verification, ensuring the required reliability level is achieved requires adequate consideration of the uncertainties regarding actions, structural modelling and the determination of action effects. The differentiation of the partial safety factors depending on the uncertainties in actions, material properties and applied models is addressed in chapter 4.

At this stage the target accuracy for the estimate of overall project cost might typical be +/- 5%. However, this requirement could also be applied to other factors such as environmental impact and the evaluation sustainability parameters.

- the most important dimensions, construction material properties and construction details,
- the assumed soil conditions,
- the important assumptions in the structural and analytical models,
- the accepted risks,
- advised/required additional investigations,
- other conditions relevant to the design,
- comments on the envisaged methods of construction,
- specifications for detailed design, materials and workmanship.

The extent and content of the Project Specification Document shall be adapted to the importance of the structure and the associated hazards and environmental risks.

3.5.3.7 Final design phase

Objectives

At this stage all primary structural members will be specified and typical details will be designed.

Structural analysis and static calculations report

The structural analysis should consider the behaviour of the structure in relation to the envisaged dimensioning situations, taking into account the relevant factors that significantly influence the potential performance of the structure / the structural components concerned.

The methods of structural analysis shall be based on established theories, experimentally confirmed if necessary, and engineering practice.

The results of the structural analysis shall be checked for credibility e.g. should be subject to a review utilising general engineering judgement.

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Detailing, limit measures and special provisions supplement the use of models for various purposes, such as:

- to avoid superfluous calculations,
- to satisfy the minimum performance requirement / comply with deemed-to-satisfy provisions with regard to unidentified or poorly

Final design report

The final design report shall contain all data used for design, all phases considered, including construction phases, applied static schemes, structural analysis, applied criteria and material properties, construction method considered and a traceable demonstration of compliance with the Project Specification.

The report shall also contain a risk file. The risk file must present the identified risks, how they have been managed and, if any, instructions for the next phases of design and construction.

Drawings shall present the overall layout of the project, as well as the geometry, shape and dimensions of primary structural members and typical details.

3.5.3.8 Detailed design phase

Objectives

The output of this stage shall allow construction of the project. All calculations needed to demonstrate compliance with codes and requirements/ specifications of the project will be prepared during this stage. The level of detail of drawings and specifications / site instructions shall allow unambiguous understanding by the contractor of what is required and how the scheme must be constructed, as well as how compliance with the documents must be demonstrated. A risk file must be prepared to inform contractor of the risks involved, how these risks have been handled in the previous stages of design and how the remaining risks must be handled. Issues which require special attention in this respect must be clearly noted on the construction drawings.

Dimensioning

Dimensioning concerns the determination of the dimensions, the structural materials and the detailing of a structure on the basis of structural and execution-related considerations or numerical verifications.

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quantified hazards. These measures include provision of a minimum resistance to lateral forces, multiple load paths and ties between structural components (see sections 2.1 and 3.2.3 of CEB Bulletin 191),

- to ensure the validity of calculation models, e.g. by minimum ratios of reinforcement,
- to ensure a good standard of execution and/or durability e.g. by rules for bar spacing and concrete cover depth.

ENV 13670-1 "Execution of concrete structures – Part 1: Common rules" defines a set of minimum requirements for the execution.

The dimensioning may be assisted by testing, for example if:

- actions, structural materials or soil properties are not adequately known,
- no appropriate analytical models are available,
- the structure contains components for which there is limited experience and which have a critical influence on the reliability of the structure.

Calculations report

The basis and the results of the detail design phase shall be documented.

Technical report and design drawings

The dimensions, the structural materials and the detailing of a structure as determined during dimensioning shall be documented in the technical report and design drawings.

3.5.4 Quality Management in Construction

3.5.4.1 Objectives

To meet the minimum requirements for quality management in construction, as specified in the execution standard and as assumed in the design.

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The expected outcome would be that either (a) the conformity evaluation confirmed that the design assumptions had been met or (b) give the basis for corrective measures.

BCD would provide a record of at least the following:

- verification of the as built condition of the structure and a record of the standard of execution / variability achieved during construction,
- a known Benchmark for reference on service life design matters,
- initial data as required for the verification of the limit states (in particular limit states associated with durability).

The data gathered in BCD would also allow:

- a first review of service life predictions based upon the initial measured data.
- –assessment of compliance / non-compliance with the design requirements and provide support for decision-making regarding any interventions / remedial activities required

A proper inspection regime during the service life of a structure and documentation of the inspection results will give the owner the possibility to perform condition control during the service life and to apply protective measures when the expectations for the service life design are not met.

3.5.4.2 "As-Built Documentation": Birth Certificate Document

The "As-Built-Documentation" shall be a reliable representation of the project as actually constructed. It shall include the results of the initial inspection of the completed work / project. The extent of the inspection of the completed work and the content of the "As-built-documentation" will depend on the nature and size of the project, on the design assumptions and on the verification methods, as well as on the quality management and the control measures for the project.

Information included in "As-Built-Documentation" shall allow a conformity evaluation to be performed upon the completed work / elements of the project.

An extract of the "As-built-Documentation", named Birth Certificate Document (BCD), will include the results of an initial inspection of a new structure. The content of the BCD is usually limited to the documentation of the direct input parameters for the future condition control of the structure, such as cover thickness to the reinforcement, diffusion coefficient for the concrete cover etc

BCD might serve as a basis for monitoring of the condition of the structure and for planning conservation activities during its service life. Recommendations upon conservation procedures, which depend on the specifics of the project, are given in Chapter 9.

3.5.5 Quality Management in Conservation

3.5.5.1 Objectives

To control and manage the activities and measures taken which seek to ensure that the condition of a structure remains within satisfactory bounds to meet the performance requirements for a defined time; that is in respect of structural safety and functional performance requirements, which may

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For new structures, recording during conservation would be expected to draw upon information obtained for and detailed in BCD. For existing structures, there is the expectation that recording during conservation would draw upon / contribute to the preparation of a Re-Birth Certificate Document (RCD), depending on whether a previous version had been prepared and was to be up-dated.

The RCD would provide a record of at least the following:

- verification of the condition of the structure after an intervention (preventative or remedial) has been made and a record of the standard of execution / variability achieved in that process and previously,
- updated (in-service) Benchmark for reference on service life design matters,
- updated data as required for revision of verification of the limit states (in particular limit states associated to durability).

The data gathered would also allow:

- a review of service life predictions based on updated (in-service) measured data and a revised prognosis on future performance,
- assessment of compliance / non-compliance to design requirements and planning for any future preventative / remedial activities required.

include considerations such as aesthetics. This is achieved through activities which may involve condition survey, monitoring the performance of the structure through-life, condition assessment, condition evaluation, decision-making and the execution of any necessary interventions; together with recording of the conservation activities and measures undertaken.

3.5.5.2 Service-Life File

The Service-Life File shall document the conservation activities carried out during the life of the structure. The Service-Life File shall include results of inspection of the structure or its components that take place during the service life of the structure. Such a record shall include:

- classification of the structure and conservation strategy,
- reference to relevant agencies, drawings, details of the immediate and surrounding environment,
- details concerning inspection and evaluation procedures (including results of inspection and monitoring carried out, results of deterioration, rate estimation and evaluation of the structure),
- details of the plan and actual execution of the preventive or remedial interventions carried out.

An extract of the Service-Life File, named the Re-Birth Certificate Document (RCD), includes results of in-service inspection of an existing structure after preventative or remedial action has been undertaken. The content of the RCD usually corresponds to the information included in the Birth Certificate Document.

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There may be a range of additional activities associated with the dismantlement / demolition works, such as those involved in the cleaning-up and / or treatment of the site in order to decontaminate it / make it suitable for future use or redevelopment.

For dismantlement a plan should be made that regards at lest the following aspects:

- provision of adequate structural safety in all stages of dismantlement
- minimization of societal hindrance by dust, dirt and noise
- minimization of contamination of soil respecting at least the local regulations
- conditioning and removal of operating wastes in such a way that the principles of sustainability as formulated in clause 3.4 are satisfied
- recycling the appropriate parts of the dismantled material
- cleaning the site and reintegration in the environment after dismantlement

The Service-Life File shall be preserved while a structure remains in service. It may also be desirable to keep such records for an indefinite period for reference purposes for the design, construction and conservation of other similar structures

The records shall be kept in an easy to understand format.

3.5.6 Quality Management in Dismantlement

3.5.6.1 Objectives

To control and manage the activities and measures taken to allow the safe removal an existing structure and the clearance of the site as appropriate by means of:

- dismantling the structure into its component parts, or
- demolishing the structure by physically breaking it up

or a combination of such measures, facilitating the re-use and / or re-cycling of the original component parts and materials into new uses in a manner that minimises the associated environmental and social impacts.

3.5.6.2 Dismantlement Document

The dismantlement / demolition document sets down the activities, measures and procedures which will allow the safe removal an existing structure and the clearance of the site in a manner that minimises the associated environmental and social impacts.

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4 Principles of structural design

4.1 Design situations

Actions, environmental influences and structural properties may vary with time. Such variations, which occur throughout the lifetime of the structure, should be considered by selecting design situations, each one representing a certain time interval with associated load cases and other hazards, conditions and relevant structural limit states. The dimensioning situations considered shall include all foreseeable conditions that can occur during execution and use.

In the design procedures, various design situations should be identified as relevant, by distinguishing:

- persistent situations, which refer to conditions of normal use of the structure and are generally related to the working life of the structure;
- transient situations, which refer to temporary conditions of the structure, in terms of its use or its exposure;
- accidental situations, which refer to exceptional conditions of the structure or its exposure;
- seismic situations, which refer to conditions of the structure under an earthquake event.

In many cases judgement is necessary to supplement codified provisions, in order to identify those design situations that are to be taken into account for a particular structure.

For persistent situations a reference period t_R is commonly considered equal to the design service life for new structures or the residual service life for existing ones. Usually, for persistent situations in case of new structures a reference period t_R of 50 years is adopted for buildings and 100 years for bridges and tunnels.

The various types of design situations are defined in section 3.2.2 of CEB Bulletin 191.

Accidental action is defined as action of usually short duration, that is unlikely to occur with a significant magnitude on a given structure during the design service life, but its consequences might be catastrophic, e.g. fire, explosions or impact from vehicles. The insensitivity requirement is defined in section 2.1 of CEB Bulletin 191.

Unlike accidental actions, which cannot be associated with a statistical probability of exceedance, seismic actions can.

Construction states can be considered as persistent or transient dimensioning situations. Accidental dimensioning situations involve either the accidental situation itself or they refer to the situation immediately after the accidental event.

Examples of appropriate length of design service life for new structures are given in subclause 3.3.2 (see also EN 1990, Chapter 2).

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See also EN 1991-1-6 where for specified nominal duration shorter return periods are considered.

In accidental design the failure probability is conditional on occurrence of that event.

In seismic design the failure probability is found by convoluting the probabilities of occurrence of seismic actions greater or less than the design one during the design service life for new structures or the residual one for existing structures.

Failure of the structural components and failure of the system shall be analyzed for all possible damage states and exposure events relevant for the design situation under consideration.

Section 3.2.3 of CEB Bulletin 191 gives same guidance on the choice of a design procedure appropriate to limit damages liable to result from identified or unidentified hazards.

The general principles and the procedures for the verification of robustness are given in clause 7.9.

For transient situations a reference period t_R of 1 year is normally taken.

Accidental situations are considered to be instantaneous and the corresponding reference period t_R is defined as the duration of the design event.

In the context of seismic situations a reference period t_R is normally taken equal to the design service life for new structures or the residual service life for existing structures.

4.2 Design strategies

Structures shall be designed for all relevant design situations (i.e. persistent, transient, accidental and seismic design situations, if relevant).

Depending on the type of action or damage state, the following strategies shall be applied in design for different categories of the design situations:

- strategies applied in persistent and transient design situations for limiting the consequences of identified permanent and variable actions are:
  - design the structure to sustain the action

    or
  - design the structure to avoid the action
- strategies applied in accidental or seismic design situations for limiting the consequences of identified accidental or seismic actions are:
  - design the structure to sustain the action

    or
  - design the structure to avoid the action

  and
  - design the structure to have sufficient robustness.

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The limit states either refer to the entire structure, to structural elements or to local regions of elements.

The assessment of s (**s**) may be referred to as overall analysis, while the assessment of r (**r**) may be referred to as local analysis.

4.3 Design methods

4.3.1 Limit state design principles

The structural performance of a whole structure or part of it should be described with reference to a specified set of limit states which separate acceptable states of the structure from unacceptable states.

In general terms, attainment of a limit state can be expressed as:

$$g(\mathbf{s}, \mathbf{r}) = 0 \tag{4.3-1}$$

where

g (**s**, **r**) is the limit state function, **s** represents sets of loads (actions) and **r** represents resistance variables.

Conventionally, failure (i.e. an unacceptable state) is represented as:

$$g(\mathbf{s}, \mathbf{r}) \leq 0 \tag{4.3-2}$$

Although limit state equations representing different limit state conditions are various, the limit state function g (**s**, **r**), can often be separated into one resistance function r (**r**) and one loading (or action effect) function s (**s**). In such a case equation (4.3-1) can be expressed as:

$$r(\mathbf{r}) - s(\mathbf{s}) = 0 \tag{4.3-3}$$

Consequently, equation (4.3-3) lends itself to the following representation of failure:

$$r(\mathbf{r}) \leq s(\mathbf{s}) \tag{4.3-4}$$

4.3.2 Safety formats

Verification of a structure with respect to a particular limit state is carried out via a model describing the limit state in terms of a function (called the limit state function) whose value depends on all relevant design parameters.

Verification of the limit states shall be realised by a probability-based method. This Model Code recommends for verification of the limit states to use one of the following safety formats:

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The probabilistic safety format (sometimes referred to as fully probabilistic design method) allows explicitly including the reliability requirements in terms of the reliability index β and the reference period. This may be used for structures to be designed and for existing structures in cases where such an increased effort is economically justified, However, it will seldom be used for the design of new structures due to lack of statistical data. The probabilistic format is more suited for the assessment of existing structures, in particular for the calculation of residual service life.

The partial safety factor format is the usual way of verifying structural design. It is a simplified verification concept, which is based on past experience and calibrated in such a way that the general reliability requirements are satisfied with a sufficient margin during a defined period of time. In the future this safety format might also be applicable for the verification of service life, provided that sufficiently long term experience will be gained or a sufficient amount of data will be available for a calibration by the probabilistic method.

In the global resistance format the resistance is considered on a global structural level, as compared to local verification of sections with partial safety factors. It is especially suitable for design based on non-linear analysis, where verification of limit states is performed by numerical simulations.

The deemed-to-satisfy approach includes a set of appropriate values from a set of predetermined alternatives given in a standard. This method is the normal way of verifying service life design of new structures.

Design by avoidance is applicable both for the verification of traditional structural design and design for service life.

The variables pertaining to the various limit states may be time-dependent.

– probabilistic safety format, see clause 4.4,

– partial safety factor format, see clause 4.5,

– global resistance format, see clause 4.6,

– deemed-to-satisfy approach, see clause 4.7,

– design by avoidance, see clause 4.8,

For each specific limit state the relevant basic variables should be identified, i.e. the variables which characterize actions and environmental influences, properties of materials and soils, geometrical parameters, etc.

The variability of basic variables shall be analysed based on the available information. In case of the probabilistic format the basic variables are treated as random variables, or random fields. In case of the partial factor format, the basic variables are treated as deterministic quantities. In case of the global safety format the global resistance is treated as a random variable.

In a component analysis where there is one dominating failure mode the limit state condition can normally be described by one limit state equation. In a system analysis, where more than one failure mode may be determining, there are several such equations.

A probabilistic safety format shall be applied in accordance with the principles and recommendations laid down in the JCSS Probabilistic Model Code and in JCSS publication Probabilistic Assessment of Existing Structures.

Examples of cases characteristic for existing structures, where reliability of existing structures may need to be assessed are the following:

- doubts about the performance of the structure;
- the expiration of (design or residual) service life (e.g. granted on the basis of design or an earlier assessment of the structure);
- detection of design- or construction errors;
- occurrence of unusual incidents during use, which could have damaged the structure;
- a planned change of the use of the structure.

Examples of design situations that are out of the application range of this Model Code and shall therefore be analysed according to a probabilistic safety format are the following:

- actions and hazards laying outside the range covered by this Model Code;

For each limit state, models should be established, which describe the behaviour of a structure. These models include mechanical models, which describe the structural behaviour, as well as other physical or chemical models, which describe the effects of environmental influences on the material properties. The parameters of such models should in principle be treated in the same way as the basic variables and model uncertainties shall be analysed.

Models for the verification of the limit states can be either analytical (see clauses 7.3 - 7.8) or numerical (see clause 7.11), eventually supported by testing (see clause 7.12).

4.4 Probabilistic safety format

4.4.1 General

The main objective of a reliability analysis by the probabilistic approach is a probabilistic assessment of the safety of the structure by estimating the failure probability (or the reliability index β).

The probabilistic safety format is a very suitable approach for the assessment of performance of existing structures.

The probabilistic approach may support the design according to the partial factor format or deemed-to-satisfy approach, e.g. to ensure an appropriate robustness of structures or to account for specific requirements out of the range of application of this Model Code.

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- use of construction materials and combination of construction materials outside the usual range of experience;
- new construction materials with properties outside the range covered by this Model Code;
- service life requirements outside the range covered by this Model Code;
- reliability level not covered by this Model Code;
- extraordinary structural systems or extraordinary geometry of a structure;
- cases where failure would lead to serious consequences.

A proper choice of the distribution of the basic random variables is of importance, since the results of the reliability analysis can be very sensitive to the type of distribution adopted.

4.4.2 Basic rules for probabilistic approach

The verification of a structure with respect to a particular limit state is carried out via estimation of the probability of occurrence of failure in this case in a specified reference period and its verification against reliability requirements, see subclause 3.2.5.

With the failure criteria formulated according to Eq. (4.3-2), the probability of occurrence of failure can be generally expressed as:

$$P_f = \text{Prob}\,\{g(\mathbf{s}, \mathbf{r}) \leq 0\} = \text{Prob}\,\{M \leq 0\} \qquad (4.4\text{-}1)$$

where

$M = g(\mathbf{s}, \mathbf{r})$ represents the safety margin

If the limit state function is expressed in the form of Eq. (4.3-3) and parameters characterising actions, environmental influences, material and geometry are represented by the random variables **S** and **R**, the probability of occurrence of failure can be expressed as:

$$P_f = \text{Prob}\,\{r(\mathbf{R}) \leq s(\mathbf{S})\} = \text{Prob}\,\{R \leq S\} \qquad (4.4\text{-}2)$$

where

$S = s(\mathbf{S})$ and $R = r(\mathbf{R})$ are the basic random variables associated with loading and resistance, respectively.

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This separation is theoretically not correct, and in practice not complete, because the various factors are not mutually independent. Hence, constant values given in partial factors should be considered as approximations having limited fields of validity. This approximation of using constant values for partial factors may not apply in the following cases·

- non-linear limit state equations
- mutually correlated variables
- design by testing.

For application of partial factors to non-linear analysis see subclause 7.11.3.

The general method of deriving the updated design values to be used in the partial factor method in case of existing structures is given in ISO 2394 "General principles on reliability for structures" and ISO 13822 "Basis for design of structures – Assessment of existing structures".

These reliability margins seem to cover the whole set of uncertainties, however, a part of the model uncertainties is commonly directly covered by the codified models themselves.

This does not exclude that some actions (e.g. shrinkage) can be negligible in particular cases. What is to be considered as one individual action is defined in the corresponding standard and explained in section 4.2.1 of Bulletin 191. For prestress, see subclause 4.5.1.4.2 of this Model Code.

4.5 Partial factor format

4.5.1 General

The partial factor format separates the treatment of uncertainties and variabilities originating from various causes by means of design values assigned to variables. In this Model Code the representative values of the variables and the partial safety factors are chosen in such a way that the reliability requirements for the design of new structures, which are expressed in 3.2.5 in terms of β related to the reference period, are met.

In case of existing structures, the same principles of the partial factor format can be applied as for new structures. However, the design values of the variables (i.e. the characteristic values and the partial factors) for existing structures need to be updated in order to guarantee that the reliability requirements for assessment of existing structures are satisfied at the level discussed in subclause 3.3.3.

4.5.1.1 Basic variables

For basic variables, design values include reliability margins. For other variables, whose dispersion may be neglected or is covered by a set of partial factors, they are normally taken equal to their most likely values.

In this Model Code the following variables are considered as basic:

- actions (F), unless specified otherwise in particular clauses,
- material or product properties (f), unless specified otherwise in particular clauses (e.g. strengths, creep and friction coefficients)

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For these basic geometrical quantities, tolerances should be carefully fixed (see subclause 4.5.1.4.4) and controlled. For the other geometrical quantities, tolerances generally reflect usual practice. For all geometrical quantities it would not be realistic to specify tolerances less than twice the mean deviation expected or minimum attainable. As a consequence, tolerances may, according to the case considered, be either the basis for tile design or necessary complements to the design.

More information is found in sections 4.1 and 6.1 of Bulletin 191. Identifying and selecting the other relevant basic variables is one of the major responsibilities of a designer who faces a problem having some unusual aspects.

According to the limit state under consideration, the design conditions may have to be formulated

- either in the space of internal and external moments and forces and directly presented as in Eq. (4.3-3), or
- some geometrical quantities (a),
- variables which account for the model uncertainties (θ).

Occasionally other variables should be considered as basic variables. This may be the case for the numbers of repetitions of loads in fatigue verifications.

4.5.1.2 Design condition

With reference to the representation of failure given in Eq. (4.2-2), the design condition can be expressed in terms of design values of basic variables as:

$$g\,(F_d, f_d, a_d, \theta_d, C) \geq 0 \qquad (4.5\text{-}1)$$

where

F_d are design values of actions,

f_d are design values of material and soil properties,

a_d are design values of geometrical quantities,

θ_d are design values of the variables which account for model uncertainties,

C are serviceability constraints.

The relationship given in Eq. (4.3-4) lends itself to the following representation of the partial factor checking format:

$$S(F_d, \ldots) \leq r(f_d, \ldots) \qquad (4.5\text{-}2)$$

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– in the space of forces, as

$F_S \leq F_R$ (4.5-3)

(F_R being for example a carrying capacity), or

– in the space of stresses as

$\sigma \leq \alpha f$ (4.5-4)

– or in the space of geometrical quantities, as

$e \leq D$ (4.5-5)

where:

D is e.g. a deflection, a crack width or a plastic rotation.

Typically, the design value x_{di} of any particular variable x_{ki} is given by

$x_{di} = \gamma_i \, x_{ki}$ in case of loading variables (4.5-6a)

or

$x_{di} = x_{ki} / \gamma_i$ in case of resistance variables (4.5-6b)

where:

x_{ki} is a characteristic value strictly defined as the value of a random variable which has a prescribed probability of not being exceeded (or of being attained); in time-varying loads, a value other than the characteristic may be introduced; for material properties a specified or nominal value is often used as a specified characteristic value,

γ_i is a partial factor with a value commonly greater than unity.

Some actions (e.g. non-closely bounded hydraulic actions) should be expressed in another way, as mentioned in section 4.1 of Bulletin 191. Furthermore, for verifications relating to fatigue and vibrations, the format is generally different (see subclause 4.5.2.3 for verifications relating to fatigue and subclause 4.5.2.5 regarding limitation of vibrations).

4.5.1.3 Design values of basic variables

In this Model Code the design values of the basic variables are expressed as follows:

(a) Design values of actions:

$F_d = \gamma_F \, F_{rep}$ (4.5-7)

where

F_{rep} is the representative value of the actions, defined in 4.5.1.4.1

γ_F is a partial safety factor.

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For material properties other than strengths (e.g. modulus of elasticity, creep, friction coefficients) see the relevant parts of the chapters 5 and 6.

Numerical values of γ_M may be different in various parts of the limit state equation given by Eq. 4.3-4, especially for the calculations of s (**s**) and *r* (**r**); for example (see provisions regarding γ_M factors in subclause 4.5.2.2(b)) γ_M may be reduced for the assessment of *s* (**s**) by a non-linear analysis.

For concrete and steel, γ_M usually covers the deviations of structural dimensions not considered as basic variables and includes a conversion factor η converting the strength obtained from test specimens to the strength in the actual structure. For practical applications, see the provisions regarding γ_M in subclause 4.5.2.2(b).

Other factors, applied to f_d or implicitly included in design formulae, take into account the variations of strength due to non-standardized loading conditions.

As explained in sections 6.3 and 6.6 of Bulletin 191, γ_M may in some cases be substituted by one or two partial factors γ_{Rd},, applicable to the resistance, and a partial factor γ_m applicable to f_k.

Liquid levels representing hydraulic actions should in some cases be expressed as $a_k + \Delta a$, where a_k is a characteristic level and Δa an additive or reducing reliability margin.

A part of the model uncertainties is commonly directly covered by the codified model itself. Partial factors for model uncertainties γ_d take account of uncertainties of models as far as can be found from measurements or comparative calculations.

For existing structures model uncertainties shall be considered in the same way as in the design of new structures, unless previous structural behaviour (especially damage) indicates otherwise. In some cases model factors, coefficients and other design assumptions may be established from measurements on the existing structures (e.g. wind pressure coefficient, effective width values, etc.). For more information, reference is made to ISO 2394 “General principles on reliability for structures” and ISO 13822 “Basis for design of structures – Assessment of existing structures”.

(b) Design values of material or product property:

$$f_d = f_k / \gamma_m \qquad (4.5\text{-}8a)$$

or in case uncertainty in the design model is taken into account by:

$$f_d = f_k / \gamma_M = f_k / \gamma_m * \gamma_{Rd} \qquad (4.5\text{-}8b)$$

where

f_k is the characteristic value of the resistance

γ_m is a partial safety factor for a material property,

γ_{Rd} is a partial safety factors associated with the uncertainty of the (resistance) model plus geometric deviations if these are not modelled explicitly,

$\gamma_M = \gamma_m * \gamma_{Rd}$ is a partial safety factor for a material property also accounting for the model uncertainties and dimensional variations.

(c) Design values of geometrical quantities to be considered as basic variables are generally directly expressed by their design values a_d.

(d) Design values of the variables which account for the model uncertainties are expressed as γ_d or $1/\gamma_d$, where γ_d are partial factors for model uncertainties (e.g. γ_{Rd} associated with the uncertainty of the resistance model).

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For a resistance parameter X, the updated design value x_d can be obtained from the following procedure according to ISO 13822:

$$x_d = \mu(1 - \alpha\,\beta\,V) \text{ for a normal random variable} \quad (4.5\text{-}9a)$$

or

$$x_d = \mu\,\exp(-\alpha\,\beta\,\sigma\text{-}0.5\sigma^2) \text{ for a lognormal random variable} \quad (4.5\text{-}9b)$$

where:

x_d is the updated design value of X

α is a sensitivity factor

β is the target reliability index for an existing structure

V is the updated coefficient of variation

$\sigma^2 = \ln(1 + V^2)$

The value of β for existing structures is discussed in chapter 3.X. The values of α can be taken equal to those commonly used for new structures (0.7 for the dominating parameter, 0.8 for the dominating resistance parameter and 0.3 for non-dominating variables according to ISO 2394).

As an alternative procedure, one might also determine first a characteristic value x_k and calculate the design value by applying the appropriate partial factor γ_m.

Here:

$$x_d = x_k / \gamma_m \quad (4.5\text{-}10)$$

and

$$x_k = \mu(1 - k\,V) \text{ for a normal random variable} \quad (4.5\text{-}11a)$$

or

In the design of new structures the design values of the basic variables should be determined using representative values of the basic variables and partial safety factors given in subclause 4.5.1.4.1 (representation of actions), 4.5.1.4.2 (representation of prestress), 4.5.1.4.3 (representation of material properties), 4.5.1.4.4 (representation of geometrical quantities).

When assessing existing structures, reconsideration of the design values of the basic variables may be required. Guidance is given in subclauses 4.5.1.4.1 to 4.5.1.4.4, where relevant.

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$x_k = \mu \exp(- k \sigma - 0.5\sigma^2)$ for a lognormal random variable (4.5-11b)

where:

k = 1.64 is usually used

For loads and geo-mechanical properties, a similar procedure may be applied, but usually other distribution types will be more appropriate. For more information, reference is made to ISO 2394 "General principles on reliability for structures" and ISO 13822 "Basis for design of structures – Assessment of existing structures".

For practical classifications of the most common actions, see the relevant Appendices to ISO 2394 and Bulletin 191.

The seismic action is considered accidental action or as a variable one, depending on the geographic location of the structure(see for instance EN 1998). In general, the seismic action is considered as variable action wherever the available information is sufficient to quantify the representative values of the seismic action with a prescribed probability of not being exceeded during a reference period t_R. If there is not sufficient information to this end (for instance in regions of very low seismicity), the seismic action is considered as accidental.

Permanent actions, self weight included, although usually classified as fixed, may have to be considered as partially free where the effects are very sensitive to their variation in space, e.g. for static equilibrium and analogous verifications.

Soil reactions, e.g. soil pressure underneath foundation slabs or footings, are strongly influenced by soil-structure interaction. They should be determined by analysis, but the result should commonly be considered widely uncertain, especially the distribution in space.

4.5.1.4 Representative values of basic variables

4.5.1.4.1 Representation of actions

Actions should be classified as:

- direct or indirect,
- permanent, variable or accidental,
- static, quasi-static or dynamic,
- closely bounded or non-closely bounded,
- fixed or free.

Reactions, mainly on supports, should also be distinguished from directly imposed actions. Although they are taken into account like actions for some verifications, they are in reality effects of actions and may need specific reliability measures in design.

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Load arrangements are sometimes defined in the load standards. If several actions are free, the load cases (fixing the arrangements of all actions by taking into account their compatibility) are sometimes defined in the same documents. More information on load arrangements is given in section 4.2.3 of CEB Bulletin 191. See also EN 1992-1 for the load arrangements due to traffic actions.

When overloading has been observed in the past, it may be appropriate to increase representative values. When some loads have been reduced or removed completely, the representative values of the load magnitudes can be appropriately reduced and/or the partial factors can be adjusted. Guidelines are given in ISO 2394 "General principles on reliability for structures" and ISO 13822 "Basis for design of structures – Assessment of existing structures".

In the first two cases, G is considered as a mean value and should be calculated from nominal dimensions.

In the third case it is defined as G_{sup} or G_{inf}.

The difference between G_{sup} or G_{inf} and G_m should not exceed 0.1 G_m. For some types of prestressed structures this maximum acceptable difference may have to be reduced to 0.05 G_m.

This case is mainly applicable to finishes and equipment. G_{sup} and G_{inf} may normally be defined as corresponding to 0.95 and 0.05 fractiles plus (or minus) the expected variation in time of G_m.

For each free action, different load arrangements should be defined.

The representative values of actions to be applied in design of new structures are given below.

When assessing existing structures, the load characteristics should be introduced with values corresponding to the actual situation.

Representative values of permanent actions

Each permanent action is represented by a single representative value G if at least one of the following conditions is satisfied:

- the variability of the action in time and with regard to the design is small
- the influence of the action on the total effect of the actions is small
- it is evident that one of the two representative values (the upper or the lower) governs for all parts of the structure.

In the other cases, two representative values (upper and lower, G_{sup} and G_{inf}) should be defined, taking into account variations which can be foreseen.

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For structures to be designed for the most common variable actions these values are given in standards or codes associated with the same γ_F values as in this Model Code.

Ψ values depend on the model of the action, see ISO 2394.

An example of the choice of the coefficients ψ_i according to EN 1990 (Eurocode 0), "Basis of structural design", is given in Table 4.5-1.

Table 4.5-1 The coefficients ψ_i according to EN 1990

Action	ψ_0	ψ_1	ψ_2
Variable loads in buildings,			
Category A : domestic, residential	0.7	0.5	0.3
Category B : office areas	0.7	0.5	0.3
Category C : congregation areas	0.7	0.7	0.6
Category D : shopping areas	0.7	0.7	0.6
Category E : storage areas	1.0	0.9	0.8
Category F : traffic area, ≤ 30kN	0.7	0.7	0.6
Category G : traffic area, 30 –160 kN	0.7	0.5	0.3
Category H : roofs	0	0	0
Snow load: H ≤ 1000 m a.s.l.	0.6	0,2	0
Wind loads on buildings	0.6	0,2	0

These values are associated with the methods of verification defined in subclause 4.5.2.3.

Nominal numerical values of densities are given in subclause 5.1.3 for plain, reinforced and prestressed concrete, and in ISO 9194 for other materials. For future possible permanent equipment an upper value should be specified.

The representative values of the prestress are defined in subclause 4.5.1.4.2.

Representative values of variable actions

Each variable action may be represented, by

- characteristic value Q_k
- combination value $\Psi_0 Q_k$
- frequent value $\Psi_1 Q_k$
- quasi-permanent value $\Psi_2 Q_k$.

where

Ψ_0 coefficient for the combination value of a variable action, taking into account the reduced probability of simultaneous occurrence of the most unfavourable values of several independent actions.

Ψ_1 coefficient for the frequent value of a variable action, generally representing the value that is exceeded with a frequency of 0.05, or 300 times a year

Ψ_2 coefficient for the quasi-permanent value of a variable action, generally representing the value that is exceeded with a frequency of 0.50, or the average value over a period of time.

Besides, for some variable actions, specific representative values are defined for fatigue verifications.

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For structures to be designed these values are normally defined by the competent public authority or by the client and correspond to the values beyond which a high probability of integrity of the structure can no longer be assured.

For ordinary facilities appropriate multiple representative seismic actions are the following:

- for the serviceability limit states defined in subclause 3.3.1.1:
  - for the operational limit state: A "frequent" seismic action, expected to be exceeded at least once during the design service life of the structure (i.e. having a mean return period much shorter than the design service life).
  - for the immediate use limit state: An "occasional" earthquake, not expected to be exceeded during the design service life of the structure (e.g. with a mean return period about twice the design service life).
- for the two ultimate limit states defined in subclause 3.3.1.2:
  - for the life safety limit state: A "rare" seismic action, with a low probability of being exceeded (10%) during the design service life of the structure.
  - for the near collapse limit state: A "very rare" seismic action, with very low probability of being exceeded (2 to 5%) in the design service life of the structure.

For facilities whose consequences of failure are very high, the "very rare" seismic action may be appropriate for the life safety limit state. For those which are essential for the immediate post-earthquake period a "rare" seismic action may be appropriate for the immediate use limit state or even the operational limit state.

Representative values of accidental actions

Each accidental action can be given by a single representative value, which is usually the design value A_d.

Representative values seismic actions

A representative seismic action, with a prescribed probability of not being exceeded during a reference period t_R, is defined for each limit state considered.

Depending on the use and importance of the facility, competent authorities will chose how many and which limit states should be verified as a minimum and to which representative seismic action they will be paired off.

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It is not sufficient to define a representative seismic action by scaling standard spectral shapes to a single ground motion parameter, notably the effective or the peak ground acceleration. Instead, the seismic action should be defined in terms of its full spectrum, throughout the full range of structural periods of relevance.

Normally it is sufficient to consider only the two horizontal translational components of the ground motion.

For buildings or similar structures, in general the vertical component may be neglected, with the possible exception (depending on seismicity) of:

- horizontal members with significant concentrated masses along the span;
- long horizontal spans (e.g. over 20 m) or cantilevers (e.g. over 5 m);
- prestressed horizontal members.

In bridges, the vertical component should always be taken into account for the design of prestressed decks or bearings.

Simulated records are produced from mathematical models of the seismic source which dominates the seismic hazard, including the rupture event, the wave propagation through the bedrock to the site and through the subsoil to the ground surface.

Historic records should come from seismic events with magnitude, fault distance and mechanism of rupture at the source which are consistent to those dominating the seismic hazard for the representative seismic action in question. Their travel path and the subsoil conditions of the recording station should preferably resemble those applying at the site.

Artificial (or "synthetic") records, mathematically derived from the target elastic response spectrum, are not realistic if they are rich in all frequencies in

The basic definition of each representative seismic action is through its elastic response spectrum for a single-degree-of-freedom oscillator, as a function of viscous damping (the default being 5% of critical damping). The spectrum applies to the top of the ground under free-field conditions and should be specified taking into account the site's subsoil conditions and the local topography and geology, if relevant.

The elastic response spectrum is the same for the horizontal components of the ground motion, but should be specified separately for the vertical.

The components of the seismic action should be taken to act simultaneously.

Time-histories of the relevant components of the ground motion are needed for response-history analyses of the structure.

Preference should be given to historic or simulated records over artificial ones.

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the same way as the target spectrum. Therefore, perfect matching of the elastic response spectrum should be avoided.

The period range of interest may be taken to extend from twice to 20% of the fundamental period of the structure in the direction of the seismic action component in question.

For the estimation of peak response quantities, a minimum of seven such events is needed if the corresponding results of the analyses are averaged, or a minimum of three, if the most adverse peak response from the analyses is used. Many more seismic events than these minimum numbers are necessary for the estimation of residual deformations or displacements through nonlinear response-history analyses.

Generally, during prestressing, the external forces are imposed and the associated elongations of the tendons are controlled. The prestressing load is determined at the time of its application

Even where prestress has to be considered as an action, a prestrain $\varepsilon_p(x, t)$ has commonly also to be considered in some parts of the calculations especially in verifications with regard to the ULS. Where only immediate losses are considered $\varepsilon_p(x, t)$ is deduced from $P(x, t)$ by dividing it by the product E_pA_p. Where also long-term losses are considered, this simple division may have to be supplemented by a correction transforming the relaxation of the tendon into a variation of strain.

Length and angular deviation may be considered small if the ratio $\Delta P_m(x, t)/P(0, 0)$ is not, at any time t, greater than 0.30.

To conform with the basic definition of the representative seismic action, each individual component time-history should be scaled so that the values of its elastic response spectrum for the default damping are at least 90% of the specified spectrum throughout the period range of interest.

A sufficient number of independent seismic events (in terms of component time-histories) should be used for the derivation of meaningful and robust statistics of the action effects.

4.5.1.4.2 Representation of prestress

Prestressing forces are regarded as actions on the structure.

Representative values

Losses are numerically defined as mean values $\Delta P_m(x, t)$ in the subclauses 5.4.5 and 5.4.6 assuming that the structure is submitted to the quasi-permanent combination of actions defined in subclause 4.5.1.3.

For a given set of tendons, considered in the same calculation of losses, the mean value of the prestressing force is defined as:

$P_m(x, t) = P(0, 0) - \Delta P_m(x, t)$ (ΔP in absolute value)

Two characteristic values of the prestressing force are also defined.

In the cases where the length and angular deviation of the tendons are not exceptionally large, the following formulae, although conservative if the angular deviation is small, may be used as acceptable approximations.

(a) Bonded tendons

$P_{k\ \mathrm{sup}}(x, t) = 1.1\ P_m(x, t)$

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$P_{k\ \mathrm{inf}}(x, t) = 0.9\ P_m(x, t)$

(b) Unbonded tendons

$P_{k\ \mathrm{sup}}(x, t) = 1.05\ P_m(x, t)$

$P_{k\ \mathrm{inf}}(x, t) = 0.95\ P_m(x, t)$

The design values of forces in prestressing tendons are discussed in subclause 5.3.9.

4.5.1.4.3 Representation of material properties

Representative values

Strengths and other material properties to be considered as basic variables are represented by their characteristic values f_k (or X_k) or by their mean values.

When assessing existing structures the material properties shall be considered according to the actual state of the structure.

The significance of these values is shown in clause 6.3 of Bulletin 191. In exceptional cases, where an increase of the strength results in a decrease in reliability, upper characteristic values and specific γ_m values (smaller than 1) should be used.

When the original design documents are available and no serious deterioration, design errors or construction errors are observed or suspected, the characteristic value in accordance with the original design should be used. If appropriate, destructive or non-destructive inspections should be performed and evaluated using statistical methods. For more information, reference is made to ISO 2394 “General principles on reliability for structures” and ISO 13822 “Basis for design of structures – Assessment of existing structures”.

Mean and characteristic values for strength properties of concrete and steel are given in subclause 7.2.3.

Where strengths and other material properties are not considered basic variables in limit state equations, they may be represented by mean values f_m (or X_m) which usually are the most likely values of f, and not by other fractiles taken out of the same statistical populations as f_k values. However, these may generally be substituted by characteristic values f_k , as an acceptable approximation for such verifications.

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When the original design documents are available and no change in dimensions has taken place or other evidence of deviations is present, the nominal dimensions in accordance with the original design documents should be used in the analysis. These dimensions shall be verified by inspection to an adequate extend. For more information, reference is made to ISO 2394 "General principles on reliability for structures" and ISO 13822 "Basis for design of structures – Assessment of existing structures".

In this clause, only geometrical quantities representing the structure are considered. For most of the quantities, their deviations within the specified tolerances should be considered as statistically covered by γ_{Sd} and γ_{Rd} , i.e. by γ_F and γ_M factors. Only those quantities, which might in some verification be one of the main variables, should, in those verifications only, be taken as basic.

The depths of reinforcement in thin members are taken into account by modifying their nominal values by additive reliability margins.

Dimensions in slabs larger than intended may significantly increase the self weight, whereas smaller dimensions and/or lever arms of steel bars may significantly reduce the resistance. Similarly, smaller than nominal values of concrete cover may endanger the durability or the anchorage resistance of steel bars. An unintended inclination of columns may disproportionately increase their action effects.

Because of the complicated nature of the related phenomena, no explicit figure of general validity can be given on the amount of such performance reduction, however, it is considerably less than 4%.

4.5.1.4.4 Representation of geometrical quantities

Representative values

The representative values of geometrical quantities to be applied in design of new structures are defined below.

When assessing existing structures the dimensions of the structural elements shall be taken according to the actual state of the structure.

Unintentional eccentricities, inclinations and parameters defining curvatures affecting columns and walls and the depth of reinforcement in members thinner than 100 mm, are unique geometrical quantities defined in this Model Code to be taken into account as basic variables, if not specified otherwise. The other geometrical quantities are as specified in tile drawings of the design.

The basic geometrical variables are directly fixed as design values in the chapters where the relevant limit states are treated.

Tolerances

The possible deviations in the geometry of the concrete elements, of the cover, or of the position of steel, shall not alter significantly either the SLS or the ULS performance of the relevant elements.

As a general rule for these geometrical basic variables, the corresponding specified tolerances may be taken equal to their design values of the deviations divided by 1.2 and should be controlled accordingly.

For the other geometrical variables, the values of the materials partial safety factors included in this Model Code, are meant to cover small reductions of performance (resistances, mainly) which may result from their deviations.

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In the absence of a more justified set of tolerances, the following limitations may apply:

(a) *Table 4.5-1: Tolerances for concrete sectional dimensions, according to ISO 22966 (for Tolerance Class 1 and 2)*

Elements and dimension (mm)	**Permitted deviation** Δ (mm) Class 1	Class 2
Beams slabs and columns		
$a < 150$ mm	± 10 mm	± 5 mm
$a = 400$ mm	± 15 mm	± 10 mm
$a \geq 2500$ mm	± 30 mm	± 30 mm

with linear interpolation for intermediate values

(b) *Table 4.5-2: Tolerances for the location of ordinary and prestressing reinforcement, according to ISO 22966 (for Tolerance Class 1 and 2)*

Height of cross-section h (mm)	**Permitted deviation** Δ (mm) Class 1	Class 2
Ordinary reinforcement		
$h \leq 150$ mm	+ 10 mm	+ 5 mm
$h = 400$ mm	+ 15 mm	+ 10 mm
$h \geq 2500$ mm	+ 20 mm	+ 20 mm

with linear interpolation for intermediate values

Prestressing reinforcement	
$h \leq 200$ mm	± 0.03 h
$h > 200$ mm	the smaller of ± 0.03 h or ± 30 mm

Depending on the quality assurance scheme applicable, relevant tolerance values should be respected for each category of possible deviations under well specified conditions of measurements and evaluations. Possible foreseen higher deviations should lead to additional design steps taking into account all the consequences of deviations that exceed the specified tolerances.

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(c) Tolerance of cover: $c_{nom} - c_{act} < 10$ mm.

(d) *Table 4.5-3: Tolerances of unintentional deviations of columns, walls, beams and slabs according to ISO 22966 (for Tolerance Class 1)*

Elements and type of deviation	**Permitted deviation** Δ (mm)
Columns; walls	
- inclination of a column or wall at any level in a single- or multi-storey building	the larger of $h/300$ or 15 mm where h is free height
- deviation between centre	the larger of $t/30$ or 15 mm but not more than 30 mm where $t = (t_1 + t_2)/2$
- lateral deviation of a column between adjacent storey levels	the larger of $h/300$ or 15 mm but not more than 30 mm where h is free height
- location of a column or a wall at any storey level, from a vertical line through its intended centre at base level in a multi-storey structure	the smaller of 50 mm or $\Sigma h/(200\ n^{1/2})$, where h is free height and n is the number of storeys and $n>1$
Beams and slabs	
- location of a beam-to-column connection measured relative to the column	the larger of $\pm\ b/30$ or $\pm$ 20 mm, where b is dimension of column in the same direction as Δ
- position of bearing axis of support when structural bearings are used	the larger of $\pm\ l/20$ or $\pm$ 15 mm where l is intended distance from edge

The tolerance values apply to the structure, after compaction and hardening of the concrete.

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The basic design rules differ according to the limit state under consideration.

In some cases, defined in other chapters, some limit state calculations may be substituted by detailing rules or special provisions.

These numerical values are considered to be appropriate in the design of new structures for the socioeconomic conditions in most European countries. In some countries where different conditions prevail (and possibly depending on the type of building or civil engineering works), γ factors for design may be reduced.

The $\gamma_{G\,\mathrm{sup}}$ and γ_Q values given in subclause 4.5.2.2 may be reduced in the following cases:

- design of one-storey buildings (ground floor plus roof) with spans not exceeding 9 m, that are only occasionally occupied (storage buildings, sheds, green-houses, small silos and buildings for agricultural purposes),
- floors resting directly on the ground,
- light partition walls,
- lintels,
- sheeting,
- ordinary lighting masts.

4.5.2 Basic rules for partial factor approach

4.5.2.1 General

In design by the partial factor method it should be proven that the structure, given the design values for the basic variables, does not reach the relevant limit states for loads below the design load. The basic design rules given in this section are applicable to the limit states as defined in chapter 3.

In principle, all relevant limit states should be considered, as well as all relevant design situations, load arrangements and load cases and combinations of actions.

The numerical values of γ factors given in subclause 4.5.2 are applicable to the design of new structures and commonly also to existing structures.

In subclause 4.5.1.3 explanations are given with regard to updating the design values of the variables. After the evaluation of the updated design values, one may check the structural reliability of existing structures using the standard procedures for new structures.

The numerical values of γ factors given in subclauses 4.5.2 are applicable to the design of buildings and civil engineering works not subject to variable actions having an exceptional variability.

In the design of new structures the $\gamma_{G\,\mathrm{sup}}$ and γ_Q values given in subclause 4.5.2.2 may be reduced respectively to 1.2 and 1.35 for reliability differentiation, provided that these reductions are not associated with a reduced quality assurance level.

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Some γ_M factors may however have to be increased in cases where quality measures, considered normal in the actual case, would not be expected, but this is intended to maintain the reliability degree, not to modify it.

The general content of γ_F factors is defined in section 6.2.2 of CEB Bulletin 191.

An example of particular actions is that of some hydraulic actions (see CEB Bulletin 201).

If the basic set of γ factors given in this clause is adopted, any increase of the reliability degree is normally limited to the consideration of supplementary hazards or higher values of accidental actions, and more refined analyses.

4.5.2.2 Ultimate limit states

Design principle

It should be verified that the following condition is satisfied :

$S_d < R_d$ if a one-component action-effect is to be considered,

$S_d < R_d^*$ if a multi-component action-effect is to be considered,

where

S_d denotes a design action-effect,

R_d denotes a design resistance (and R_d^* a design resistance domain).

Partial factors and ways to introduce them into the calculations

(a) γ_F factors

(a1) γ_F factors for persistent and transient situations

The numerical values applicable to non-particular actions for the limit state of static equilibrium are given in the following tables and clauses.

Table 4.5-4: Partial γ_F factors for the limit state of static equilibrium

Actions, γ_F.	**Unfavourable effect** (γ_{sup})	**Favourable effect** (γ_{inf})
Permanent (G), γ_G	1,05 - 1.1	0.9 – 0.95
Prestress (P), γ_P	1.1	1.0
Leading variable action ($Q_{k,1}$), γ_Q	1.5	Usually neglected
Accompanying variable action ($Q_{k,i}$), γ_Q	1.5 $\Psi_{0,i}$	Usually neglected

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Basic values given in Table 4.5-5 are in some cases conservative for the design of new structures. Reference is made to subclause 3.3.3.1 and subclause 4.5.2 for reliability differentiation.

The basic numerical values applicable to the ultimate limit state in case of non-particular actions not involving geotechnical actions are given in the following table and clauses.

Table 4.5-5: Partial γ_F factors for the design of structural members not involving geotechnical actions: basic values

Actions, γ_F	**Unfavourable effect** (γ_{sup})	**Favourable effect** (γ_{inf})
Permanent (G), γ_G	1.35	1.0
Prestress (P), γ_P	1.1	1.0
Leading variable action ($Q_{k,1}$), γ_Q	1.5	Usually neglected
Accompanying variable action ($Q_{k,i}$), γ_Q	1.5 $\Psi_{0,i}$	Usually neglected

In the most common cases one of γ_G ($\gamma_{G,sup}$ or $\gamma_{G,inf}$) may be applied globally to all permanent actions (unfavourable or not), prestress excepted. The other cases should be identified by judgement.

Alternatively, a more refined approach can be taken in the design of structural members not involving geotechnical actions: the less favourable of the combination of the partial γ_F factors given in the following table (SET1 or SET2) can be used.

Table 4.5-6: Partial γ_F factors for the design of structural members not involving geotechnical actions: alternative combination of values

Actions, γ_F	**Unfavourable effect** (γ_{sup})	**Favourable effect** (γ_{inf})
SET1		
Permanent (G), γ_G	1.35	1.0
Prestress (P), γ_P	1.1	1.0
Leading variable action ($Q_{k,1}$), γ_Q	1.5 $\Psi_{0,1}$	Usually neglected
Accompanying variable action ($Q_{k,i}$), γ_Q	1.5 $\Psi_{0,i}$	Usually neglected

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SET2		
Permanent (G), γ_G	0.85 – 1.35	1.0
Prestress (P), γ_P	1.1	1.0
Leading variable action ($Q_{k,1}$), γ_Q	1.5	Usually neglected
Accompanying variable action ($Q_{k,i}$), γ_Q	1.5 $\Psi_{0,i}$	Usually neglected

(a2) γ_F factors for accidental or seismic situations

The values of γ_F applicable to all actions are equal to 1.

(b) γ_M factors

The numerical values of γ_M to be used for calculating R_d are given in Table 4.5-7.

Table 4.5-7: Partial factors γ_M

Basic variable	**Design situation**	
	Persistent/transient	**Accidental**
Concrete		
Compressive strength (f_{cck}), γ_c	1.5	1.2
Tensile strength (f_{ctk}), γ_{ct}	*	*
Reinforcing and prestressing steel		
Tensile strength (f_{stk}), γ_{st}	1.15	1.0
Compressive strength (f_{sck}), γ_{sc}	1.15	1.0

* See relevant clauses

Safety is normally ensured by the design values of the action or of the other parameters describing the accidental or seismic situation.

The general content of γ_M factors is defined in subsection 6.3.2 of Bulletin 191.

As a simplification a conversion factor η is included in γ_c.

The values of γ_c and γ_s, given in Table 4.5-7 should be increased if the geometrical tolerances given in subclause 4.5.1.4.4 are not fulfilled. Conversely they might be reduced by 0.1 and 0.05 respectively, at the maximum, if these tolerances are reduced by 50% and are strictly controlled (e.g. precast concrete components and structures).

A variation of γ_c or γ_s, according to the degree of control of f_{ck} (without making the control tests more severe), does not seem to be justified, because the variation of the control can more rationally be taken into account by the compliance criteria included in the control itself. In any case, it cannot be numerically fixed independently of the control criteria. Besides, even if a better quality, characterized by a lower coefficient of variation of the strength, is ensured for a given characteristic strength, this would not justify reducing the γ_M-values, because this would imply also a lower mean strength. In some cases (for example in case of very good quality management e.g. for precast concrete) the conversion factor η included in γ_c, may be reduced.

The γ_M factors applicable to other basic variables are given in the relevant clauses.

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Strengths may intervene in S_d via stiffness and the spatial distribution throughout the structure. They may generally be favourable as well as unfavourable and are not to be considered as basic variables.

These rules shall be amended for accidental situations (see the clause regarding general rules for combinations of actions in the sequel) and if possible simplifications or refinements regarding combinations of actions are applied, see Eq. (4.5-17).

Eq. (4.5-13) is the more general. Particular cases are mainly those where

- S_d is an under-proportional function of the actions (or the principal of them); in these cases Eq. (4.5-12) may be unsafe; or
- the effects of some actions have a sense opposite to the effects of the other actions and are of the same order of magnitude; in these cases Eq. (4.5-12) may be too conservative (this may be the case for the isostatic effects of prestress).

This rule (not splitting γ_M into γ_M and γ_{Rd}) is not applicable in design by testing.

For the definition of individual actions, reference is made to subsections 1.2.1 and 6.2.1 of *fib* Bulletin 191.

For the Ψ factors, reference is made to the clause regarding representative values of variable actions in subclause 4.5.1.4.1.

Whenever strengths intervene in the value of the action-effect S_d the associated γ_M values should be taken equal to 1. This rule is not applicable to buckling verifications, in which strengths are important favourable basic variables.

(c) Introduction of the partial coefficients into the calculations

In most cases γ_F factors should be applied globally as follows

$$S_d = S\left\{\gamma_G G + \gamma_p P + \gamma_Q\left(Q_{1k} + \sum_{i>1}\Psi_{oi}Q_{ik}\right)\right\} \qquad (4.5\text{-}12)$$

In particular cases, defined in the relevant clauses of other chapters or to be identified by judgement, for persistent or transient situations, this formula may be substituted by

$$S_d = \gamma_{Sd} S\left\{\gamma_g G + \gamma_p P + \gamma_q\left(Q_{1k} + \sum_{i>1}\Psi_{oi}Q_{ik}\right)\right\} \qquad (4.5\text{-}13)$$

where the partial factors should be taken by referring to the preceding clause (*a1*).

These two formulae are partially symbolic and should be applied by following in detail the combination rules given in the sequel.

The use of a sum of permanent actions $\gamma_{G,i}G_{k,i}$ instead of a single permanent load G is allowed.

γ_M factors should generally be applied globally.

<u>*Combinations of actions*</u>

(a) General rules

The combinations of design values to be taken into account for applying Eqs. (4.5-12) and (4.5-13) are as follows, in symbolic presentation:

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– fundamental combinations applicable for persistent and transient situations

$$S_d = \gamma_{G\sup} G_{\sup} + \gamma_{G\inf} G_{\inf} + \gamma_P P + \gamma_{Q,1} Q_{k,1} + \sum_{i>1} \gamma_{Q,i} \Psi_{0,i} Q_{k,i} \quad (4.5\text{-}14)$$

– accidental combinations, applicable for accidental situations

$$S_d = G_{\sup} + G_{\inf} + P + (A_d \ or \ 0) + (\Psi_{1,1} \ or \ \Psi_{2,1}) Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i} \quad (4.5\text{-}15)$$

– seismic combinations, applicable for seismic situations

$$S_d = G_{\sup} + G_{\inf} + P + A_{Ed} + \sum_{i\geq 1} \Psi_{2,i} Q_{k,i} \quad (4.5\text{-}16)$$

In these combinations:

– $G_{\sup}$ and $G_{\inf}$ refer to the unfavourable and favourable parts of the permanent actions, respectively;

– P refers to prestressing;

– $Q_{k,i}$ refers to any variable action, in succession;

– A_d denotes the unique accidental action associated with the accidental situation, if this situation is due to this action. If it is due to another event or to a past action, A_d is substituted by 0.

– A_{Ed} denotes the design seismic action having a prescribed probability of not being exceeded during the reference period t_d and associated with the ultimate limit state of interest in this specific seismic situation.

The actions to be included in any combination are only those that are mutually compatible or are considered as such, as an acceptable approximation. Non-simultaneous actions should be considered in the same combination if their effects are simultaneous.

(b) Possible simplifications

As an approximation to be recognized by judgement, it is frequently sufficient to limit the total number of variable actions to a maximum of three in any fundamental combination and to two in any accidental combination.

Ψ factors take account of the reduced probability of simultaneous occurrence of actions. The choice between $\Psi_{1,1}Q_{k,1}$ or $\Psi_{2,1}Q_{k,1}$ depends on the type of accidental design situation e.g. impact, fire or survival after an accidental event or situation.

In seismic situations masses are consistent with the gravity loads corresponding to the combination $G_{\sup} + G_{\inf} + \sum_{i\geq 1} \Psi_{2,i} Q_{k,i}$

Prestressing P should be added, if relevant.

In most cases some variable actions, which obviously are not the leading ones for a given verification, need not be considered as $Q_{k,1}$.

For fire situations, apart from the temperature effect on the material properties, A_d should represent the design value of the indirect thermal action due to fire.

In general, there will be two different levels of A_{Ed}, one for each ultimate limit state introduced in subclause 3.3.1.2.

The cases of incompatibility or negligible compatibility are very numerous. They are given in the codes or standards on actions or identified by judgement (e.g. snow and maximum climatic temperature).

Other simplifications may be envisaged and discussed, for example by giving directly design combinations for a given set of common variable actions, such as some imposed loads, wind, snow and temperature.

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Judgement is necessary because the concept of one action is very blurred. For example the actions of wind, snow, water and imposed loads should be considered as different actions, but the imposed loads on different floors should be considered as one action.

This simplification is mainly intended for common buildings. The influence of this simplification on the resulting reliability should be carefully analysed.

Attention is drawn to the risk that an accident results in consequences on variable actions; for example many persons may gather in some places in order to escape during or immediately after an accident.

This may be the case, for example, if a failure should be limited to a small part of the structure.

This introduces one more combination. Attention is drawn to the necessity, in this case, to verify more completely and carefully than usual the serviceability limit states, which may be less covered than usually by ultimate limit state verifications.

In many cases this does not result in important changes of design.

Fundamental combinations that are obviously identified as non-critical may be omitted in the calculations.

In many cases Ψ_{oi} factors may be merged with γ_Q and S_d may then be calculated, for persistent and transient situations, by

$$S_d = S\left(\gamma_G G + \gamma_Q \sum_1^n Q_{ik} \right) \qquad (4.5\text{-}17)$$

where

γ_G = 1 or 1.35 (take the more unfavourable)

γ_Q = 1.5 for n = 1, or 1.35 for $n \geq 2$ (take the more unfavourable).

In accidental combinations $\Psi_{1,1}$ may often be substituted by the lower value $\Psi_{2,1}$, for most, or all, variable actions, as a judged approximation or because the occurrence of a greater value during the accidental situation is judged to be very unlikely.

(c) Possible refinements

In cases where the most likely consequences of a failure do not seem to be exceptionally severe, the following reductions of γ_F factors in fundamental combinations are possible.

– reduce $\gamma_{G\,\mathrm{sup}}$ to 1.2 or, alternatively, $Q_{k,1}$ to $\Psi_{01}Q_{k,1}$, or

– reduce to 1.2 the γ_Q value applicable to $\Psi_{oi}Q_{k,i}$ ($i > 1$).

4.5.2.3 Fatigue verification

Design principles

Fatigue design shall ensure that in any fatigue endangered cross-section the expected damage D will not exceed a limiting damage D_{lim}. The verifications of this requirement can be performed according to four methods with increasing refinement.

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This is an indirect verification that the loss of strength will not be significant. The representative values of P should be chose such that the unfavourable situation is covered.

In assessing the stress range, stress variations in opposite senses (due for example to successive arrangements of a moveable load) shall be, if relevant, taken into account.

Other design properties associated with tensile stress of concrete (e.g. a formal shear stress) may also have to be considered.

If the stress analysis is sufficiently accurate or conservative, and this fact is verified by in-situ observations, it may be possible to take $\gamma_{Ed} = 1{,}0$.

In Eq. 4.5-24 the term between the brackets is the static part and the term Q_{fat} is the dynamic part.

For Q_{fat} in many cases the frequent value $\Psi_{1,1}Q_k$ may be used as an equivalent or conservative approach.

(a) Verification by simplified procedure

It is verified whether the following stresses or stress ranges are not exceeded:

- the maximum design stress range in the steel $\gamma_{Ed}\,\Delta\sigma_s(G, P, \Psi_1 Q_k)$
- the maximum concrete compressive stress $\gamma_{Ed}\,\sigma_{c,max}(G, P, \Psi_1 Q_k)$
- the maximum design tensile stress in plain concrete

$$\gamma_{Ed}\,\sigma_{ct,max}(G, P, \Psi_1 Q_k)$$

The load factor γ_{Ed} is assumed to be 1,1.

(b) Verification by means of a single load level

This verification refers to a representation of the variable load dominant for fatigue by a single magnitude Q associated with a number of repetitions n during the required lifetime.

The stresses in the structural materials, or the stress range, are calculated under the following combination of actions:

$$(G_{\sup} + G_{\inf} + P + \Psi_{1,1}\,Q_{k,1} + \sum_{i>1} \Psi_{2,i} Q_{k,i}) + Q_{fat} \qquad (4.5\text{-}18)$$

where

Q_{fat} is to the relevant fatigue load (e.g. traffic load or other cyclic load).

The stresses found under the load according to Eq. 4.5-18 is multiplied by $\gamma_{Ed} = 1{,}1$ or $1{,}0$ if accurate stress analysis is possible. At the resistance side the strength of the materials is divided by $\gamma_{s,fat} = 1{,}15$ for the steel and $\gamma_{c,fat} = 1{,}5$ for the concrete.

(c) Verification by means of a load spectrum

According to this method, the load history during the required life should be represented by a spectrum in a discretized form. The accumulation of fatigue damage is calculated on the basis of the Palmgren-Miner summation.

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Some of these rules may in some cases be substituted by stress limitations, detailing rules or other indirect verifications.

The α-factor (e.g. 0.6 for excessive compression) describes the limit state and is not a reliability factor.

In such equations f generally is not to be considered as a basic variable.

This rule may in some cases be substituted by a maximum slenderness ratio.

If not fixed by the Code, C_d should be fixed by the contract or chosen by the designer, possibly depending on non-structural parts.

See subclause 7.6.6.

4.5.2.4 Verification of structures subjected to impact and explosion

Impact and explosions are regarded as accidental loads, Eq. 4.5-15 applies.

4.5.2.5 Serviceability limit states

Design principle

(a) Limit state of cracking and excessive compression

It should be verified that in any cross-section

$\sigma(F_d) < \alpha f_d$ for crack formation and excessive creep effects

$w(F_d, f) < w_{lim}$ for maximum crack width

$\sigma(F_d) \leq 0$ for crack re-opening

where:

σ is a defined stress

f_d is a tensile, shear or compressive design strength

w is a defined crack width.

(b) Limit state of deformations

It should be verified that:

$$a(F_d, f_d) \leq C_d \qquad (4.5\text{-}19)$$

where

a is a defined deformation (generally a deflection or a rotation at a member end).

(c) Limitation of vibrations

In the most common cases the limitation is ensured by indirect measures, such as limiting the deformations or the periods of vibration of the structure in order to avoid the risk of resonance. In the other cases a dynamic analysis is necessary.

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Pragmatic values smaller than 1 may be envisaged for indirect actions.

In general, there will be two different levels of A_{Ek}, one for each serviceability limit state introduced in subclause 3.3.1.1.

In the seismic situations masses are consistent with the gravity loads corresponding to the combination $G_{\sup} + G_{\inf} + \sum_{i\geq 1} \Psi_{2,i} Q_{k,i}$.

Values of partial factors

(a) γ_F factors are taken equal to 1.

(b) γ_M factors are taken equal to 1.

Combinations of actions

(a) General rules

The combinations which should be considered depend on the particular limit state under consideration and are identified in the corresponding chapters

They are defined as follows, in a symbolic presentation

characteristic: $$G + P + Q_{k,1} + \sum_{i>1} (\Psi Q_{k,i}) \quad (4.5\text{-}20)$$

frequent: $$G + P + \Psi_{1,1} Q_{k,1} + \sum_{i>1} (\Psi_{2,i} Q_{k,i}) \quad (4.5\text{-}21)$$

quasi-permanent: $$G + P + \sum_{i\geq 1} (\Psi_{2,i} Q_{k,i}) \quad (4.5\text{-}22)$$

seismic: $$G + P + A_{Ek} + \sum_{i\geq 1} (\Psi_{2,i} Q_{k,i}) \quad (4.5\text{-}23)$$

where

G is taken according to clause 4.5.1.4.1,

$Q_{k,i}$ refers to any variable action, successively

A_{Ek} is the representative seismic action prescribed for the serviceability limit state of interest.

(b) Possible simplification

The first two paragraphs of clause 4.5.2.2 regarding possible simplifications for combination of actions may be applied to combinations for serviceability limit states.

In common cases for reinforced concrete structures, the characteristic

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combinations may be simplified by avoiding reference to various Ψ_{oi} factors. They are substituted, in a symbolic presentation, by

$$G + Q_{k,1} \tag{4.5-24}$$

or

$$G + 0.9\sum_{1}^{n} Q_{k,i} \quad \text{(take the more unfavourable)} \tag{4.5-25}$$

in which $Q_{k,i}$ is the most unfavourable variable action.

4.6 Global resistance format

4.6.1 General

The global resistance format treats the uncertainties of the structural behaviour as described by the limit state condition according to Eq. 4.3-4 on the level of structural resistance. The effects of various uncertainties (of material properties, geometrical dimensions, etc.) are integrated in a global design resistance and can be also expressed by a global safety factor. The representative values of the global resistance variables and the global safety factors must be chosen in such a way that the reliability requirements for the design of new structures, which are expressed in subclause 3.2.5 in terms of reliability index β related to the reference period, are met.

The global resistance approach was initiated by the introduction of non-linear analysis, which is based on a global structural model and offers tools for the safety assessment. It is a general approach, which follows the probabilistic safety concept more rationally than the to partial factor method. It is applicable to the safety check on structural level. However, it can be applied also to members or sections as well.

The global safety factor reflects the variability of the structural response due to random properties of basic variables. The effect of random variation of basic variables, such as strength f on resistance R is dependent on the type of limit state function $r(f,..)$. The limit state function is represented by non-linear numerical analysis. Thus, in case of dominating concrete failure the resistance variability is much greater then in case of steel failure. This also means, that the variability of resistance is in general not constant for given set of material parameters and their random variations and depends on a structural model.

4.6.2 Basic rules for global resistance approach

4.6.2.1 Representative variables

The representative variable for the global resistance is the structural resistance R.

The global resistance has a general meaning and usually describes the response of an element or a structure to given load actions. The resistance can be described by a scalar, vector or a function, depending on design and limit

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state formulation. A significant feature of the structural resistance is the integration of various random effects of material properties, dimensions, etc., and their interactions. Unlike in the partial factor design method, the uncertainties are evaluated on a global structural level and not in local material points.

The meaning of global resistance can be illustrated by an example of a simple beam under the action of a force. The global resistance is expressed by the ultimate force, which can be resisted by the beam. This resistance covers all material properties, geometry, reinforcement, boundary conditions and modes of failure. Typically, the beam can fail in bending or in shear and both of these failure modes are described by the same variable – maximum force resisted by the beam. The same calculation model, for example a finite element analysis, is used and the failure mode is detected automatically in the analysis.

The uncertainty of resistance R is described by its random distribution function with its parameters: function type, mean, standard deviation, (and possibly others). The parameters of scatter for a given random distribution of resistance can be used to derive the mean, characteristic and design values of resistance R_m, R_k, R_d. The global safety can be expressed either by a global safety factor of by a reliability index.

In contrast, if the same beam is verified by the partial safety factor method, a specific section is considered and local checks are made for specific actions in a cross section. Two separate verifications are performed in the section, one for the bending failure and another one for shear failure. The global safety is not evaluated, but it is guaranteed by the formulation of partial safety factors.

In many cases it is possible to estimate the mean and the characteristic values of resistance by the values of resistance derived from mean and characteristic values of the basic variables, respectively. When the mean value obtained in this way differs from the mean value obtained by other means (e.g. experiments) special care is advised.

The uncertainty of resistance is expressed by the following values of resistance:

R_m - mean value of resistance,

R_k - characteristic value of resistance, (corresponding to a probability of failure 5%),

R_d - design value of resistance.

The basic variables defined for the partial factors in subclause 4.5.2.1, are used for calculating the resistance values. The values of these variables (f, a,…) must be chosen in accordance with the safety formats described further in this chapter. The value of action F is considered in the same way as in the partial factor method.

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It is important to recognize, that in the present formulation the global safety factor γ_R^* is related to the mean variable. To distinguish this from the partial safety factors, which are referring to characteristic values the notation with asterisk superscript is used.

Furthermore, it is useful to introduce a scaling factor for a loading pattern. In general, action F_d and resistance R_d, which appear in design Eq. (4.6-2), may include many components (for example vertical and horizontal forces, body forces, temperature, etc,) and can be described by a point in a multi-dimensional space. The resistance scaling factor k_R describes the relation between resistance and action and has the same meaning as a safety factor. In a symbolic form, considering a pair of corresponding components it can be defined as:

$$k_R = \frac{R}{F_d} \tag{4.6-4}$$

Then, the design condition in Eq. 4.6-2 can be rewritten as:

$$k_R \geq \gamma_R^* \tag{4.6-5}$$

Where γ^*_R is a required global safety factor for resistance. In this, if relevant, the global safety factor can include the model uncertainty. The factor k_R can be used to calculate the relative safety margin m_R for resistance:

$$m_R = k_R - \gamma_R^* \tag{4.6-7}$$

The model uncertainty factor γ_{Rd} should be chosen based on the knowledge of the design conditions of the structure during its life time. The value $\gamma_{Rd} = 1.0$ should be used only in exceptional cases, when an evidence of the model validation in design conditions is available. An example of such a condition is the case of assessment of an existing structure.

The value γ_{Rd} = 1.06 should be used for models based on a refined numerical analysis, such as non-linear finite element analysis. The model should be objective (low mesh sensitivity) and validated. The factor 1.06

4.6.2.2 Design condition

The design condition derived from Eq. (4.3-4) for the global format takes the following form:

$$s(F_{d,\ldots}) \leq r(R_{d,..}) \tag{4.6-1}$$

In a simplified force representation it can take the form:

$$F_d \leq R_d \tag{4.6-2}$$

The design and mean values of resistance are related as

$$R_d = R_m / \gamma_R^* \tag{4.6-3}$$

where γ_R^* is the global safety factor for mean resistance.

The global safety factor γ^*_R accounts for random uncertainties of model parameters, namely of material properties. An uncertainty due to model formulation, shall be treated by a separate safety factor for model uncertainty γ_{Rd}. This can be applied either to the action, or to the resistance. In the later case the design resistance takes the form:

$$R_d = \frac{R_m}{\gamma_R^* \gamma_{Rd}} \tag{4.6-6}$$

The value of the model uncertainty factor depends on the quality of formulation of the resistance model. The recommended values are

$\gamma_{Rd} = 1.0$ for no uncertainties

$\gamma_{Rd} = 1.06$ for models with low uncertainties

$\gamma_{Rd} = 1.1$ for models with high uncertainties.

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does not cover the errors due to approximations in the numerical model. It covers the other effects not included in the numerical model, such as time effects, environmental effects, etc. An example of such a case is the usual design according to the partial safety factor method.

The value $\gamma_{Rd} = 1.1$ should be used for models sufficiently validated as in the case above, but with a higher uncertainty of structural conditions due to an unknown design situation. An example of such a case is a design under uncertain load history due to actions imposed by environmental effects.

The deemed-to-satisfy approach is applicable both for the traditional structural design and for the design associated to durability. The method may comprise sets of predetermined alternatives given in a standard. In most operational standards the design associated with durability is based on the deemed-to-satisfy approach.

Traditionally, durability related deemed-to-satisfy provisions include requirements to the workmanship, concrete composition, possible air entrainment, cover thickness to the reinforcement, crack width limitations and curing of the concrete. However, other provisions may also be relevant.

Examples of the calibration of deemed-to-satisfy criteria based on a probabilistic safety format and data derived from 10 – 15 years old structures are given in *fib* Bulletin 34, "Model code for service life design".

4.7 Deemed-to-satisfy approach

4.7.1 General

The deemed-to-satisfy approach is a set of rules for

- dimensioning,
- material and product selection and
- execution procedures

that ensures that the target reliability for not passing the relevant limit state during the design service life is not exceeded when the concrete structure or component is exposed to the design situations.

The specific requirements for design, materials selection and execution for the deemed-to-satisfy approach shall be determined in either of two ways:

- on the basis of statistical evaluation of experimental data and field observations according to requirements of clause 4.4 regarding the probabilistic safety format;
- on the basis of calibration to a long term experience of building tradition.

The limitations to the validity of the provisions, e.g. the range of cement types covered by the calibration, shall be clearly stated.

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Durability related exposure categories in the design situations may be classified in exposure classes. For more information on classification of environmental actions as exposure classes reference is made to ISO 22965-1, "Concrete – Part 1: Methods of specifying and guidance for the specifier".

In Table 4.7-2 a classification of exposure classes according to ISO 22965-1 is given. The same classification is adopted by the European CEN standards on the design of concrete structures.

Table 4.7-2: Exposure classes related to environmental conditions for concrete with reinforcement or embedded metal according to ISO 22965-1

Class designation	Environmental conditions and examples
No risk of corrosion or attack	
X0	exposure to very dry environment e.g.: components inside buildings with very low air humidity and no risk of corrosion or attack
Corrosion induced by carbonation	
XC1	exposure to dry or permanently wet environment e.g.: interior of buildings with low air humidity, components permanently submerged in water e.g.: surfaces exposed to airborne chlorides
XC2	exposure to wet or rarely dry environment e.g.: surfaces subject to long term water contact, foundations e.g.: swimming pools, components exposed to industrial waters containing chlorides
XC3	exposure to moderate humid or cyclic wet and dry environment e.g.: components inside buildings with moderate or high air humidity, exterior of buildings sheltered from rain
Corrosion induced by chlorides other than from sea-water	
XD1	exposure to moderate humid environment and chlorides from sources other than from sea water (e.g. chlorides from de-icing agents) e.g.: surfaces exposed to airborne chlorides

4.7.2 Durability related exposure categories

In the absence of a more specific study, the durability related exposure categories related to environmental conditions may be classified for concrete with reinforcement or embedded metal as given in Table 4.7-1.

Table 4.7-1: Durability related exposure categories related to environmental conditions for concrete with reinforcement or embedded metal

Exposure categories	Environmental conditions
No risk of corrosion or attack	Exposure to very dry environment
Corrosion induced by carbonation	Exposure to air and moisture
Corrosion induced by chlorides other than from sea-water	Exposure to moisture and chlorides from sources other than from sea water (e.g. chlorides from de-icing agents)
Corrosion induced by chlorides from sea-water	Exposure to sea-water
Freezing and thawing attack	Exposure to moisture and freeze/thaw cycles
Chemical attack	Exposure to aggressive chemical environment e.g. components exposed to aggressive chemical environment (gas, liquid or solid) or aggressive industrial atmosphere

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XD2	exposure to wet or rarely dry environment and chlorides from sources other than from sea water (e.g. chlorides from de-icing agents)
XD3	exposure to cyclic wet and dry environment and chlorides from sources other than from sea water (e.g. chlorides from de-icing agents) e.g.: pavements, car park slabs, components exposed to spray containing chlorides.
Corrosion induced by chlorides from sea-water	
XS1	exposure to exposure to airborne salt but not in direct contact with sea water e.g.: surfaces near to or an the coast
XS2	exposure to permanent saturation in seawater e.g.: components of marine structures permanently submerged in seawater.
XS3	exposure to sea-water in tidal, splash and spray zones e.g.: components of marine structures
Freezing and thawing attack	
XF1	exposure to freeze/thaw cycles and moderate water saturation without de-icing agent e.g.: vertical surfaces exposed to rain and freezing
XF2	exposure to freeze/thaw cycles moderate water saturation in combination with de-icing agent e.g.: vertical surfaces of road structures exposed to freezing and airborne de-icing agents
XF3	exposure to freeze/thaw cycles and high water saturation without de-icing agent e.g.: horizontal surfaces exposed to rain and freezing
XF4	exposure freeze/thaw cycles high water saturation in combination with de-icing agent e.g.: road and bridge decks exposed to de-icing agents; surfaces exposed to direct spray containing de-icing agents and freezing; splash zone of marine structures exposed to freezing
Chemical attack	
XA1	exposure to slightly aggressive chemical environment
XA2	exposure to moderate aggressive chemical environment
XA3	exposure to highly aggressive chemical environment

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The assumed effectiveness of the actual concept shall be documented, for instance for products by complying with relevant minimum requirements in product standards.

4.8 Design by avoidance

Traditional structural design involving the avoidance method includes a concept based on avoiding the detrimental effect, e.g. sheltering the structure from certain loads like environmental loads, wind, wave loads impact by vehicles of missiles, etc.

In design for durability the avoidance-of-deterioration method implies that the deterioration process will not occur due to for instance:

- separation of the environmental action from the structure or component, e.g. by cladding or membranes;
- using non-reactive materials, e.g. certain stainless steels or alkali-non-reactive aggregates;
- separation of reactants, e.g. keeping the structure or component below a critical degree of moisture;
- suppressing the harmful reaction, e.g. by electrochemical methods.

In seismic design seismic isolation may be introduced at certain horizontal levels:

- between the superstructure of buildings or similar structures and the foundation or the ground;
- between a bridge deck and the top of the piers; or
- between sensitive equipment, containers of hazardous materials, important artefacts, etc., and the supporting structure or foundation.

The specific requirements for design, materials selection and execution for the avoidance-of-deterioration method can in principle be determined in the same way as for the deemed-to-satisfy approach.

The limitations to the validity of the provisions shall be clearly stated.

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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.

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 subclause 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.

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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.

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.

In some countries intermediate concrete grades are also often used:

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

The information given in subclauses 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.

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

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 three categories of oven-dry density:

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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.

The values given in Table 5.1-2 are valid for plain and reinforced light-weight 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.

– Lightweight aggregate concrete (800 – 2000 kg/m³)
– Normal weight concrete (> 2000 – 2600 kg/m³)
– Heavy weight concrete (> 2600 kg/m³)

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/m³]

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 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/m³]		801 - 1000	1001 - 1200	1201 - 1400	1401 - 1600	1601 - 1800	1801 - 2000
Density [kg/m³]	**Plain concrete**	1050	1250	1450	1650	1850	2050
	Rein-forced concrete	1150	1350	1550	1750	1950	2150

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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

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 subclause 5.1.2.

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:

$$f_{cm} = f_{ck} + \Delta f \qquad (5.1\text{-}1)$$

$$f_{lcm} = f_{lck} + \Delta f \qquad (5.1\text{-}2)$$

where:

Δf = 8 MPa

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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 subclause 5.1.5.1 below.

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

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

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 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}:

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

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

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:

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

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

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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.

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.

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):

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

where:

$f_{ct,sp}$ is the mean value of splitting tensile strength determined according 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:

$$f_{ctm} = A_{fl} \cdot f_{ct,fl} \qquad (5.1\text{-}8)$$

where:

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

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

h_b is beam depth [mm]

α_{fl} = 0.06

5.1.5.2 Fracture energy

The fracture 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):

$$G_F = 73 \cdot f_{cm}^{0.18} \qquad (5.1\text{-}9)$$

where:

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

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 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).

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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.

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:

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

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

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

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

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

$$G_{F,l} = G_{FoA} + 16 \cdot f_{lctm} \qquad (5.1\text{-}10)$$

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

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:

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

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

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

where

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

$$\cos 3\theta = \frac{3\sqrt{3}}{2}\cdot\frac{J_3}{J_2^{3/2}} \qquad (5.1\text{-}13)$$

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.

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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 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.

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

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

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

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

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

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

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

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.

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

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

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

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

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.

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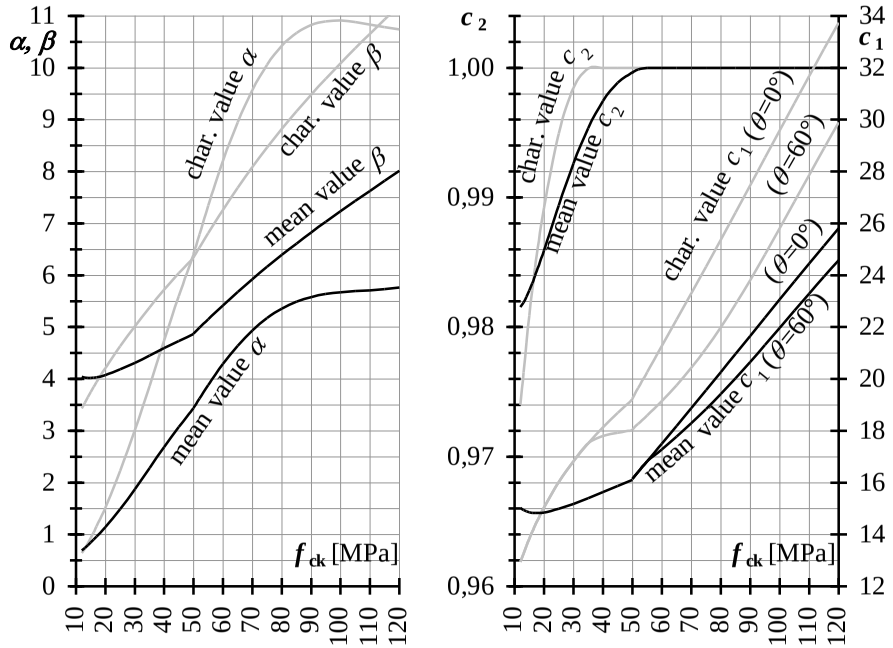


Figure 5.1-1: Coefficients for Eq.(5.1-11), normal weight concrete

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

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

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):

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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 compression and uniaxial tension according to subclauses 5.1.8.1 and 5.1.8.2, respectively, as well as for an estimate of creep acc. to Eq. (5.1-60), subclause 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

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

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 subclause 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):

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

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

$$E_{lci} = \eta_E \cdot E_{ci} \qquad (5.1\text{-}22)$$

where:

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

ρ is the oven-dry density of the lightweight aggregate concrete 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

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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 than 3 % and may be neglected.

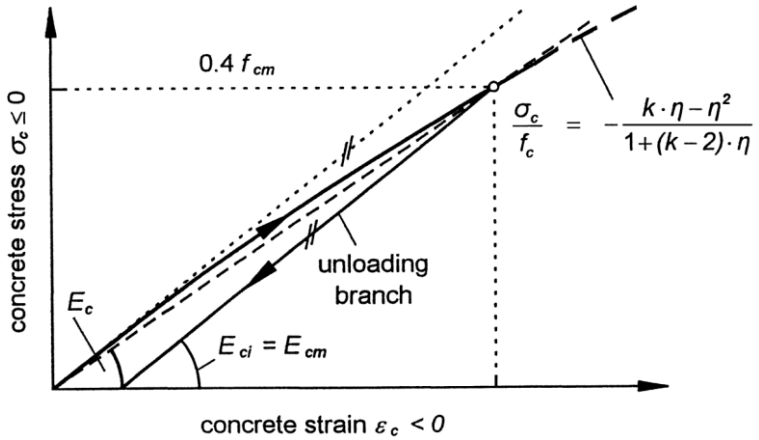


Figure 5.1-2: Definition of different moduli of elasticity (according to fib Bulletin 42)

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 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.

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

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):

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

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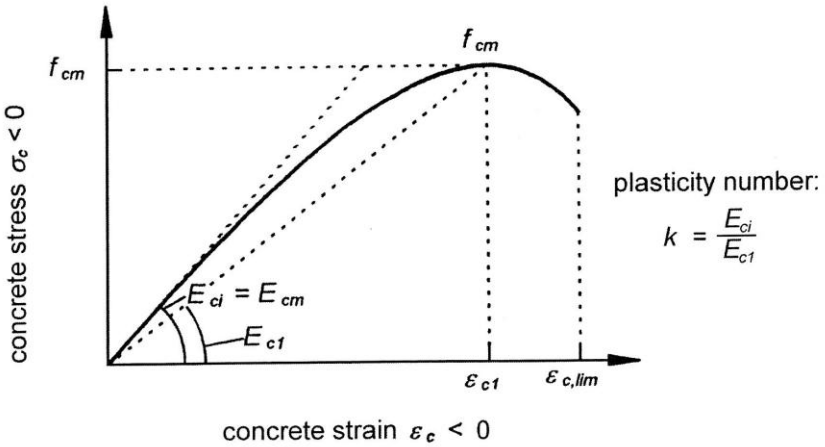


Figure 5.1-3: Schematic representation of the stress-strain relation for uniaxial compression (according to fib Bulletin 42)

5.1.7.3 Poisson's ratio

For a range of stresses $-0.6 \cdot f_{ck} < \sigma_c < 0.8 \cdot f_{ctk}$ 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 $\nu_c = 0.20$ meets the required accuracy.

The value of $\nu_c = 0.20$ is also applicable for lightweight aggregate concrete.

5.1.8 Stress-strain relations for short-term loading

5.1.8.1 Compression

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

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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. 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.

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 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

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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.

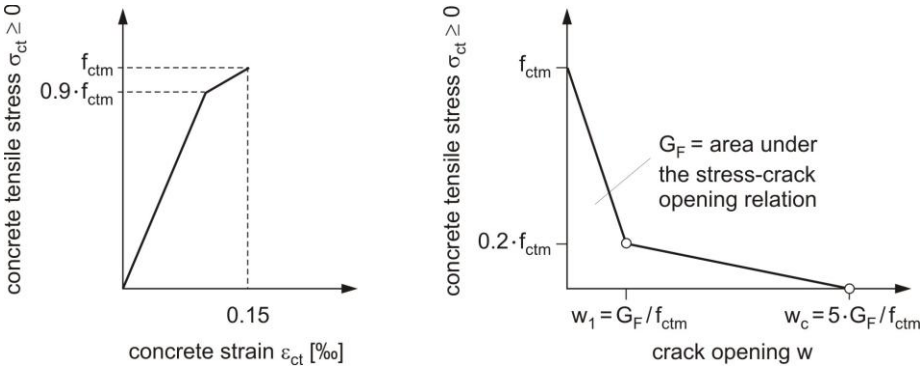


Figure 5.1-4: Schematic representation of the stress-strain and stress-crack opening relation for uniaxial tension (according to fib Bulletin 42)

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

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

$$\Delta\sigma_c = E_{ci} \cdot \Delta\varepsilon_c \qquad (5.1\text{-}28)$$

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):

$$\sigma_{ct} = E_{ci} \cdot \varepsilon_{ct} \quad for \quad \sigma_{ct} \le 0.9 \cdot f_{ctm} \qquad (5.1\text{-}29)$$

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

where:

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

ε_{ct} is the tensile strain

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:

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

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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 subclause 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 plasticity 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.

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

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 isotropic 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):

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

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:

$$\dot{\boldsymbol{\varepsilon}}_p = \lambda \frac{\partial g}{\partial \boldsymbol{\sigma}} \tag{5.1-34}$$

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:

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- 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
- 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.

$$\lambda \geq 0, \qquad f\lambda = 0, \qquad f \leq 0 \tag{5.1-35}$$

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):

$$\dot{f} = \frac{\partial f}{\partial \boldsymbol{\sigma}} \cdot \dot{\boldsymbol{\sigma}} + \frac{\partial f}{\partial \boldsymbol{\alpha}} \cdot \dot{\boldsymbol{\alpha}} = 0 \tag{5.1-36}$$

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

$$\dot{\boldsymbol{\alpha}} = \lambda \mathbf{h}(\boldsymbol{\sigma}, \boldsymbol{\alpha}) \tag{5.1-37}$$

result in an incremental constitutive law in case of loading:

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

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):

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

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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 for example 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 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.

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

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

result in an incremental constitutive law in case of loading:

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

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.

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

For an anisotropic formulation with a higher order tensorial damage variable refer for example 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.

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

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

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

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

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

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.

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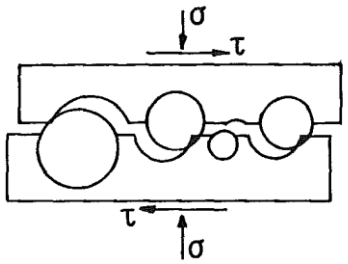


Figure 5.1-5: Principle of shear friction in concrete crack, with unbroken aggregates

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:

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

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

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 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.

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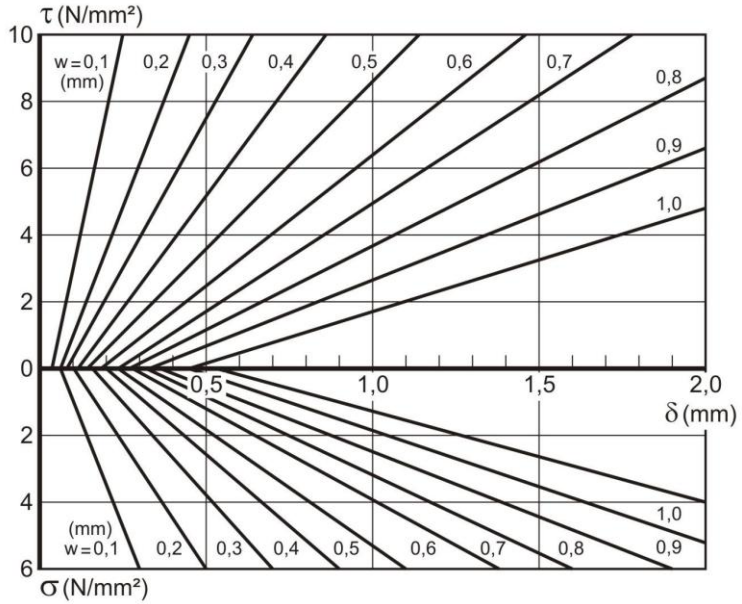


Figure 5.1-6: Relations (Eqs. 5.1-48/49) for f_c = 30 MPa, according to fib Bulletin 42

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.

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

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

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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 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.

where

$f_{cm}(t)$ 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:

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

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

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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 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.

5.1.9.2 Strength under sustained loads

5.1.9.2.1 Sustained compressive strength

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 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:

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

with

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

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 $(t–t_0) > 0.015$ 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]

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

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 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.

5.1.9.2.2 Sustained tensile strength

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

$$f_{ctk,sus} = \alpha \cdot f_{ctk} \tag{5.1-55}$$

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

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

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

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.

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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 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:

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

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

where:

$\varepsilon_{ci}(t_0)$ is the initial strain at loading

$\varepsilon_{cc}(t)$ 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: $\varepsilon_{cn}(t) = \varepsilon_{cs}(t) + \varepsilon_{cT}(t)$

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

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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 subclause 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 $0.4 \cdot f_{cm}(t_0)$.

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.

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.

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

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

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]

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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.

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 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.

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

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

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

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:

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

(b) Creep coefficient

The creep coefficient may be calculated from:

$$\varphi(t,t_0) = \varphi_0\beta_c(t,t_0) \qquad (5.1\text{-}63)$$

where:

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)

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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.

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

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]

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

The development of creep with time is described by:

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

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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 $(63/f_{cm})^{0.9}$.

with

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

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]

$$\alpha_3 = \left[\frac{35}{f_{cm}}\right]^{0.5} \qquad (5.1\text{-}71)$$

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

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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):

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

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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 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.

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

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

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:

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

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

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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 true also for ordinary concrete in case the concrete structures react sensibly 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".

where

$\varphi_{0,k}$ is the nonlinear notional creep coefficient, which replaces φ_0 in Eq. (5.1-63)

k_σ $= |\sigma_c| / f_{cm}(t_0)$ 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):

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

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

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

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

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

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 $\beta_{as}(t)$:

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

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

where

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

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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

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

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

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

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

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:

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

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

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

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

where

α_{ds1}, α_{ds2} are coefficients, dependent on the type of cement (see Table 5.1-13)

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For normal weight high strength concrete ($60 \le f_{cm} \le 130$ 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

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 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]

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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 structures.

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.

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

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

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

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 approximately -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

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):

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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 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

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:

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

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

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

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

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

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):

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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.

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

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

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

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

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 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]

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]

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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.

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.

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

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

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

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):

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

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

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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 effect of temperature prior to loading may be taken into account using 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):

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

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

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with

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

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):

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

with

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

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):

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

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 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:

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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.

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.

$\varepsilon_{cas0}(f_{cm})$ 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):

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

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):

$$\beta_{RH,T} = \beta_{RH} \cdot \beta_{sT} \qquad (5.1\text{-}101)$$

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

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

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

$$RH_T = 99 \cdot \beta_{s1} + \beta_{s1,T} \le 100\ \% \qquad (5.1\text{-}104)$$

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

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

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).

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-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)

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.

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”.

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.

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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

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

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

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

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

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

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

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

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

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 CEB Bulletin 188, "Fatigue of concrete structures" (1988) and *fib* Bulletin 42, "Constitutive modelling of high strength / high performance concrete" (2008).

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.

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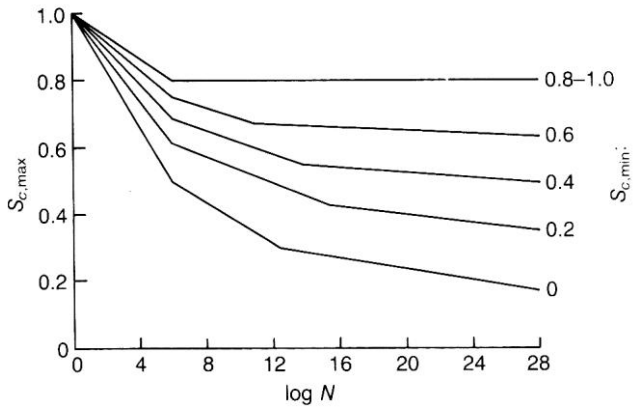


Figure 5.1-7: S-N relations according to Eqs. 5.1-107 to 5.1-109

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 %.

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 lightweight aggregates the crack propagates in the cement paste and through the aggregates due to the relatively higher strength of the cement paste. Thus,

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:

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

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

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

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

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

with

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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.

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.

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

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 subclause 5.1.9.1, Eq. (5.1-51)

$\beta_{c,sus}(t,t_0)$ is a coefficient which takes into account the effect of high mean stresses during loading. For fatigue loading it may be taken as 0.85

(IV) Spectrum of load-levels

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

$$D = \sum_i \frac{n_{Si}}{n_{Ri}} \qquad (5.1\text{-}113)$$

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.

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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 $(t - t_0) = (1/1440)\cdot(n/f)$ = 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.

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.

5.1.11.1.2 Fatigue strains

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

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

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]

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 subclauses 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 < $|\dot{\sigma}_c|$ < 10^7 MPa/s and $30\cdot10^{-6}$ s^{-1} < $|\dot{\varepsilon}_c|$ < $3\cdot10^2$ s^{-1}, respectively. In the correspondent equations all strain and stress values have to be used as absolute values.

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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:

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

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

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

and

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

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

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:

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

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

with $\dot{\varepsilon}_{ct0} = 1 \cdot 10^{-6}$

and

$$f_{ct,imp,k} / f_{ctm} = \left(\dot{\sigma}_{ct} / \dot{\sigma}_{ct0}\right)^{0.018} \quad \text{for } \dot{\sigma}_{ct} \le 0.3 \cdot 10^6 \text{ MPa s}^{-1} \qquad \text{(5.1-118a)}$$

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$$f_{ct,imp,k} / f_{ctm} = 0.0062\left(\dot{\sigma}_{ct} / \dot{\sigma}_{ct0}\right)^{1/3} \quad \text{for } \dot{\sigma}_{ct} > 0.3 \cdot 10^{6} \text{ MPa s}^{-1} \qquad (5.1\text{-}118b)$$

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):

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

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

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

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

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where

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

ε_{c1} is the strain a maximum load for static loading from subclauses 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 subclause 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).

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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.

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 micro-cracks 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"), 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

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

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]

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):

$$K_w = K_{w0} \cdot \frac{1}{f_{cm}^{\ 6}} \qquad (5.1\text{-}122)$$

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penetration of water under pressure" and converted into a coefficient of water permeability, but it has to be considered as an approximate value only.

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

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

$$V = \bar{K}_g \frac{A}{l} \frac{p_1 - p_2}{p} t \qquad (5.1\text{-}124)$$

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 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

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

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

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]

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):

$$K_g = K_{g0} \cdot \frac{1}{f_{cm}^{4.5}} \qquad (5.1\text{-}125)$$

where

K_g is the coefficient of gas permeability in [m²]

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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):

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

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.

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]

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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):

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

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

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

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):

$$d_c^2 = 2D_{CO2}\frac{C_a}{C_c}t \qquad (5.1\text{-}134)$$

where

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))

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):

$$D_1 = \frac{D_{1,o}}{f_{cm} - 8} \qquad (5.1\text{-}133)$$

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.

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

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

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]

The diffusion coefficient for carbon dioxide D_{CO2} through carbonated concrete may be estimated from Eq. (5.1-136):

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

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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 subclause 5.1.13.2.

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 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

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

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):

$$D_{Cl^-} = D_{Cl^-0} \cdot \frac{1}{f_{cm}^{1.5}} \qquad (5.1\text{-}137)$$

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):

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

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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: "Determination of the capillary absorption of water of hardened concrete", Materials and Structures, Vol. 32, pp. 178-179, 1999.

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]

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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.

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 [-]
$R_{NAC,0}^{-1}$ inverse effective carbonation resistance of concrete in [(mm²/years)/(kg/m³)]

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”.

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 subclause 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

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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. de-icing 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.

$$R_{NAC,0}^{-1} = k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t \qquad (5.1\text{-}142)$$

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):

$$C(x,t) = \left(C_0 + \left(C_{s,\Delta x} - C_0 \right) \cdot \left[1 - erf\left(\frac{x - \Delta x}{2 \cdot \sqrt{D_{app,C} \cdot t}} \right) \right] \right). \qquad (5.1\text{-}143)$$

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]

$$D_{app,C}(t) = k_e \cdot D_{RCM,0} \cdot k_t \cdot A(t) \qquad (5.1\text{-}144)$$

with

$D_{RCM,0}$ chloride migration coefficient in [m²/s]

k_e environmental variable [-]

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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”.

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 de-icing 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 de-icing 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 de-icing 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.

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 [-]

5.1.13.4 Freeze-thaw and freeze-thaw de-icing 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 de-icing 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 de-icing agent are important.

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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

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:

(b) Models

Models to be included here, though being rather crude, are still under discussion

5.1.13.5 Alkali-aggregate reaction

(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

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:

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a) mineral acids

b) buffer solutions including organic acids, carbonic acid or ammonium salts

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.

So far no prediction formula for the constant k_p may be given. This constant should be determined by appropriate experiments.

$$d = k_c \sqrt{ct} \tag{5.1-146}$$

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]

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 $0.5 < p \leq 1.0$

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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 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).

5.1.13.7 Leaching progress

The cumulative leaching of a substance from a given concrete surface area in constant contact with water is given empirically by:

$$E = k_1(1-e^{-k_2 t}) + k_3\sqrt{t} + k_4 t \qquad (5.1\text{-}150)$$

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

$$k_3 = 2Ac_{mo,0}\sqrt{\frac{D_{eff}}{\pi}} \text{ in [mol/s}^{0.5}\text{]} \qquad (5.1\text{-}152)$$

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.

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Bars can be either directly produced in straight lengths or by straightening reinforcing steel in coils.

Surface characteristics include the characteristics of the ribs or indentations by means of which bond with the concrete is achieved and the characteristics of the coating when applicable.

Bond aspects are covered by six Interface Characteristics.

Durability mainly covers the corrosion protection properties of "special" reinforcing steel such as (either metallic and/or organic) coated steel or corrosion resistant steel. Behaviour at extreme temperatures may include either the behaviour at low temperature (e.g. for cryogenic applications) or high temperature (e.g. behaviour in case of fire).

Examples of relevant international product standards are ISO 6935 Parts 1 to 3.

Types of reinforcement not covered by product standards may be used after it has been shown that they meet the specified minimum requirements.

Mechanical devices for splicing are specified in 7.13.2.6.

5.2 Reinforcing steel

5.2.1 General

Products used as reinforcing steel may be:

- bars;
- welded fabric.

Reinforcing steel is characterized by:

- geometrical properties:
  - size;
  - surface characteristics.
- mechanical properties:
  - tensile strength and yield strength;
  - ductility;
  - fatigue behaviour;
  - behaviour under extreme thermal conditions.
- technological properties:
  - bond characteristics;
  - bendability;
  - weldability;
  - thermal expansion;
  - durability;
  - behaviour at extreme temperature.

Reinforcing steels shall comply with national or international product standards that specify their geometrical, mechanical and technological properties.

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The properties of reinforcing steels are generally confirmed by certification programs and certificates of compliance. The methods of testing and certification of conformity are as defined in relevant International or European Standards or RILEM Recommendations.

Cutting and bending of steel reinforcement, welding and mechanical splicing may be controlled via a standard for execution of concrete structures such as EN 13670 or ISO 22966.

Inappropriate welding procedures can adversely affect tensile or other properties of reinforcing steels, see subclause 8.3.4.

The simultaneous use of steels of various types on the same site is allowed only on condition that no confusion between the types is possible during the construction.

It should be possible to distinguish clearly between:

- plain bars of various grades and/or of various ductility classes
- high bond bars of various grades and/or of various ductility classes
- reinforcement that is weldable and that which is not.

The nominal diameter is a conventionally fixed value, e.g. in product standards which serves as a basis for the calculation of the nominal cross-sectional area taken as the area of a circle with a diameter equal to the nominal diameter.

5.2.2 Quality control

The fabrication of reinforcing steel shall be subject to a factory production control by the manufacturer, and a continuous external control by an independent qualified body, including certification and regular audits.

5.2.3 Designation

The designation of reinforcing steels normally includes:

- the relevant product standard;
- the nominal diameter or size;
- the steel grade related to the characteristic yield strength, the ductility properties and the weldability.

Each product shall be clearly identifiable with respect to this designation.

5.2.4 Geometrical properties

5.2.4.1 Size

The size of reinforcing steel is defined by a nominal diameter (for bars and coils) and a set of nominal diameters (welded fabric).

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The actual cross-sectional area is determined by weighing a given length of bar, assuming a steel density of 7850 kg/m^3.

For welded fabric the following applies:

- twin bars are allowed in one direction only
- adequate stiffness of the welded fabric should be ensured either by a limitation of the maximum spacing of the bars, or by introducing a minimum ratio between the diameter of the transverse bars and the diameter of the longitudinal bars.

Plain smooth wires (cold drawn wires) should not be used for reinforced concrete, except as non-structural reinforcement (spacers etc.) or in the form of welded fabric.

The rib parameters may be specified by either the relative rib area f_R, or by a combination of rib spacing, rib height and rib inclination of the transverse ribs, or by both criteria.

The indentation parameters may be specified by either the relative indentation area f_P, or by a combination of indentation spacing, indentation depth and inclination of the indentations, or by both criteria.

Poor straightening of ribbed or indented bars and wires from coils can significantly reduce the relative rib or indentation area and thus the bond properties of the straightened bars or wires.

The most common coatings applied to reinforcing steel are either metallic (e.g. zinc or zinc alloy) or organic (e.g. epoxy).

Examples of relevant international product standards for coated reinforcing steel are ISO 14645 and ISO 14657.

The difference between actual and nominal cross-sectional area shall not exceed the limiting values specified in relevant product standards.

5.2.4.2 Surface characteristics

The surface of reinforcing steel may be:

- ribbed;
- plain;
- indented.

Ribbed bars or wires and some indented products are considered as high bond reinforcements if they satisfy the conditions and requirements imposed by the relevant product standards.

Bars not satisfying these requirements should be treated as plain bars with respect to bond. For indented products, which cannot be considered as high bond reinforcements, reference should be made to relevant standards or technical documents.

The characteristics of the coatings are in general:

- finish and appearance;
- adherence;
- mass of the coating deposited per unit area;
- continuity.

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These characteristics shall conform to the requirements specified in the relevant product standards.

5.2.5 Mechanical properties

The mechanical properties are defined on the basis of standard tests.

5.2.5.1 Tensile properties

The characteristic values of

- the tensile strength (f_t)
- the yield strength (f_y)
- the percentage total elongation at maximum force (ε_u)

are respectively denoted f_{tk}, f_{yk} and ε_{uk}.

The tensile properties of steel increase with speed or rate of strain application.

For quality control purposes and design calculations, the mechanical properties of a product are referred to the nominal cross-sectional area.

The standard tests are defined in relevant International and European Standards and RILEM Recommendations.

The requirements apply to the product in the condition in which it is delivered. In the case of coils (wire or rods), the requirements apply to the material after straightening.

The value of f_{yk} should correspond to a 0.2% offset in the characteristic σ - ε diagram.

For steels totally or partially cold-worked by means of axial tension, it will generally be the case that

$f_{yc} \neq f_{yt}$

where f_{yc} and f_{yt} are actual yield strengths, for compression and tension respectively. The value of f_{yc} to be used in calculation should therefore be stipulated in relevant standards.

There are a number of situations where the designer expects reinforcement to yield before failure of an element or structure. If in fact the reinforcement is appreciably stronger than assumed in design, it is possible that, in a flexural situation, the concrete could reach its ultimate compression strain before the yielding of the tensile reinforcement. This could lead to a brittle, rather than ductile failure. It is therefore important to ensure that the actual yield strength of the reinforcement is not significantly higher than that assumed in design. A value of between 5 and 10% may be appropriate.

See also CEB Bulletin 242 “Ductility of reinforced concrete structures”.

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5.2.5.2 Steel grades

The steel grade denotes the value of the specified characteristic yield strength in MPa, and possibly, the specified ductility and the weldability properties.

5.2.5.3 Stress-strain diagram

Due to the diversity and evolution of the manufacturing processes for bars and wires, various stress-strain diagrams may be encountered.

Indicative stress-strain diagrams of reinforcing steel in tension are represented in Figure 5.2-1.

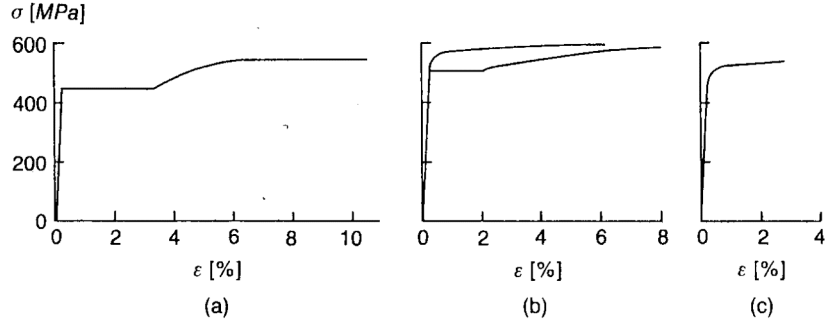


Figure 5.2-1 Stress-strain relationships of reinforcing steel: (a) hot-rolled bars; heat-treated bars; micro-alloyed bars; (b) low-carbon, heat-treated bars; cold-worked bars; (c) cold-worked wires

5.2.5.4 Ductility

Adequate ductility is necessary whether or not moment redistribution is taken into account in the calculations.

The characteristic value of the ratio f_t/f_k corresponds to the 5% fractile of the relation between actual tensile strength and actual yield stress.

Ductility Class definitions A, B, C and D are only valid for steel grades with a characteristic yield strength ≤ 600 MPa.

Four ductility classes are defined for design purposes.

These classes are defined by minimum specified values for the characteristic value of the ratio f_t/f_y and the characteristic percentage total elongation at maximum force ε_{uk} as follows.

Class A: $(f_t/f_y)_k \geq 1.05$ and $\varepsilon_{uk} \geq 2.5\%$

Class B: $(f_t/f_y)_k \geq 1.08$ and $\varepsilon_{uk} \geq 5\%$

Class C: $(f_t/f_y)_k \geq 1.15$ and ≤ 1.35 and $\varepsilon_{uk} \geq 7\%$.

Class D: $(f_t/f_y)_k \geq 1.25$ and ≤ 1.45 and $\varepsilon_{uk} \geq 8\%$.

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Classes C and D should be used where high ductility of the structure is required (e.g. in seismic regions). In seismic design an additional requirement for $f_{y,act}/f_{yk}$ for Classes C and D (e.g. $f_{y,act}/f_{yk} \leq 1.3$) can be introduced.

Should it be required to quantify a level of ductility in relation to the deformation capacity of a concrete member, since it is misleading to focus on isolated physical characteristics of the steel. In practice, this may be done by giving consideration to the 'equivalent steel concept', where the overall steel ductility parameter, 'p', may be regarded as being equivalent to:

for cold worked steel:

$$\rho = \varepsilon_{\mu}^{0.75} \cdot \left[\frac{f_t}{f_y} - 1 \right]^{0.8} \qquad (5.2\text{-}1)$$

for hot rolled steel:

$$\rho = \left(\left(\varepsilon_{\mu} - \varepsilon_{sh} \right) + 3 \cdot \varepsilon_{sh} \right)^{0.75} \cdot \left[\frac{f_t}{f_y} - 1 \right]^{0.8} \qquad (5.2\text{-}2)$$

For more information see CEB Bulletins 218 and 242.

Fatigue behaviour depends on factors such as bar size, rib geometry, bending of bars and welded connections, thus making it is difficult to give generalized S-N curves. More information can be found in CEB Bulletin d'Information N 188 "Fatigue of Concrete Structures".

Bending of steel at temperatures below -5°C is permitted only if allowed by the project specification and additional precautions are documented (ref: EN 13670, clause 6.3 or ISO 22966).

5.2.5.5 Shear of welded joints in welded fabric

Where welded joints are taken into account for calculation of the anchorage, each welded joint shall be capable of withstanding a shear force not less than 0.3 $A_s f_{yk}$, where A_s denotes the nominal cross-sectional area of the anchored wire.

5.2.5.6 Fatigue behaviour

The fatigue behaviour of reinforcing steel is described in Table 6.7.1.

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Detailed information may be found in FIP Special Report SR 88/2, June 1988. Information on the behaviour at very low temperatures of ribbed steels may be found in Beton- und Stahlbetonbau 10/1985, pp. 271-274 and 293-299.

Reinforcing bars should not be bent to a radius less than that used in the relevant bend and/or rebend test specified in the product standards.

A factory production control system which complies with ISO 9001 and addresses the requirements of the relevant product standard is considered as appropriate. Processing may include cutting, cutting and bending, mechanical splicing, and welding.

5.2.5.7 Behaviour under extreme thermal conditions

The above properties are valid only for "normal" temperatures between – 40 °C and + 40 °C.

Tensile strength and yield strength decrease and percentage total elongation at maximum force increases if the temperature increases. For typical data, see 5.2.9.

Tensile strength, yield strength and modulus of elasticity increase if the temperature decreases. The percentage total elongation at maximum force increases initially but decreases rapidly below a temperature of about -150°C

5.2.6 Technological properties

5.2.6.1 Bendability

The requirements concerning the bendability are specified in relevant standards.

5.2.6.2 Weldability

The requirements concerning the weldability are specified in relevant standards.

Depending on the type of reinforcement used, the methods for welding may be restricted.

5.2.6.3 Coefficient of thermal expansion

Within the temperature range from –20 °C to 180 °C the coefficient of thermal expansion of steel may be taken as 10×10^{-6}/°C.

5.2.6.4 Provisions for quality control

Reinforcing steel should be manufactured and processed under an appropriate permanent system of factory production control, which should include evaluation of the specified properties.

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Some specific effects might have to be considered in design and / or detailing for special types of steels: e.g. possible sensitivity to fretting fatigue of some stainless steels, see TR 51.

Other special types of steels which are not confirmed such as low-carbon chromium steel bars, bars with stainless steel cladding or similar new developments are available.

Combination of non-coated and stainless steel does not cause problems, reference to TR 51.

Detailed information on these three special types of steels may be found in *fib* Bulletin 49.

The carbon dioxide equivalent may be used for declaration.

Maximum diameter of reinforcing steels may be limited for certain design checks.

The evaluation of conformity of reinforcing steels should be based on the verification of their properties by batch sampling and testing as specified in the relevant product standards and may include determination of long term quality levels.

The conformity of reinforcing steel both as manufactured and subsequently processed may be attested by certification programs operated by third party certification bodies.

5.2.7 Special types of steels

The following confirmed special types of steels that show enhanced corrosion protection properties can be used:

- galvanised steels;
- epoxy coated steels;
- stainless steels;

subject to possible application provisions.

5.2.8 Sustainability aspects

Manufacturers of steels should declare the quantity of greenhouse gases emitted during the fabrication and processing of steels.

5.2.9 Assumptions used for design

The parameters of reinforcing steel to be used for design are as follows:

- modulus of elasticity, E_S;
- characteristic yield strength, f_{yk} (or 0.2% proof strength, $f_{0.2k}$);
- ductility parameters, i.e. characteristic percentage total elongation at maximum force, ε_{uk} and characteristic ratio tensile strength/yield strength, $(f_t/f_y)_k$.

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The actual diagram for a particular steel may be used if it is duly verified by the producer. See also commentary relating to subclause 5.2.5.1 on over-strength of reinforcement.

For high strength steels, the σ - ε diagram is non-symmetrical in compression and in tension.

Some cold-worked steels have a lower modulus of elasticity in compression than in tension. The difference is not important in practice.

As a simplification, actual stress-strain diagrams can in calculations be replaced by an idealized characteristic diagram according to Figure 5.2-2, assuming a modulus of elasticity E_s equal to 200 GPa.

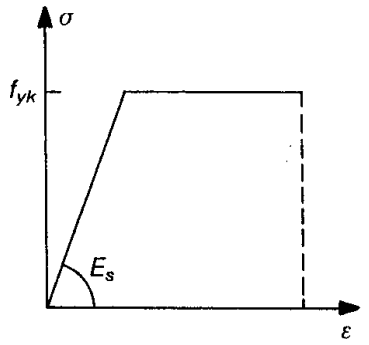


Figure 5.2-2: Idealized stress-strain diagram

The main parameters of reinforcing steel to be used for fire design are as follows:

- modulus of elasticity at temperature θ, $E_{s,\theta}$;
- proportional limit at temperature θ, $f_{sp,\theta}$;
- maximum stress at temperature θ, $f_{sy,\theta}$.

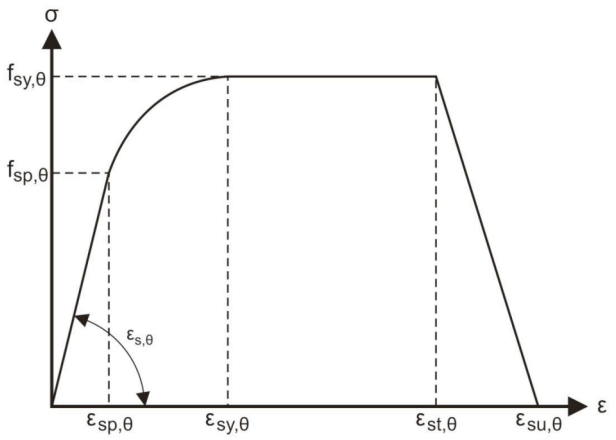
As a simplification, the idealized characteristic diagram according to Figure 5.2-3 can be used for fire design.

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The values for the parameters $E_{s,\theta}$, $f_{sp,\theta}$ and $f_{sy,\theta}$ given in Table 5.2-1 may be used.

Table 5.2-1: Values for the parameters $E_{s,\theta}$, $f_{sp,\theta}$ and $f_{sy,\theta}$

Steel temperature θ (°C)	$E_{s,\theta}/E_s$		$f_{sp,\theta}/f_{yk}$		$f_{sy,\theta}/f_{yk}$	
	Hot-rolled	Cold-worked	Hot-rolled	Cold-worked	Hot-rolled	Cold-worked
20	1.00	1.00	1.00	1.00	1.00	1.00
100	1.00	1.00	1.00	0.96	1.00	1.00
200	0.90	0.87	0.81	0.92	1.00	1.00
300	0.80	0.72	0.61	0.81	1.00	1.00
400	0.70	0.56	0.42	0.63	1.00	0.94
500	0.60	0.40	0.36	0.44	0.78	0.67
600	0.31	0.24	0.18	0.26	0.47	0.40
700	0.13	0.08	0.07	0.08	0.23	0.12
800	0.09	0.06	0.05	0.06	0.11	0.11
900	0.07	0.05	0.04	0.05	0.06	0.08
1000	0.04	0.03	0.02	0.03	0.04	0.05
1100	0.02	0.02	0.01	0.02	0.02	0.03
1200	0	0	0	0	0	0



Strain range	**Stress** σ_θ	**Tangent modulus**
$\varepsilon_{sp,\theta}$	$\varepsilon E_{s,\theta}$	$E_{s,\theta}$
$\varepsilon_{sp,\theta} \le \varepsilon \le \varepsilon_{sy,\theta}$	$f_{sp,\theta} - c + (b/a)[a^2 - (\varepsilon_{sy,\theta} - \varepsilon)^2]^{0,5}$	$\dfrac{b(\varepsilon_{sy,\theta} - \varepsilon)}{a[a^2 - (\varepsilon - \varepsilon_{sy,\theta})^2]^{0.5}}$
$\varepsilon_{sy,\theta} \le \varepsilon \le \varepsilon_{st,\theta}$	$f_{sy,\theta}$	0
$\varepsilon_{st,\theta} \le \varepsilon \le \varepsilon_{su,\theta}$	$f_{sy,\theta}[1-(\varepsilon-\varepsilon_{st,\theta})/(\varepsilon_{su,\theta}-\varepsilon_{st,\theta})]$	-
$\varepsilon = \varepsilon_{su,\theta}$	0	-
Parameters	$\varepsilon_{sp,\theta}=f_{sp,\theta}/E_{s,\theta}$ $\varepsilon_{sy,\theta}=0.02$ $\varepsilon_{st,\theta}=0.15$ $\varepsilon_{su,\theta}=0.20$ except for Class A reinforcement for which: $\varepsilon_{st,\theta}=0.05$ $\varepsilon_{su,\theta}=0.10$	
Functions	$a^2=(\varepsilon_{sy,\theta}-\varepsilon_{sp,\theta})(\varepsilon_{sy,\theta}-\varepsilon_{sp,\theta}+c/E_{s,\theta})$ $b^2=c(\varepsilon_{sy,\theta}-\varepsilon_{sp,\theta})E_{s,\theta}+c^2$ $c=\dfrac{(f_{sy,\theta}-f_{sp,\theta})^2}{(\varepsilon_{sy,\theta}-\varepsilon_{sp,\theta})\varepsilon_{sp,\theta}-2(f_{sy,\theta}-f_{sp,\theta})}$	

Figure 5.2-3: Idealized stress-strain diagram for fire design and corresponding mathematical model

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5.3 Prestressing steel

5.3.1 General

Steels for prestressing are delivered as:

- wire;
- 2-wire strands, 3-wire strands, 7-wire strands, 19-wire strands;
- bars.

Prestressing steel is characterized by:

- geometrical properties :
  - surface characteristics : plain, indented or ribbed,
  - nominal diameter,
  - nominal cross – sectional area,
  - nominal mass per meter,
  - nominal dimensions of indentations (wire and strand),
  - pitch length (strand),
  - left or right lay (strand),
  - nominal pitch and dimensions of ribs or threads (bars),
  - left or right thread (bars).
- mechanical properties:
  - the tensile properties: ultimate tensile strength (UTS), 0.1% proof stress, and total elongation at ultimate tensile strength ,
  - modulus of elasticity,
  - fatigue behaviour,
  - behaviour under extreme thermal conditions,

The standard tests are defined in ISO 15630/3.

The 0.1% proof stress is sometimes called “yield stress”.

Behaviour at extreme temperature may cover either the behaviour at low temperature (e.g. for cryogenic applications) or high temperature (e.g. behaviour in case of fire).

Bond aspects are covered in Chapter 6 Interface Characteristics.

Coatings mainly serve as supplementary corrosion protection. Some coatings modify the bond characteristics of prestressing steel.

Sheathings can serve either serve as corrosion protection and/or for containment of a lubricating filler inside the sheathing. For soft fillers inside the sheathing like grease or wax the prestressing steel remains permanently unbonded.

Resins have also been used as filler. Resins with delayed hardening can provide bond.

Examples for product standards for prestressing steels are: EN10138 - ASTM A416, A421 and A722 - JIS G3536 and G3109.

Types of prestressing steels not covered by product standards may be used after it has been shown that they meet the specified minimum requirements.

The properties of prestressing steels are generally ensured by certification schemes and certification compliance. The methods of testing and of certification of conformity are defined in the relevant national or international standards or recommendations by RILEM.

- bond characteristics,
- the ductility properties appropriate to the product type such as:
  - percentage reduction of area, resistance to reverse bending, and resistance to bending.

Prestressing steel can be provided during the manufacture with coatings or sheathing:

- metallic coating,
- organic coating,
- exterior sheathing with a filling product.

Prestressing steels shall comply with national or international product standards that specify their geometrical, mechanical and technological properties. Testing of prestressing steel shall be carried out in accordance with national or international standards such as ISO 15630-3.

5.3.2 Quality control

The fabrication of prestressing steels shall be subject to a factory production control by the manufacturer, and a continuous external control by an independent qualified body, including certification and regular audits.

5.3.3 Designation

The designation of the product shall consist of:

- the reference of the relevant product standard,
- the steel designation consisting of:
  - the type of the prestressing steel (wire – strand – bar),

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- the nominal Ultimate Tensile Strength (UTS) in MPa,
- the nominal diameter,
- the relaxation class,
- the fatigue class,
- the stress corrosion resistance class.

Information on complementary characteristics specific to a product may include:

- for wire:
  - the type of surface: plain or indented (with the type of indentation),
  - the type of coating (if any);
- for strand:
  - the number of wires (2, 3, 7 or 19),
  - the type of coating (if any),
  - the type of sheathing and filler (if any),
  - the type of surface: plain, indented or compacted,
  - the performance level of deflected tensile test behaviour;
- for bars:
  - the type of surface: plain or ribbed,
  - the type of ribs: hot rolled or cold rolled.

Each product shall be clearly identifiable with respect to this designation.

5.3.4 Geometrical properties

Geometrical properties are mainly nominal diameter and nominal cross sectional area.

Products shall comply with the geometrical properties specified in national or international product standards.

The difference between actual and nominal geometrical properties shall not exceed the tolerances specified in the relevant product standards.

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The process of manufacture of prestressing steel may influence several properties. This is particularly true for bars and the following properties:

- modulus of elasticity,
- bending,
- the ratio 0.1% proof stress to UTS,
- ductility.

UTS value expressed in MPa is often called grade of prestressing steel.

The ratio of 0.1% proof stress to UTS:

- ≥ 88% for wires,
- ≥ 86% for strands.

The ratio for bars should be declared by the manufacturer.

For more detailed information reference is made to CEB Bulletin 187, pp.3.20-2.27.

Due to specific details of the manufacturing process the stress-strain diagram may be somewhat different between different manufacturers.

5.3.5 Mechanical properties

The standard tests are defined in ISO 15630-3.

5.3.5.1 Tensile properties

The characteristic values of

- the UTS (f_{pt})
- the 0.1% proof stress ($f_{p\,0.1}$)
- the percentage total elongation at maximum force (ε_{pu})

are respectively denoted f_{ptk}, $f_{p\,0.1k}$ and ε_{puk}.

The tensile properties of prestressing steel increase with speed or rate of strain application.

5.3.5.2 Stress-strain diagram

Indicative stress-strain diagrams of prestressing steel in tension are represented in Figure 5.3-1 and Figure 5.3-2.

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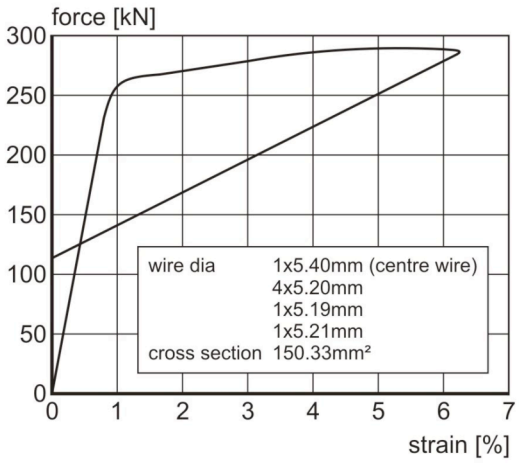


Figure 5.3-1: Typical stress-strain diagrams for wire and strand

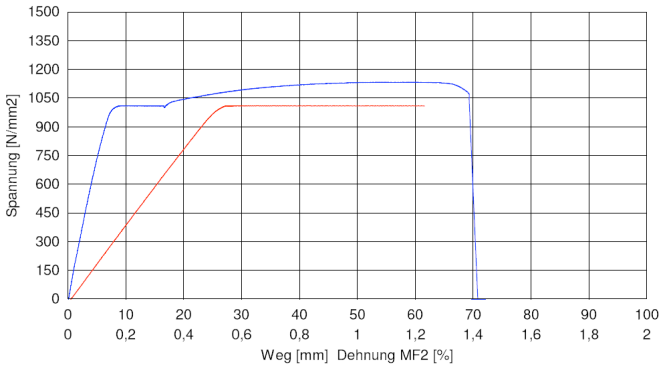


Figure 5.3-2: Typical stress-strain diagrams for bar prestressing steel

Nominal value of the modulus of elasticity of the prestressing steel E_p may be taken equal to:

- 205.000 MPa for wires,
- 195.000 MPa for strands (approximately)

The value for bars should be requested from the manufacturer.

The modulus of elasticity of the prestressing steel E_p shall be declared and certified by the manufacturer.

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Other fatigue behaviour may be required if the steel is in an aggressive environment (e.g. near seaside) or at temperatures other than room temperature.

In Japan, the fatigue behaviour is defined in JSSC (Japanese Society of Steel Construction).

Recommendations for fatigue testing:

The frequency of load cycles should not exceed:

– 120Hz for wire and bar;

– 20Hz for strand.

The temperature during the test should not exceed 40°C.

The minimum fatigue stress ranges indicated are valid for prestressing steels tested in air. If the prestressing steels are in contact with metal or concrete/grout surfaces, their fatigue strength may be reduced (fretting fatigue).

It may be assumed that prestressing steels are typically suitable for use under cryogenic conditions showing 0,1% proof stress and ultimate strength higher than at room temperature and an elongation at maximum load over 2% when tested at –196 °C.

Stress-strain diagrams are suitable for assessment of the behaviour at cryogenic temperature.

Refer to the FIP State of the Art Report "Cryogenic behaviour of materials for prestressing concrete" 1982 and FIP Special Report SR88/2, June 1988.

5.3.5.3 Fatigue behaviour

Prestressing steel shall comply with fatigue stress range requirements determined at $2x10^6$ load cycles performed at an upper stress of 70% or 80% of UTS. The minimum fatigue stress range shall comply with Table 5.3-1.

Table 5.3-1: Minimum fatigue stress range of prestressing steels at upper stress of 70% or 80% of UTS

Type of steel		Stress range [MPa]
Plain wire		200
Indented wire		180
Plain strand		190
Indented strand		170
Plain bars	$d \leq 40$	200
	$d > 40$	150
Ribbed bars	$d \leq 40$	180
	$d > 40$	120

5.3.5.4 Behaviour under extreme thermal conditions

The above values are valid for “normal” temperatures between -40°C and +40°C.

Cryogenic conditions

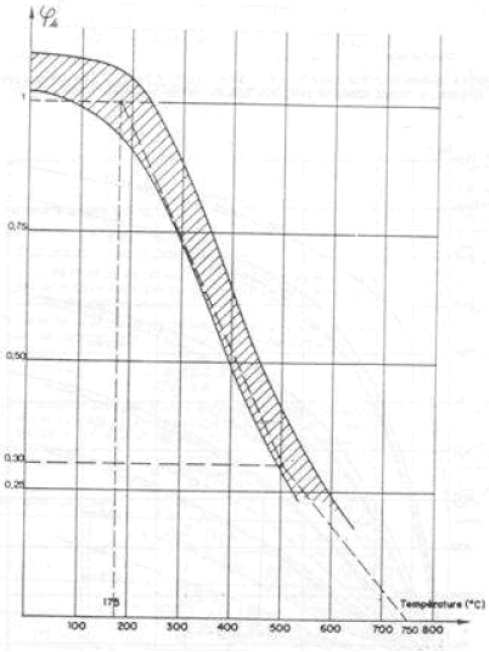
Prestressing steels for cryogenic conditions shall be specifically ordered for this purpose. The temperature for which they shall be tested shall be specified.

The producer shall provide test results of the prestressing steel for the specified cryogenic temperature.

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The threshold value is in the temperature range of 150 °C to 250 °C.

At 350 °C, tables and formulas show a 40% reduction of performance (test results have given 25% to 35% reduction of performance). Figure 5.3-3 gives an indication of the loss of strength as a function of temperature.



N.B.: φ_A is the reduction factor of strength of prestressing steel

Figure 5.3-3: Effect of temperature on the strength of prestressing steel

Quality control tests for the bond characteristics of prestressing steel should be performed in accordance with national or international standards. Alternatively, North American Strand Producers (NASP) Bond Test protocol for strand bond test may be used. Minimum bond strength should comply with relevant standards or should be agreed with the customer.

High temperature

Maximum force and 0.1% proof force decrease and elongation increases if the temperature increases beyond normal range.

5.3.5.5 Bond characteristics

The bond characteristics of prestressing steel vary depending on the surface characteristics and the manufacturing process. For prestressing steel intended to be used in bonded conditions, the producer shall document the bond characteristics through representative quality control testing performed on each production batch.

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The process of manufacture may influence several properties such as:

- stress relaxation,
- stress corrosion resistance (environmental cracking).

Relaxation values at an initial force of 80% UTS may be agreed between producers and purchasers.

Mostly two classes of relaxation are used (very low and normal relaxation). Sometimes three classes are used (very low, low and normal relaxation).

The trend worldwide is to use mostly very low relaxation prestressing steel. In Japan, wire and strand with a relaxation of 8 % is also manufactured (ordinary products).

The loss by relaxation increases if the stress in and the temperature of the prestressing steel increase, see Figure 5.3-4 and Figure 5.3-5.

The relaxation loss after infinite time may be assumed to 2 to 3 times the value at 1000 hours.

5.3.6 Technological properties

The standard tests are defined in ISO 15630-3.

5.3.6.1 Isothermal stress relaxation

The loss of force by relaxation shall be established at a nominal temperature of 20°C for a period of 1000h from an initial force of 70% of UTS.

Prestressing steels are available in different classes of relaxation. Table 5.3-2 gives values for very low relaxation.

Table 5.3-2: Maximum values of force loss of prestressing steels with very low relaxation after 1000 h.

Type of steel	**Initial stress**	**Specified maximum loss**
Wire / Strand	70% UTS	2.5%
Wire / Strand	80% UTS	4.5%
Bars ≤ 15mm	70% UTS	6%
Bars > 15mm	70% UTS	4%

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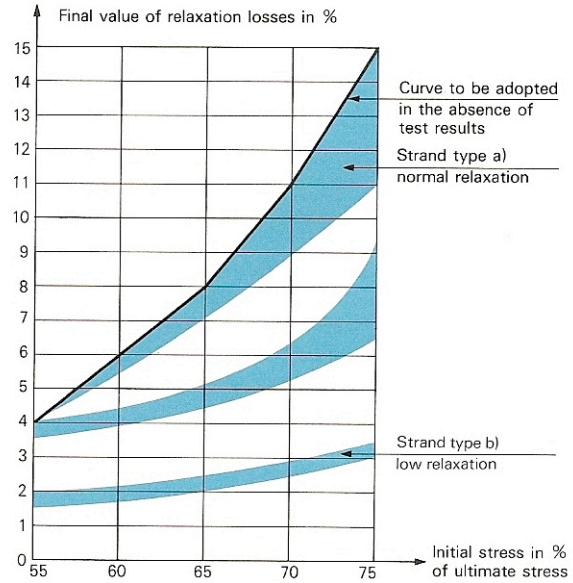


Figure 5.3-4: Effect of stress on relaxation loss

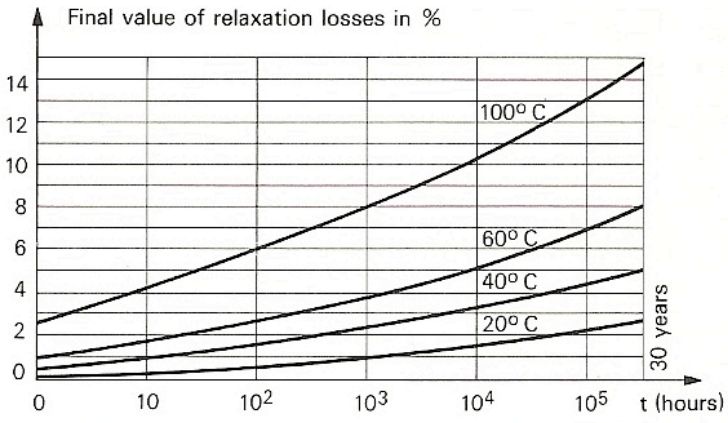


Figure 5.3-5: Effect of temperature on relaxation loss

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This test simulates the behaviour of strand in anchorages and curved ducts of post-tensioning tendons.

For details see FIP report on deflected tensile test.

For specific applications (stay cables or specific post-tensioning), a lower maximum reduction of 20% may be specified to the manufacturer.

The NH4SCN test (solution A of ISO 15630-3) is considered a suitable test for quality control of prestressing steels. It is a legitimate test when the dominant failure mechanism under service conditions is by hydrogen embrittlement.

Any new type of prestressing steel should be subject to stress corrosion testing using solution B of ISO 15630-3. In parallel, stress corrosion testing with solution A of ISO 15630-3 should be carried out to establish minimum individual and median values of lifetime to failure for such new types of prestressing steels.

Residual stresses, particularly the stresses created during cold-drawing of wires, have an influence on the shape of the stress-strain curve (i.e. the ratio of the 0,1% proof stress to UTS) on the stress relaxation losses and on environmental assisted cracking.

Average values or an upper limit of the surface residual stresses on the steel surface may be provided by the producer on request.

Usually it is assumed that prestressing wires of low relaxation have surface tensile stresses lower than 50 MPa.

5.3.6.2 Deflected tensile behaviour (only for strands with nominal diameter ≥ 12.5 mm)

For normal prestressing applications the maximum permitted reduction of the tensile strength of prestressing steel in a deflected tensile test shall be 28%.

5.3.6.3 Stress corrosion resistance

Prestressing steels shall be subject to ongoing quality control testing for stress corrosion. The minimum individual and median values of lifetime to failure shall be determined using solution A of ammonium thiocyanate specified in ISO 15630-3. The values shall be in accordance with the values given in the relevant product standards.

New types of prestressing steels shall be subject to initial approval testing for stress corrosion. Such prestressing steels shall pass 2000 hours in solution B specified in ISO 15630-3.

5.3.6.4 Coefficient of thermal expansion

Within the temperature range from -20 °C to 180 °C the coefficient of thermal expansion of prestressing steel may be taken as 10×10^{-6}/°C.

5.3.6.5 Residual stresses

Residual stresses on the surface influence the technological properties of prestressing steel.

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For more information reference is made to: J. Ruiz, J.M. Atienza, M. Elices, “Residual stresses in wires: Influence of wire length”, Journal of Material Engineering and Performance, Vol. 12(4), August 2003, pp. 480-489.

Prestressing steels are available with coatings and sheathing either for improved corrosion protection and/or to maintain the prestressing steel permanently unbonded inside the concrete or grout.

Metallic coating may be applied to wire, strand and bar.

Metallic coating may be applied to wire, strand and bar.

Sheathing is mostly applied to strand.

Properties of plastics and wax or grease are mentioned in NF A 35-037/NBNI10-008.

In Japan, see "Recommendation for design and construction of partially prestressed concrete (class III of prestressed concrete) structures" by architectural institute of Japan.

5.3.7 Special types of prestressing steel

Prestressing steels are available with coatings and with sheathing and different fillers.

5.3.7.1 Metallic coating

This coating is made by a process by hot-dip immersion in a zinc bath or zinc plus aluminium bath. It shall be made before the final thermo-mechanical treatment.

Specific quality controls are specified in standards (or in approval documents) and include:

- the mass per meter of coating,
- the appearance of the coating,
- the continuity of the coating,
- the adherence of the coating on the steel.

5.3.7.2 Organic coating

This coating is applied after the final thermo-mechanical treatment. Specific quality controls on the organic coating are specified in the relevant product standards.

5.3.7.3 Exterior sheathing with a filling product

Sheathing shall be made of high-density polyethylene or polypropylene. Fillers may be either grease or wax.

Application of sheathing and filler is made after the final thermo-mechanical treatment of the prestressing steel.

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The strand with low-friction sheathing is typically used for internal unbonded tendons (tendons cast inside concrete). This type of strand is filled with grease.

The strand with high-friction sheathing is typically used outside of concrete for non-injected tendons such as stay cables. This type of strand may be filled either with wax or grease depending on the application. If such strand is cast into concrete or inside grouted ducts, stressing is difficult or impossible due to the adherence.

All specific tests are defined in NF A 35-037 part 1/NBN I10-008 or in JIS K2220, 2246, 2265 and JIS K6922-2

The sliding test measures the actual adherence of the sheathing on the strand:

- for post-tensioning (low-friction sheathing),
- for stay cable (high-friction sheathing).

Static and dynamic leak tightness tests have been specified.

The carbon dioxide equivalent may be used for declaration.

Two basic types of sheathed products are available:

- strand with a low-friction sheathing (sliding strand) filled with grease,
- strand with high-friction sheathing (adherent strand) filled with either grease or wax.

Specific controls on the sheathing and on the filling products are specified in the relevant standard product (or in the approval document) and include:

- the mass of the sheathing,
- the mass of the filling product,
- the type of the sheathing and its geometrical characteristics,
- the properties of the finished product based on the following tests:
  - the sliding test,
  - the bonding test under thermal variation,
  - the splitting (cracking) test on the sheathing,
  - the leak tightness test,
  - the impact resistance.

5.3.8 Sustainability aspects

Manufacturers of prestressing steels should declare the quantity of greenhouse gases emitted during the fabrication and processing of prestressing steels.

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A general equation approaching the real behaviour is the Menegotto-Pinto function:

$$\frac{\sigma}{\sigma_0} = b\left(\frac{\varepsilon}{\varepsilon_0}\right) + d = b\left(\frac{\varepsilon}{\varepsilon_0}\right) + \frac{(1-b)\left(\frac{\varepsilon}{\varepsilon_0}\right)}{\left[1+\left(\frac{\varepsilon}{\varepsilon_0}\right)^n\right]^{1/n}}$$

where b is the ratio of the final to the initial stiffness and d is a value that is graphically defined in Figure 5.3-7. In the normalized space of stress and strain, the initial stiffness has a slope 1, the slope of the final tangent stiffness is b, and d varies from 0 to (1-b) as $\varepsilon / \varepsilon_0$ progressively increases from 0 to a maximum value at the last data point.

5.3.9 Assumptions used for design

The main parameters of prestressing steel to be used for design are as follows:

- characteristic UTS, f_{ptk},
- characteristic percentage total elongation at maximum force, ε_{puk} ,
- modulus of elasticity, E_p
- final relaxation loss.

For design a stress-strain diagram of prestressing steel similar to the real behaviour, see Figure 5.3-1 and Figure 5.3-2, but limited to a maximum stress of f_{ptk}, may be used. However, an idealized bi-linear stress-strain diagram as shown in Figure 5.3-6 may also be used assuming a nominal value for the modulus of elasticity of the particular type of prestressing steel.

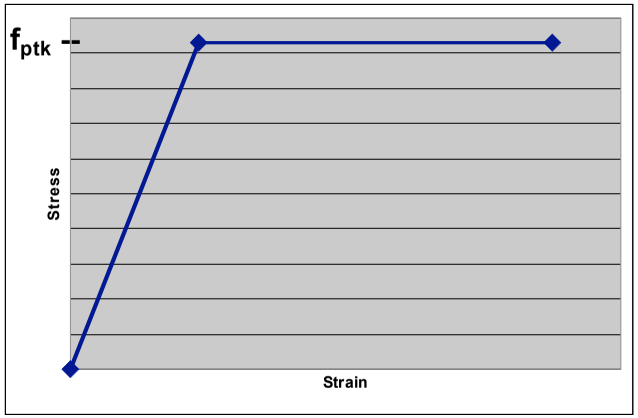


Figure 5.3-6: Idealized stress-strain diagram for prestressing steel

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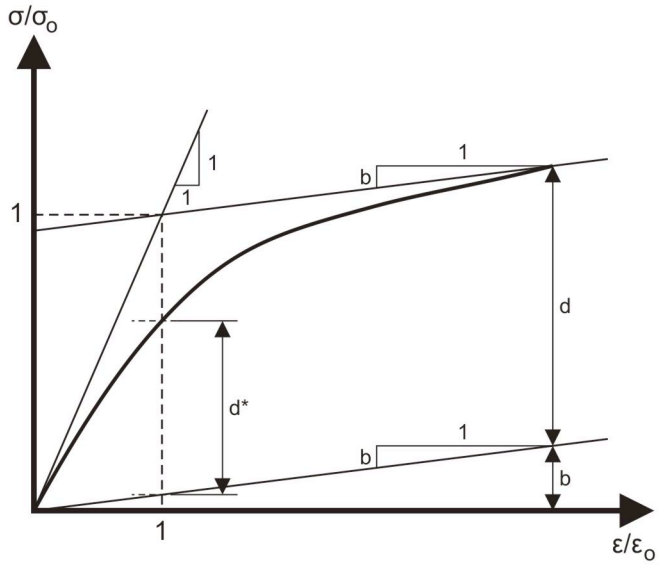


Figure 5.3-7 Menegotto–Pinto expression for the stress strain relation of steel

Without more detailed information, the final value of relaxation shall be assumed as three times the value at 1000h.

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Prestressing steels are provided as wires, strands or bars.

FRP materials are provided as wires, strands, bars or plates.

Extradosed tendons and stay cables which are partially or totally outside the outline of the structure, and prestressed ground anchors are not covered by this Model Code. Reference is made to: *fib* Bulletin 30, SETRA Recommendations, FIP Recommendations for Prestressed Ground Anchorages

Optional features of prestressing tendons may be provided such as to obtain:

- re-stressable tendons;
- exchangeable tendons;
- tendons for cryogenic temperatures;
- fully encapsulated tendons;
- monitorable tendons.

In the absence of applicable national or international approval procedures, ETAG 013 is recommended to be used. It includes detailed test procedures and acceptance criteria for verification of the system performance.

The CEN Workshop Agreement CWA 14646 is recommended as reference for the qualification of specialist companies and for the training of supervisors and personnel.

5.4 Prestressing systems

5.4.1 General

The prestressing tendons considered in this Model Code are made either of prestressing steel in accordance with clause 5.3, or of FRP materials in accordance with clause 5.5. These tendons may be:

- internal to the concrete, and
  - pretensioned, or
  - post-tensioned; in this case they may be bonded by grouting, or temporarily or permanently unbonded
- external to the concrete but totally within the external outline of the structure.

Prestressing may be used for any type of structure for:

- new construction;
- repair and strengthening of existing structures.

Post-tensioning systems shall comply with national or international approval procedures.

Prestressing tendons with all necessary components shall be installed by qualified specialist companies with sufficiently experienced supervisors and suitably trained personnel.

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5.4.2 Post-tensioning system components and materials

5.4.2.1 Anchorages and coupling devices

General

After hardening of the concrete, the tensile elements of tendons are tensioned and their extremities are fixed within anchorages, which transfer the prestressing forces to the concrete.

There are three basic types of anchorages:

- stressing anchorages which permit stressing of the tendon;
- fixed anchorages which do not permit stressing of the tendon;
- coupling devices to connect the end of a tendon, which has been tensioned first, to the end of a second tendon, placed as an extension of the first, and which will be tensioned in a second stage.

With unbonded tendons, special attention should be given to the potential consequences of an accidental tendon failure because the tendon force is lost over the entire tendon length.

With external prestressing, deviating devices are placed between the tendons and the structure to deflect the tendon as needed. These devices and their fixing zones, have to be designed to transfer the corresponding design action, taking the permissible installation tolerances into account.

With external prestressing, provision shall be made for the future replacement of the prestressing tendons.

Information relating to anchorage arrangements is given in the approval documents. When the assumptions or service conditions differ from those envisaged by these, additional experimental checks may be necessary.

Fixed anchorages can be mechanical devices or the tendon anchored by bond.

It may be necessary to place intermediate anchorages, functioning in both directions, to reduce the risk of progressive collapse, when the strength of the structure is achieved by one set of tendons extending over many spans.

When tendons are bonded by cement grouting, the transfer of the prestressing force may be ensured by bond in sections adjacent to the failure.

The deviators have to be designed for both transverse (deviating) and longitudinal (friction) effects and the corresponding displacements of the tendon.

It is recommended to make special provisions for access and anchorage attachment in the diaphragms, and for deviation devices to permit future addition of external tendons. These provisions shall be made for an additional prestressing force specified by the Engineer. A minimum provision for 10% of the primary prestressing force or moment capacity is recommended.

The future addition of external tendons is intended for additional future load or deflection adjustment, if ever needed.

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The producer should assess values of fracture toughness (K_{IC} or J_C) of new materials without prior experience for this application used for anchor heads and coupling devices. Susceptibility to environmentally assisted cracking of these materials should be ascertained.

Relevant test procedures are specified in the FIP Recommendations for the acceptance of post-tensioning systems or in the ETAG 013.

The performance of tendons made of prestressing steel is based on either the actual tensile strength or the strength specified in standards. A minimum elongation of 2% is specified to ensure yielding and ductility of the tendon.

The anchorage performance of tendons made of FRP materials is based on the tensile strength of the tendon declared by the supplier for short-time loading since material properties are not standardised yet. Since FRP materials do not yield, the minimum elongation of the tendon at maximum load shall be declared by the supplier and be used as reference for acceptance.

Design of structures with FRP tendons shall be based on the long-term strength of the FRP tendon with consideration of stress rupture.

Anchorages shall be made of materials suitable for this purpose.

Anchorage and coupling device performance

Post-tensioning tendon anchorages and coupling devices shall have the following minimum performance in the specified tests:

(a) Tendons made of prestressing steel:

- in the tensile test, achieve not less than 95% of the actual tensile strength of the prestressing steel with an elongation under maximum load of not less than 2%
- in the fatigue test, have not more than 5% of the tendon cross section fail with a stress range of 80 MPa at an upper stress of 65% of the tensile strength of the prestressing steel over $2x10^6$ load cycles
- in the load transfer test, resist to not less than 110% of the specified tendon strength at the specified minimum concrete strength for stressing to the maximum force.

(b) Tendons made of FRP materials:

- in the tensile test, achieve not less than 95% of the declared tensile strength of the FRP tendon and a minimum elongation at maximum load which is equal to or exceeds the value declared in the system documentation
- in the fatigue test, have not more than 5% of the tendon cross section fail with a stress range of 80 MPa at an upper stress of 65% of the declared tensile strength of the FRP tendon over $2x10^6$ load cycles
- in the load transfer test, resist to not less than 110% of the declared tendon strength at the specified minimum concrete strength for stressing to the maximum force.

Technological aspects

Friction loss characteristics of the tendons inside the anchorages and coupling devices, and tendon seating loss characteristics in the anchorages

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When FRP tendons are used as external tendons, they are often applied without duct.

Other types of ducts may be considered if they provide the required features and if they offer equivalent performance to corrugated metal and plastic ducts or to smooth steel and plastic pipe, as may be applicable. The formation of deleterious galvanic cells between the tensile elements and the duct material should be avoided.

Performance requirements for ducts, testing procedures and corresponding acceptance criteria may be found for the different types of ducts in:

- corrugated metal ducts: EN 523 and 524,
- corrugated plastic ducts: *fib* Bulletin 7,

and coupling devices, shall be declared in the post-tensioning system documentation. These characteristics shall be declared in technical approval documents where these exist.

5.4.2.2 Ducts

General

Depending on the intended use, ducts for prestressing systems shall provide one or several of the following features:

- forming a cavity in the structure for the installation of the tensile elements and defining the tendon path in the structure for internal tendons,
- forming a conduit for the installation of the tensile elements and deviating the tendon at specified locations in the structure for external tendons,
- providing an interface suitable for the transfer of bond stresses from the tensile elements to the structure for bonded tendons,
- providing a low friction interface suitable for unbonded tendons,
- providing corrosion protection to the tensile elements if made from a non-corrodible material and forming a leak tight envelope.

Ducts for prestressing tendons shall be either:

- corrugated metal ducts,
- corrugated plastic ducts,
- smooth steel pipes,
- smooth plastic pipes.

Performance of ducts

Ducts shall be designed, fabricated and installed such that they are fit for the intended purpose and that they have the expected durability.

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- smooth steel pipe: ISO 4200 "Plain end steel tubes, welded and seamless; general tables of dimensions and masses per unit length", (1991),
- smooth plastic pipe: ETAG 013.

The cross-sectional area of the ducts should normally be in the range of 2.0 to 2.5 times that of the actual area of the tensile elements, depending on the length and geometry of the tendon as well as on the installation method use (pushing strand by strand versus pulling complete bundle). The above recommendations correspond to duct filling ratios between 0.4 and 0.5.

The diameter of ducts for external unbonded post-tensioning should be generally of the same size as for internal bonded since the requirement for complete filling of the duct is the same.

Relevant properties of ducts may include:

- range of recommended friction and wobble coefficients,
- bond characteristics,
- minimum duct wall thickness,
- permissible minimum radius of tendon curvature (for wear resistance),
- recommended support spacing and details,
- ageing characteristics / durability of plastic materials (e.g. minimum Oxidation Induction Time).

Technological aspects

The relevant properties of ducts shall be declared in the system documentation. These characteristics shall be declared in technical approval documents where these exist.

5.4.2.3 Filling materials

General

Depending on the intended use, filling materials for prestressing tendons shall provide one or several of the following features:

- providing permanent corrosion protection to the tensile elements,
- providing bond to the tensile elements and transferring bond stresses to the duct and structure for bonded tendons,

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– providing lubrication between the tensile elements and the duct to permit stressing of permanently unbonded tendons.

Filling materials shall be either:

– cementitious grout,
– grease,
– wax,
– resin.

Performance requirements

Filling materials shall be designed, mixed and installed such that they are fit for the intended purpose and that they have the expected durability.

Resins and other filling materials may be considered if they provide the required features and if they offer equivalent performance to cementitious grout, grease or wax.

FRP tendons often use resins for filling and/or bonding to the structure.

Performance requirements for filling materials, testing procedures and corresponding acceptance criteria may be found for the different types of materials in:

– cementitious grout: *fib* Bulletin 20, PTI Guide Specification for grouting of post-tensioned structures, EN 445-447(2007)
– grease, wax: ETAG 013, FIP Recommendations for corrosion protection of prestressing steel.

Technological aspects

Relevant technological aspects of the filling materials shall be declared in the system documentation as required. These characteristics shall be declared in technical approval documents where these exist.

5.4.2.4 Quality control

The fabrication of post-tensioning system components and materials shall be subjected to a factory production control adapted to the importance and criticality of the particular component and material.

The factory production control shall be subjected to a continuous internal control by the manufacturer, and a continuous external control by an independent qualified body, including regular audits.

Guidance for a suitable quality control of post-tensioning system components and materials may be found in ETAG 013.

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Guidance for the actual requirements, test procedures and acceptance criteria for different Protection Levels (PL1, PL2 and PL3) adapted to the protection provided by the structure may be found in *fib* Bulletin 33.

For internal and external post-tensioning tendons the following solutions may e.g. be considered to provide the required protection:

- PL1: Tendon with a duct and a filling material providing durable corrosion protection
- PL2: Tendon with PL1 plus an envelope, enclosing the tensile element bundle over its full length (including the anchorages), and providing a permanent leak tight barrier
- PL3: Tendon with PL2 plus the integrity of tendon or encapsulation to be monitorable or inspectable at any time.

5.4.3 Protection of tendons

5.4.3.1 Temporary corrosion protection

Tensile elements and anchorages in all pretensioned and post-tensioned applications shall obtain a suitable temporary corrosion protection adapted to the intended use, the expected environmental conditions and exposure, and the expected period until the permanent corrosion protection is applied.

5.4.3.2 Permanent corrosion protection

Tensile elements and anchorages for all pretensioning and post-tensioning tendons shall obtain a suitable permanent corrosion protection adapted to the intended use, the expected environmental conditions and exposure, and the specified design life of the structure in which the tendons are placed.

The protection provided by the structure and/or other protective systems on the structure may be considered for the required level of protection on the tendon itself.

Exposed surfaces of metal components shall obtain a suitable corrosion protection. The required maintenance procedures and intervals shall be specified in the project specification.

5.4.3.3 Permanent corrosion protection of prestressing steel

Three Protection Levels for tendons are recommended:

- PL1 for all tendons used in environments which have a relatively low aggressiveness and which are well protected by the structure,
- PL3 for all tendons used in aggressive environments and/or severe exposure and with low protection provided by the structure,
- PL2 for all tendons used in all other combinations of environments and/or exposure and protection provided by the structure.

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For pretensioning tendons the following solutions may for example be considered to provide the required protection:

- PL1: Tendon with sufficient concrete cover adapted to the environmental conditions and exposure
- PL2: Tendon with PL1, and additionally protected with a suitable permanent coating and special protection of tendon ends
- PL3: Tendon with PL2, plus a suitable corrosion monitoring system.

Some FRP materials are susceptible to UV radiation and other environmental effects.

Fire protection is typically provided by sufficiently thick cover with a suitable material.

Resins in FRP tendons are particularly susceptible to the effect of fire.

Early application of prestress may prevent or reduce cracking of concrete due to shrinkage and temperature effects.

The minimum concrete strength required at the time when tensioning takes place depends mainly on the design of the anchorage, the provided local zone reinforcement, the edge distance of the anchorage, and the spacing between adjacent anchorages.

5.4.3.4 Permanent protection of FRP materials

FRP tendons shall obtain a suitable permanent protection adapted to the intended use, the expected environmental conditions and exposure, and the specified design life of the structure in which the tendons are placed.

5.4.3.5 Fire protection

Tensile elements and anchorages for all pretensioning and post-tensioning tendons shall obtain a suitable protection against the effect of fire, adapted to the intended use and exposure, and the specified fire rating of the structure or element in which the tendons are placed.

5.4.4 Stresses at tensioning, time of tensioning

5.4.4.1 Time of tensioning

The time when prestressing takes place should be fixed with due regard to:

- deformation conditions of the structure,
- safety with respect to local stresses and the compressive strength of the concrete,
- early application of a part of the prestress to reduce shrinkage effects.

The minimum concrete strength required at the time when tensioning takes place is given in the approval documents and/or system documentation for the prestressing system concerned, and shall be specified in the project specifications.

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The specified maximum force applies to the part of the tendon between end anchorages, i.e. does not apply to the part of the tendon in the stressing devices which do not form part of the permanent tendon.

In exceptional cases in post-tensioning where unforeseen deviation of frictional behaviour on the site occurs, it may be impossible to obtain the needed prestressing force under the limitations of this clause. In such exceptional cases it is possible, if the actual tensile elements and prestressing technique allow it, to apply a higher stress at the end of the tendons. This stress should never exceed the value of $0.95\,f_{p0.1k}$, see Chapter 8.

F_{pnT} is the nominal long-term tensile strength of the tendon made of FRP material with consideration of stress rupture, at a time corresponding to the declared design life of the tendon.

The initial prestress (at time $t = 0$) is calculated taking into account the prestressing force and the permanent actions present at tensioning.

Friction losses for deviated pretensioning tendons, see subclause 5.4.5.2, can be calculated similarly to post-tensioning tendons but neglecting the unintentional angular displacements.

5.4.4.2 Tendons made from prestressing steel

The maximum tensile force in the tendons made from prestressing steel, both pretensioning and post-tensioning, should generally not exceed the lower of the following values before transfer of prestressing to the concrete

$$\boldsymbol{\sigma}_{p0,\max} = 0.80\,f_{ptk} \tag{5.4-1}$$

$$\boldsymbol{\sigma}_{p0,\max} = 0.90\,f_{p0.1k} \tag{5.4-2}$$

The maximum tensile force in the tendons, both pretensioning and post-tensioning, should generally not exceed the lower of the following values after transfer of the prestressing to the concrete:

$$\boldsymbol{\sigma}_{p0,\max} = 0.75\,f_{ptk} \tag{5.4-3}$$

$$\boldsymbol{\sigma}_{p0,\max} = 0.85\,f_{p0.1k} \tag{5.4-4}$$

5.4.4.3 Tendons made from FRP materials

The maximum tensile force in the tendons made from FRP materials, should generally, after transfer of prestressing to the concrete, not exceed 75% of the tendon tensile strength declared by the supplier, with consideration of stress rupture:

$$F_{p0,\max} = 0.75\,F_{pnT} \tag{5.4-5}$$

5.4.5 Initial prestress

5.4.5.1 General

The value of the initial prestressing force (at time $t = 0$) at a given section of abscissa x, is obtained by subtracting from the force at tensioning the different immediate losses described below.

5.4.5.2 Losses occurring in pretensioning beds

The following losses should be considered in design:

- loss due to friction at the bends (in the case of curved tensile elements),
- losses due to seating of the anchoring devices (at the abutments) when anchoring on a prestressing bed,

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– loss due to relaxation of the tensile elements during the period which elapses between the tensioning of the tendons and prestressing of the concrete.

5.4.5.3 Immediate losses occurring during stressing

Losses due to the instantaneous deformation of concrete

Account should be taken of the loss in tendon force corresponding to the deformation of concrete

– in the case of post-tensioned tendons, taking into account the order in which the tendons are stressed,
– in the case of pretensioned tendons, as a result of their action when they are released from the anchorages.

Losses due to friction (post-tensioning tendons)

In a cross-section which is at a distance x from a stressing anchorage, the stress $\sigma_{p0(x)}$ in the tendon being tensioned is lower than the stress at the anchorage $\sigma_{p0,max}$. The difference between these two stresses corresponds to losses due to friction:

$$\sigma_{p0}(x) = \sigma_{p0,\max}\ e^{-\mu(\alpha + kx)} \qquad (5.4\text{-}6)$$

where

– μ denotes the coefficient of friction between the prestressing steel and the duct
– α denotes the sum of the angular displacements over a distance x, irrespective of direction or sign (in radians)
– k denotes an unintentional angular displacement (per unit length) depending on the design layout (shape) of the tendon, stiffness of duct and spacing of duct supports (in radians/m).

Values for μ and k are declared in the system documentation for the particular tendon details and deduced from previous experience or testing with the same type of materials and construction. These values μ and k shall be declared in technical approval documents where these exist.

All values given below should be considered as indicative mean values.

The coefficient of friction μ is the product of the physical coefficient of friction μ_0 and the squeezing factor. This squeezing factor is dependent on the degree of filling of the duct. Where more exact investigations are not available, this squeezing factor can be assumed to be 1.3 to 1.35 for tendons filling the duct between 50% and 60%. The physical coefficient of friction μ_0 is influenced, interalia by the surfaces of tensile elements and ducts (micro- and macro-structures), rust, pressure, elongation of the tendon, etc.

If more accurate values are not available and in the case of tensile elements and duct being both without rust, the values given below can be assumed, for μ with a 50% filling. These values which are indicative mean values can be multiplied by 0.9 if slight lubrication is present, e.g. by means of soluble oil.

Under site conditions significant variations are possible. In the case of rust, variations of 50% or even higher may occur.

For the verification of the real values of prestressing losses at tensioning it is recommended to measure the transmission of prestressing force from one end of the tendon to the other (e.g. with lift-off testing at the dead-end anchorage) on some typical and/or critical tendons.

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For external tendons, the effect of unintentional angular displacement may be neglected.

(a) Friction losses in case of bonded internal tendons made from prestressing steel

Table 5.4-1: Typical friction coefficients μ for different types of prestressing steels and ducts

Type of prestressing steel	Corrugated metal duct	Corrugated plastic duct	Smooth steel pipe	Smooth plastic pipe
Cold drawn wire	0.14-0.18	0.08-0.12	0.25-0.3	0.08-0.12
Strand	0.16-0.20	0.10-0-14	0.25-0.3	0.10-0.14
Deformed bar	0.63-0.68			
Smooth and round bar	0.3-0.35			

The coefficient k takes account of unintentional angular deviations. k is also called wobble coefficient. Its value depends on the quality of workmanship and on the distance between supports of the tendon. Values for k are given in approval documents. The typical range may be assumed to $k = 0.005\text{-}0.01$ (m^{-1}). For internal tendons in precast segmental construction higher values for k are recommended to be assumed in design.

(b) Friction losses in the case of unbonded internal tendons made from prestressing steel

Tests and practical experience have shown that the friction factors μ and k as listed below can be applied.

- For monostrands (individually greased and plastic sheathed strands; single or grouped) $\mu = 0.05\text{-}0.07$ $k = 0.006\text{-}0.01$ m^{-1}
- For multistrand or multiwire tendons inside plastic pipe and filled with grease $\mu = 0.12\text{-}0.14$ $k = 0.004\text{-}0.008$ m^{-1}
- For dry multistrand or multiwire tendons (with dry air as subsequent corrosion protection) factors as for bonded internal tendons apply.

With external prestressing, the friction is concentrated at deviation devices.

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(c) Friction losses in the case of external tendons made from prestressing steel

- For bare dry strands or wires over steel saddle
  $\mu = 0.25\text{-}0.30 \qquad k = 0$
- For lubricated strands or wires over steel saddle
  $\mu = 0.20\text{-}0.25 \qquad k = 0$
- For dry strands or wires inside plastic pipe over saddle
  $\mu = 0.12\text{-}0.15 \qquad k = 0$
- For bundle of monostrands (individually greased and plastic sheathed strands) over saddle $\mu = 0.05\text{-}0.07 \qquad k = 0$

These values apply for saddle radii as given in 5.4.9. For lower radii further test evidence may be needed.

(d) Friction losses in the case of tendons made from FRP materials

- Friction and wobble coefficients declared by the supplier of the FRP tendons should be considered for the calculation of immediate losses during stressing.

Seating of the prestressing steel in the anchorage causes a shortening (negative elongation) of the tendon with a corresponding loss of tendon force in the vicinity of the anchorage.

The values for seating to be taken into consideration are defined in the system documentation and/or approval documents for the prestressing system concerned.

Due to this seating, the highest stress along the tendon is no longer at the anchorage.

Heat treatment is mostly used in precasting, and therefore, applies particularly to precast-pretensioned applications.

The loss of prestress due to relaxation during the heat treatment can be equated to 75% of the total value of relaxation losses.

<u>*Losses caused by seating of the tensile elements*</u>

Account must be taken of the loss which occurs during seating at the anchorage of post-tensioning tendons, i.e. during the operation of anchoring after tensioning.

Transfer of the prestressing force to the concrete in pretensioning tendons produces a loss in force in the tensile elements over the transfer length.

<u>*Effect of heat treatment curing*</u>

Two types of losses have to be taken into account :

- reduction of stress in the tensile elements due to an acceleration of relaxation during heat treatment,
- direct thermal effect.

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(a) Relaxation losses

Relaxation losses can be calculated by adding to the value of time a duration defined by:

$$t_{ep} = t_{p1}\ 1.04^{\,(T\max -20)} \qquad (5.4\text{-}7)$$

where

- T_{max} is the maximum temperature of the concrete during heat treatment in °C,
- t_{p1} is the mean duration of the heating cycle, calculated by:

$$t_{p1} = (T_{max} - 20)^{-1} \int_0^{t1} [T(t) - 20]\, dt \qquad (5.4\text{-}8)$$

where

- t_1 is the age of the concrete when its temperature returns to ambient temperature
- $T(t)$ is the temperature of concrete, in °C, at time t.

(b) Losses of direct thermal origin

Direct thermal effect is caused by

- the dilation of concrete, when it is not bonded to the tensile elements,
- the restraint to the dilation of concrete presented by the tensile elements when they are bonded.

This type of loss does not exist with moulds supporting the tension of tendons and heated together with concrete.

The losses of direct thermal origin can be calculated by:

$$\Delta\sigma = 0.9 E_p\, \alpha_p\, (T_{max} - T_0) \qquad (5.4\text{-}9)$$

where

- E_p is the elastic modulus of tensile elements
- α_p is the coefficient of thermal expansion of tensile elements
- T_0 is the temperature of tensile elements at tensioning

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– T_{max} is the maximum temperature of tensile elements during heat curing.

The time-dependent losses are usually determined under permanent loads on the structure.

Data for calculation of the deformations of concrete under creep and shrinkage are given in clause 5.1.

Ordinary reinforcement has an influence on the value of time-dependent shortening of concrete.

The interaction can be estimated as described in CEB Bulletin 199.

Data on the relaxation of tensile elements are given in 5.3.6.1 for prestressing steel and 5.5.5.7 for FRP materials.

Other immediate losses

Account should be taken of all possible causes of immediate loss of tendon force related to the tensioning process or the equipment used for tensioning.

5.4.6 Value of prestressing force

The initial prestressing force in a tendon is the force existing in this tendon at the end of the stressing operation. The initial prestressing force on a prestressed element is obtained by considering all the forces existing in the tendons, at the end of the last stressing operation.

The prestressing force at a given time t is obtained by subtracting from the initial prestressing force the value of the time-dependent losses at this time t.

These losses are due to creep and shrinkage of concrete and relaxation of tensile elements.

The final value of the prestressing force is obtained by subtracting from the initial prestressing force the expected value of the time-dependent losses.

5.4.6.1 Calculation of time-dependent losses

The time-dependent losses are calculated by considering the following two reductions of stress:

– the reduction of stress, due to the reduction of strain, caused by the deformation of concrete due to creep and shrinkage, under quasi-permanent actions:
  – for bonded tendons, the local deformation at the level of the tendon has to be considered;
  – for unbonded tendons, the deformation of the whole structure between the anchorages of the tendons has to be taken into account;
– the reduction of stress within the tensile elements due to the relaxation of this material under tension.

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The reduction of strain in tensile elements due to time-dependent losses may be calculated by dividing the stress loss by the modulus of elasticity of tensile elements.

In exceptional cases, several sets of prestressing tendons (practically never more than two), should be considered separately. These cases should be identified by judgement. The criteria, to be simultaneously satisfied, are that:

- the effect of the two sets are of contrary senses,
- these effects have the same order of magnitude,
- the dispersions are relatively high and there are qualitative reasons why they should not be considered as correlated.

In cases where the design value of the prestressing force influences the behaviour of the structure in a large over-proportional way, the designer may consider providing in the design the possibility for one or several of the following options:

- provide access at both tendon ends to allow stressing of the tendons from both ends,
- provide spare anchorages and ducts for additional internal tendons as and when required. If these spare ducts are not used at the time of construction, they should be filled after successful completion of all stressing and grouting operations,
- provide spare anchorages and deviators for additional external tendons as and when required. These anchorages and deviators may allow to add prestressing force not only at the time of construction but also at a later stage during the service life of the structure.

5.4.7 Design values of forces in prestressing tendons

5.4.7.1 General

Prestressing is usually exerted by a set of tendons. The total permanent force exerted at a given section (abscissa x), and at a time t, by the whole set is considered as the prestressing force.

5.4.7.2 Design values for SLS and fatigue verifications

For all verifications relating to cracking (decompression included), deformations, and for the analysis of the fatigue effect the mean value of prestressing force is taken as design value.

In cases where the design value of the prestressing force or any variation on site would influence the behaviour of the structure in a large over-proportional way, the designer shall take adequate precautions.

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Minimum spacing may be assumed as twice the minimum cover required for bond transfer, and is a function of the concrete strength and strength of tensile elements.

Based on experience, the following minimum radii of curvature of tendons have provided satisfactory behaviour in practice for tendons made from prestressing steel:

(a) internal bonded tendons: $R_{min} = 2.8 \sqrt{(f_{ptk} A_p [MN])} \geq 2.5m$

(b) internal unbonded monostrand tendons (greased and sheathed strands): $R_{min} = 2.5m$ for 0.6" and $R_{min} = 2.0m$ for 0.5"

5.4.7.3 Design values for ULS verifications

For all verifications at ULS the pre-strain corresponding to the mean value of prestressing force at SLS is taken and increased by the strain imposed onto the corresponding cross section (bonded tendons) or on the overall tendon (unbonded tendons) at the relevant ULS load combination.

5.4.8 Design values of tendon elongations

Design values for tendon elongations shall be calculated for the initial prestressing force, subclause 5.4.5.1, before losses caused by seating of prestressing steel in the anchorage.

For pretensioning, the initial prestressing force, subclause 5.4.5.1, before losses due to relaxation in the pretensioning bed and before heat treatment shall be considered.

The assumed values for friction losses, μ and k, shall be declared.

5.4.9 Detailing rules for prestressing tendons

5.4.9.1 Pretensioning tendons

Minimum radii of tendon curvature at deviators in the pretensioning bed shall be such that the tendon capacity at deviators complies with the tensile strength requirement for anchorage and coupling devices, see subclause 5.4.2.1.

Individual pretensioned tensile elements shall be spaced such that they comply with requirements for bond transfer.

5.4.9.2 Post-tensioning tendons

Minimum radii of tendon curvature for all types of tendons shall be such that the tendon capacity in the curvature complies with the tensile strength requirement for anchorage and coupling devices, see subclause 5.4.2.1. The minimum radii of curvature given in the commentary can be deemed to comply with subclause 5.4.2.1.

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(c) external tendons: $R_{min} = 1.4 \sqrt{(f_{ptk} A_p [MN])} \geq 2.0m$

(d) loop tendons: $R_{min} = 0.6 \sqrt{(f_{ptk} A_p [MN])} \geq 0.6m$

Loops are parts of tendons which are deviated by 180 degrees and in which the prestressing steel does basically not move relative to the duct during stressing

Typically, the tendons are assumed to exit from the anchorage straight and perpendicular to the bearing plate. The minimum straight length depends on the specific type of anchorage and is specified in the post-tensioning system approval documents.

The stresses on the concrete on the inside of the tendon curvature shall be checked and splitting or confinement reinforcement shall be provided when required.

Minimum radii of tendon curvature shall be declared by the tendon supplier in the system documentation. These radii shall be declared in technical approval documents where these exist.

The tendon shall exit in a geometry from the anchorage or coupling devices similar to the set-up tested for anchorage and coupling device performance, see subclause 5.4.2.1.

Ducts of groups of curved tendons shall be spaced such that the deviation forces from the curved tendons can be safely transferred around the adjacent duct on the inside of the curve.

Ducts of groups of tendons shall have minimum spacing which permits adequate placing and compacting of concrete.

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Fibres are commonly made of carbon, glass and aramid, but other materials are emerging, such as basalt and PBO (poly phenylene bezobisoxazole). As they occupy the largest volume fraction (40-70%) and have an elastic modulus much higher than that of the matrix, fibres are the principal stress-bearing constituent, while the resin transfers stresses among fibres and protects them.

FRP (non-metallic) reinforcement is available in various forms. For new structures bars, tendons and grids are used to reinforce and prestress concrete elements. In the repair sector, these elements are used to strengthen existing structures by means of external post-tensioning and near surface mounted reinforcement. Strips, laminates, sheets or fabrics are used for externally bonded reinforcement strengthening. Grids and fabrics may also be used in combination with shotcrete or mortar overlays.

The geometrical, mechanical and technological properties of FRP reinforcement basically depend on fibre and resin type and properties, constituent volume fractions, production parameters, shape and surface texture. In general they are characterized by high axial strength, high ratio of axial-to-transverse strength, limited ultimate strain, low weight, excellent chemical resistance and non susceptibility to a wide range of aggressive media, electromagnetic neutrality, excellent fatigue characteristics (depending on fibre type), limited ratio of long-term to short-term static strength for some fibre types.

5.5 Non-metallic reinforcement

5.5.1 General

Non-metallic reinforcing elements consist of a high number of continuous, directionalized, organic or inorganic fibres, typically embedded in a polymeric matrix. Both the terms 'non-metallic reinforcement' and 'FRP (fibre reinforced polymer) reinforcement' are used for this reinforcement type.

Non-metallic reinforcing products may be in the form of:

- pre-cured bars, tendons, strips, laminates, grids, profiles;
- sheets or fabrics applied by wet lay-up or pre-impregnated (prepreg).

Non-metallic reinforcement is characterized by:

- Geometrical properties:
  - configuration;
  - size;
  - surface characteristics.
- Mechanical properties:
  - tensile strength, modulus of elasticity and ultimate strain;
  - fatigue behaviour;
  - creep behaviour;
  - relaxation;
  - behaviour at elevated temperature and at extreme thermal conditions.

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- Technological properties:
  - bond characteristics;
  - bendability;
  - thermal expansion;
  - durability.

Non-metallic reinforcement shall comply with national or international product standards that specify their geometrical, mechanical and technological properties.

Examples of relevant international product standards are ISO 10406 (Part 1 and 2).

5.5.2 Quality control

The fabrication of non-metallic reinforcement shall be subject to a factory production control by the manufacturer, and a continuous external control by an independent qualified body, including certification and regular audits.

The methods of testing and certification of conformity are as defined in standards and recommendations, at the National, European or International level. For further information reference is made to *fib* Bulletins 14 and 40.

5.5.3 Designation

The designation of non-metallic reinforcing elements normally includes:

- the relevant product standard;
- the fibre and matrix materials;
- the nominal dimensions (e.g. diameter, width, thickness);
- the characteristic tensile strength, modulus of elasticity and the ultimate strain in the direction of the fibres.

Each product shall be clearly identifiable with respect to this designation.

5.5.4 Geometrical properties

5.5.4.1 Configuration

Non-metallic reinforcing elements may be:

- pre-cured;
- wet lay-up;
- pre-impregnated (prepreg).

Pre-cured systems are manufactured in various shapes, generally by pultrusion. Wet lay-up systems are manufactured with fibres lying in one (typically) or more directions and impregnated with the matrix at the job site. Prepreg systems are manufactured with unidirectional or multidirectional fibre sheets or fabrics pre-impregnated at the manufacturing plant with

partially polymerized resin. They may be bonded externally to concrete members with or without the use of additional resin.

The definition of the nominal cross-section is for pre-cured forms typically based on the global nominal dimensions (diameter, thickness, width). Alternatively or for wet lay-up/prepreg forms, an equivalent dry fibre cross-section may be used (referring to the continuous fibres as principal stress bearing component). The later is obtained as the ratio of the fibre mass per length and the fibre density. For FRP with multiple fibre directions, where a different amount of fibres per unit length is applied in different fibre directions, the definition of the nominal dry fibre cross-section always relates to the specified fibre direction, and more than one nominal cross-section (or nominal thickness) may be given depending on the fibre direction.

As design verifications are based on equilibrium of forces, the definition of the nominal cross-section (used in a consistent way) does not influence the outcome of the design. However, if data sheets of FRP products are compared, the possible difference in definition of the nominal cross-section should be taken into account.

Surface characteristics include the characteristics of the ribs or indentations or other surface deformations (e.g. sand-coating) by means of which bond with the concrete is achieved.

FRP to concrete bond quality is product specific, related to the surface characteristics. Further requirements are given in clause 6.2.

The configuration is further characterized by the type of fibre(s) and matrix, fibre orientation(s) and constituent fractions. Constituent material fractions can either be given by mass (weight) or by volume.

5.5.4.2 Size

The size of non-metallic reinforcing elements is defined by a nominal diameter for circular bars or by the nominal cross-section dimensions for other products (e.g. thickness, width).

The difference between actual and nominal cross-sectional area shall not exceed the limiting values specified in relevant product standards.

5.5.4.3 Surface characteristics

The surface of non-metallic reinforcement may be:

- plain;
- deformed.

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For quality control purposes and design calculations, the mechanical properties of a product are referred to the nominal cross-sectional area.

The standard tests are defined in relevant standards and recommendations, at the National, European or International level. For further information reference is made to *fib* Bulletins 14 and 40.

The requirements apply to the standard product, as tested on straight test coupons. The design considers effective characteristic strength values where relevant, e.g. at bent regions.

Typical fibre material notation is C (carbon), A (aramid), G (glass), H (hybrid).

The stress-strain response is quasi-linear elastic for most FRP elements. The slope of the stress-strain diagram gives the modulus of elasticity and is typically defined as a secant modulus, following product standards.

5.5.5 Mechanical properties

The mechanical properties are defined on the basis of standard tests.

5.5.5.1 Tensile strength and ultimate strain

The characteristic values of

- the tensile strength (f_f)
- the percentage total elongation at maximum force (ε_{fu})

are respectively denoted f_{fk} and ε_{fuk}.

5.5.5.2 Type

The non-metallic reinforcing element type denotes the fibre material, the value of the specified characteristic tensile strength in MPa, the mean secant modulus of elasticity in GPa and the specified characteristic ultimate strain in the principal direction of the fibres.

5.5.5.3 Stress-strain diagram and modulus of elasticity

An indicative stress-strain diagram of a non-metallic reinforcing element in tension is given in Figure 5.5-1.

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The stress-strain diagram given in Figure 5.5-1 is indicative, as FRP reinforcement products are available in a range of strength and stiffness values (see Table 5.5-1 for overall ranges).

Table 5.5-1: Tensile properties of FRP reinforcement

	CFRP	GFRP	AFRP
Tensile strength f_f [MPa]	600-3000	400-1600	600-2500
Modulus of elasticity E_f [GPa]	80-500	30-60	30-125
Ultimate strain ε_{fu} [%]	0.5-1.8	1.2-3.7	1.8-4.0

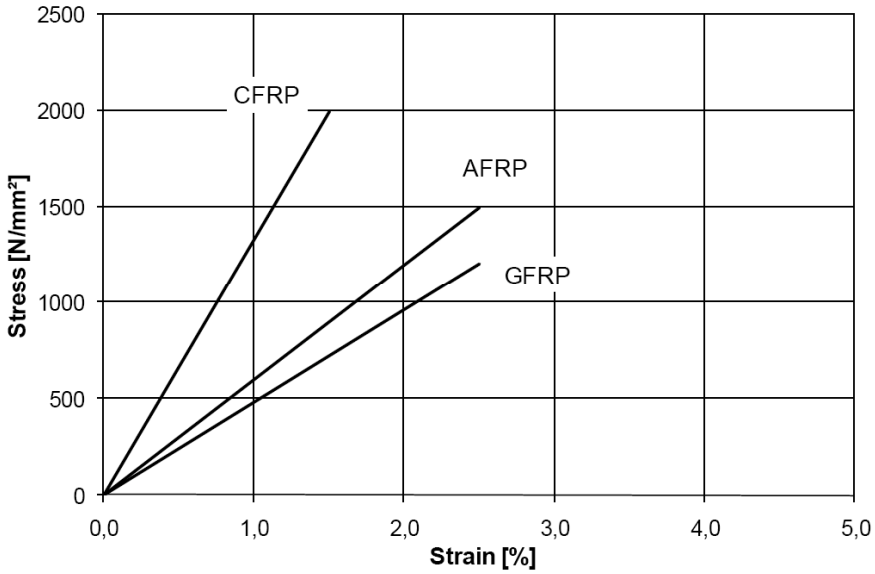


Figure 5.5-1: Stress-strain diagram of non-metallic reinforcement in the principal fibre direction

Given the generally limited compressive modulus of elasticity and the risk of microbuckling or kinking of the fibre within the restraint of the matrix material, non-metallic reinforcement is generally not used to resist high compressive stresses.

The interlaminar transverse shear strength of non-metallic reinforcement is basically dominated by the matrix and the fibres in off-axis directions.

High modulus fibre composites have superior fatigue resistance. Cyclic tension fatigue strength of unidirectional CFRP and AFRP exceeds that of prestressing steel, while that of GFRP is lower. The fatigue strength of CFRP is higher than for AFRP. Indicative values are given in Table 5.5-2.

5.5.5.4 Compressive and shear strength

The compressive or transverse shear properties for a particular reinforcing element, if needed for a particular application, should be given by the manufacturer, who should also provide a description of the test method used to determine the properties.

5.5.5.5 Fatigue behaviour

If a non-metallic reinforcing element is subjected to a large number of load cycles, growth of internal or surface flaws may occur, resulting in a reduced mechanical strength compared to the short-term static strength.

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Table 5.5-2: Fatigue strength of reinforcement after 2.10^6 cycles

	σ_{max}/f_{tk} (a) [-]	$\Delta\sigma$ (b) [MPa]
Prestressing steel	~ 0.60	~ 200
E-glass/polyester (rod)	~ 0.50	~ 60
E-glass/epoxy (rod)	~ 0.50	~ 75
Aramid/vinylester (rod)	~ 0.60	~ 235
Carbon/vinylester (rod)	~ 0.60	> 350
Carbon/epoxy (strand)	~ 0.60	~ 310

(a) Applied maximum stress as a function of the characteristic tensile strength of the reinforcement.

(b) Stress range yielding fatigue failure at 2.10^6 cycles.

FRP reinforcement combines elastic fibres, which have excellent resistance to creep, with a viscoelastic polymer matrix, which may show significant creep deformations. As FRP tensile members normally have a high degree of fibre orientation, large fibre volume fractions and a high ratio of fibre over matrix stiffness, the tensile force shared by the matrix is extremely low, so that FRP creep deformations are negligible.

The permissible stress level against stress rupture depends on the fibre/resin system, the alignment of the fibres and the fibre volume fraction. Stress rupture is adversely influenced by the environmental conditions. Generally, CFRP can withstand stress levels up to at least 80 % of its short-term strength, while considerably lower stress levels apply for AFRP (about 50 % on a 50 years basis) and GFRP (about 30 % on a 50 years basis).

Relaxation of GFRP, CFRP and AFRP prestressing elements after 50 years of loading can be estimated as 4 to 14 %, 2 to 10 % and 11 to 25 %, respectively. These values depend on the stress level and environmental influence.

5.5.5.6 Creep behaviour

Provided that the glass transition temperature is well above the service temperature, FRP creep deformations are generally negligible.

The long-term permissible stress of non-metallic reinforcement should be limited to avoid stress rupture.

5.5.5.7 Relaxation

Relaxation of non-metallic reinforcement is to be considered for prestressing loss calculations.

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Prestressing loss due to relaxation of FRP is compensated by a lower prestressing loss due to concrete shrinkage and creep (given the ratio of the modulus of elasticity E_f/E_c)

The glass transition temperature T_g is of particular importance, as it reflects the change of molecular mobility of polymer materials. For factory processed FRP elements, the matrix generally has a T_g in the range of 130 to 140 °C. The T_g of cold-cured (ambient-cured) adhesives/saturating resins may be lower (typically in the range of about 50 to 80 °C for epoxy).

FRP to concrete bond interaction is different from that of deformed steel rebars. Further requirements are given in clause 6.2.

Thermoset resin based FRP elements are not bendable in situ. Bent FRP elements are factory made and pre-cured. Thermoplast resin based FRP elements are bendable given proper application procedures.

Given the anisotropy of FRP elements, different CTE values are obtained in the longitudinal and the transverse direction. Indicative values are given in Table 5.5-3.

5.5.5.8 Behaviour under elevated temperature and under extreme thermal conditions

Although fibres exhibit relatively high thermal stability, polymer resins are strongly affected by temperature. As a result, the material properties of FRP are influenced by temperature and decrease drastically when reaching the glass transition temperature T_g. In the event of fire, sufficient concrete cover should be available so that the glass transition temperature is only reached after the required time span. For external reinforcement systems, fire protection systems may be required.

5.5.6 Technological properties

5.5.6.1 Bond characteristics

Bond characteristics of non-metallic reinforcement relate to the surface characteristics, as specified in subclause 5.5.4.3.

5.5.6.2 Bendability

The requirements concerning the bendability are specified in relevant standards.

5.5.6.3 Coefficient of thermal expansion

Values of the CTE for non-metallic reinforcement depend on the constituents and are product specific.

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Table 5.5-3: Coefficient of thermal expansion (CTE)

FRP type	**Longitudinal CTE** [10^{-6}/°C]	**Transverse CTE** [10^{-6}/°C]
AFRP	≈ - 2	≈ 30
CFRP	≈ 0	≈ 25
GFRP	≈ 5	≈ 25

In general, FRP materials appear to have a good to excellent chemical resistance against chemical attack. Glass fibre based FRP reinforcements have limited alkaline resistance, which is to be considered for internal reinforcement. FRP reinforcements may also be influenced by moisture, thermal stresses and UV radiation. For a detailed discussion on durability reference is made to *fib* Bulletins 14 and 40.

5.5.6.4 Durability

Non-metallic reinforcement has excellent chemical resistance and non-susceptibility to a wide range of aggressive media.

5.5.7 Assumptions used for design

The parameters of non-metallic reinforcement to be used for design are as follows:

- modulus of elasticity, E_f;
- characteristic tensile strength, f_{fk};
- ultimate strain, i.e. characteristic percentage total elongation at maximum force, ε_{fuk}.

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For serviceability limit state verifications a linear stress-strain response $\sigma_f = E_f \varepsilon_f$ is considered, referring to the mean value of the secant modulus of elasticity E_f.

For ultimate limit state verification, the design stress-strain curve is idealised by means of a linear response (Figure 5.5-2), given the characteristic tensile strength f_{fk} and ultimate strain ε_{fuk}. The slope of this design stress-strain curve, refers to a modulus f_{fk}/ε_{fuk}.

The material factor γ_m for non-metallic reinforcement takes into account the scattering of strength values and the consistency in material failure mode. Sometimes the application of non-metallic reinforcement is designed for durability. This can be considered explicitly in the design by considering for the relative resistance of generic FRP types to aggressive environments and the desired service life of the structure. These allow consideration of adapted values for the material factor and allowable stress level, as outlined in *fib* Bulletins 14 and 40.

For design purposes an idealized stress-strain diagram according to Figure 5.5-2 shall be used.

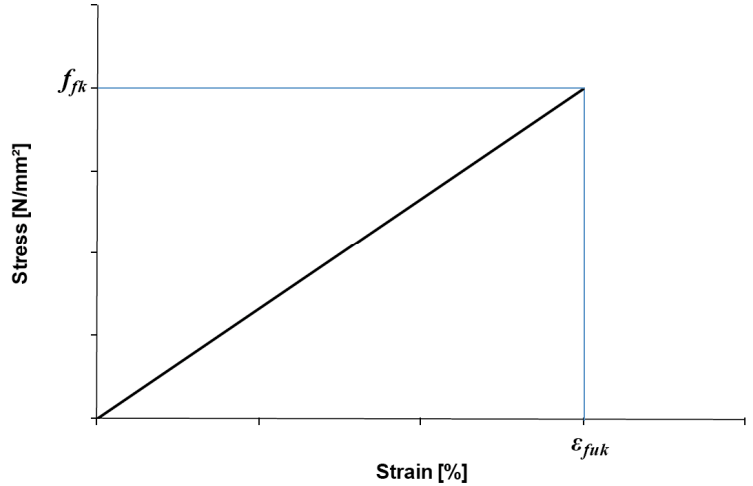


Figure 5.5-2: Idealized stress-strain diagram

The values of the material factor γ_m for non-metallic reinforcement are given in Table 5.5.-4.

Table 5.5-4: Partial factors-γ_m for non-metallic reinforcement

Fundamental basic variable	**Design situation**	
	Persistent/transient	**Accidental**
Non-metallic reinforcement Tensile strength (f_{fk}), γ_f	1.25	1.0

For stress limitation of non-metallic reinforcement, reference is made to subclause 5.5.5.6.

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Contrary to rebars or welded mesh, most fibres slip without failing in tension. Fibres are active as soon as (micro-) cracks are formed in the concrete. The main advantage of adding fibres to concrete or mortar is that they generate a post-cracking residual tensile strength in combination with a large tensile strain. As such, the material Fibre Reinforced Concrete (FRC) is characterized by substantial ductility and toughness.

The properties of the composite depend on the characteristics of the constituting materials as well as on their dosage. Other factors as the geometry, the volume fraction and the mechanical properties of the fibres, the bond between fibre and concrete matrix as well as the mechanical properties of the matrix, significantly affect the FRC properties.

Fibre orientation significantly affects fibre reinforced concrete proper-ties after cracking. Due to differences in casting and vibration procedures anisotropic distributions may occur, the effect of which should be taken into account.

The rules in this chapter are based most of all on experience with steel fibre reinforced concrete.

For ultra high strength fibre concrete additional rules may apply. Those will be published in a separate *fib* Bulletin by Task Group 8.6.

In the case of softening behaviour (a) the deformations localize in one crack. In the case of hardening behaviour (b) multiple cracking occurs before reaching the peak value.

The relation between strain softening and strain hardening behaviour is shown in Figure 5.2. Softening behaviour in tension can correspond to hardening behaviour in bending. Even a bending softening material can result in a hardening behaviour of a suitable structure (Figure 5.6-1).

5.6 Fibres/fibre-reinforced concrete

5.6.1 Introduction

Fibre-Reinforced Concrete (FRC) is a composite material characterized by a cement matrix and discrete fibres (discontinuous). The matrix is made of either concrete or mortar. Fibres can be made of steel, polymers, carbon, glass or natural materials.

Fibre materials with a Young's-Modulus which is significantly affected by time and/or thermo-hygrometrical phenomenon are not covered by this Model Code.

Mixtures of different types and/or sizes of fibres can also be used (called hybrid fibre reinforced concrete).

FRC for structural applications means the use of design constitutive laws to consider the post-cracking residual strength provided by fibre reinforcement. Other cases, like early age crack-control or fire resistance, are considered non structural use of FRC.

For structural use, a minimum mechanical performance of FRC must be guaranteed.

Fibres can be used to improve the behaviour in the SLS since they can reduce crack spacing and crack width, thereby improving durability.

Fibres can be used to improve the behaviour at the ULS where they can partially or totally substitute conventional reinforcement.

The mechanical properties of a cementitious matrix are modified when fibres are added. However, elastic properties and compressive strength are not significantly affected by fibres, unless a high percentage of fibres is used.

Depending on their composition, Fibre Reinforced Concretes (FRC) can show hardening or softening behaviour (Figure 5.6-2).

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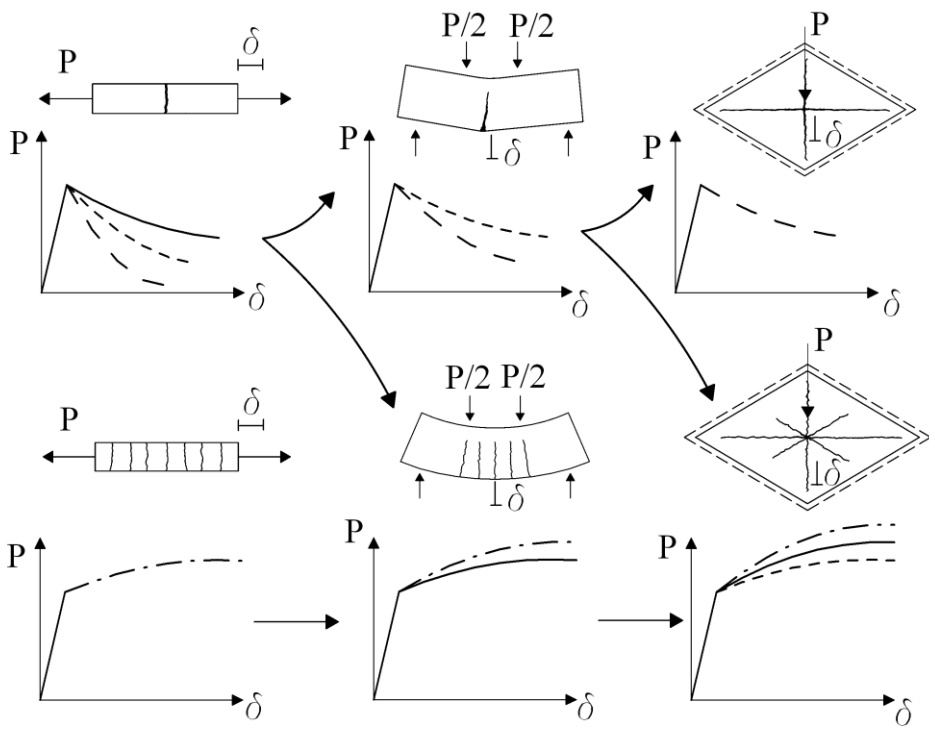


Figure 5.6-1: Main differences between materials having softening or hardening behaviour from material to structural level

Fibres can reduce the brittleness of concrete in compression, especially in high or ultra high strength concrete (Figure 5.6-3).

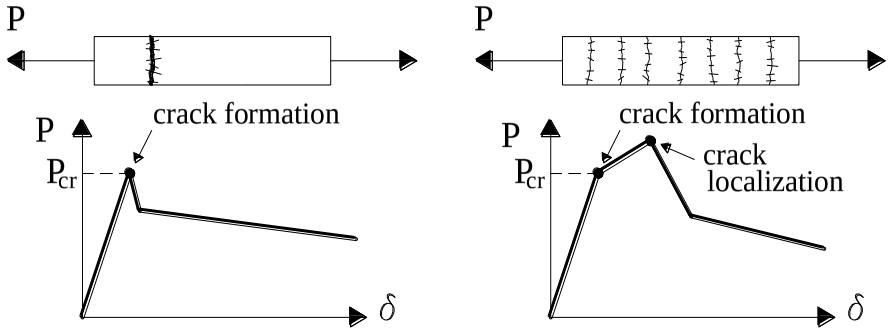


Figure 5.6-2: Softening (a) and hardening (b) behaviour in axial tension.

5.6.2 Material properties

5.6.2.1 Behaviour in compression

Generally the compressive relations valid for plain concrete apply to FRC as well.

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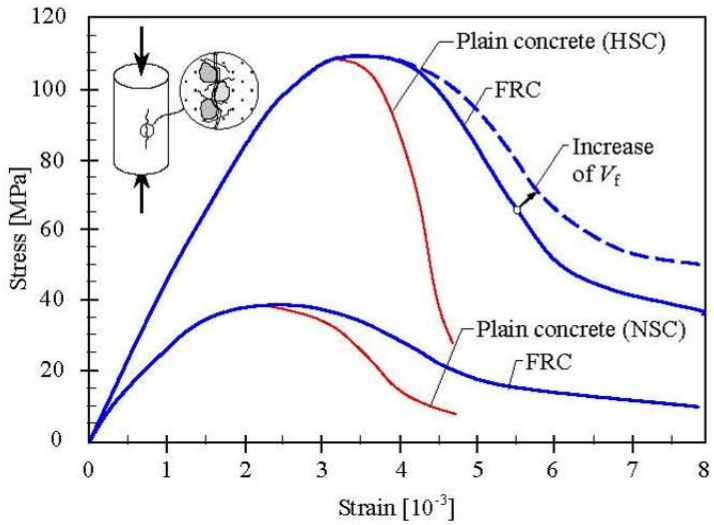


Figure 5.6-3: Main differences between materials having softening or hardening behaviour from material to structural level

Uniaxial tensile testing is not advised for standard testing of new mixtures because tensile tests are difficult to carry out and interpret.

Because the specimens are normally small, the number of fibres in the governing plane will be small as well and there can be a significant fibre orientation effect due to the method of manufacturing.

5.6.2.2 Behaviour in tension

With regard to the behaviour in tension, which is the most important aspect of FRC, various test methods are possible.

Bending tests can be carried out aiming at determining the load-deflection relation. The results can be used for deriving the stress – crack width relations by inverse analysis, performing equilibrium calculations for numerous crack openings as shown in Figure 5.6-4. A simpler approach can be found in subclause 5.6.4.

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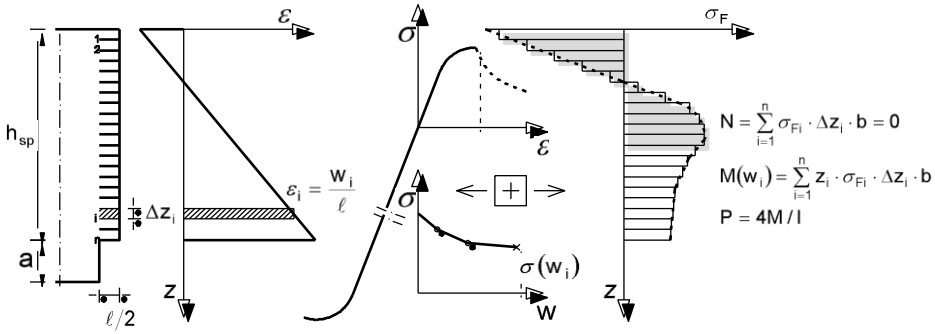


Figure 5.6-4: Inverse analysis of beam in bending performed to obtain stress – crack opening relation

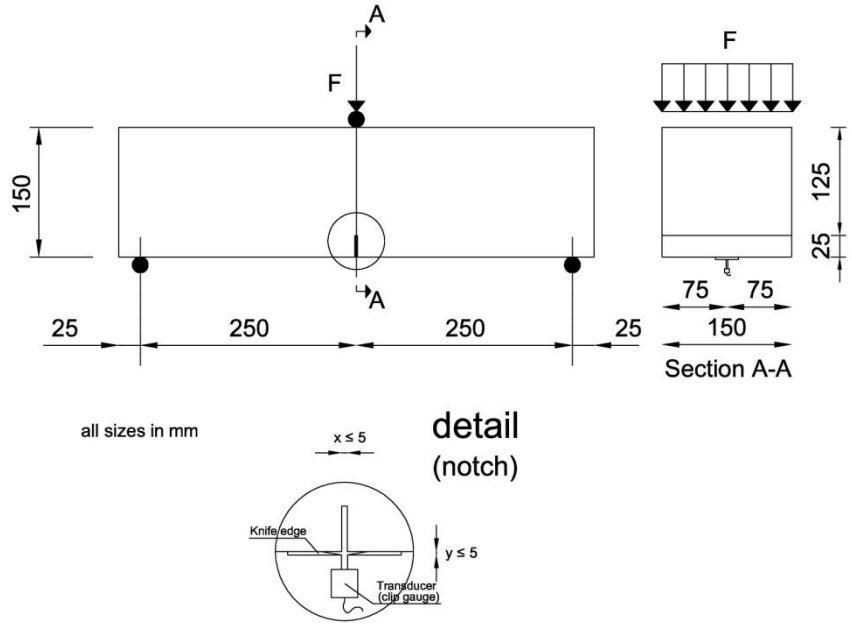


Figure 5.6-5: Inverse analysis of beam in bending performed to obtain stress – crack opening relation

Nominal properties can be determined by performing a 3-point bending test on a notched beam according to EN 14651 (Figure 5.6-5). The diagram of the applied force (F) versus the deformation shall be produced (see Figure 5.6-6). The deformation shall be expressed in terms of Crack Mouth Opening Displacement (*CMOD*)

Residual flexural tensile strength parameters, f_{Rj}, are evaluated from the *F-CMOD* relationship.

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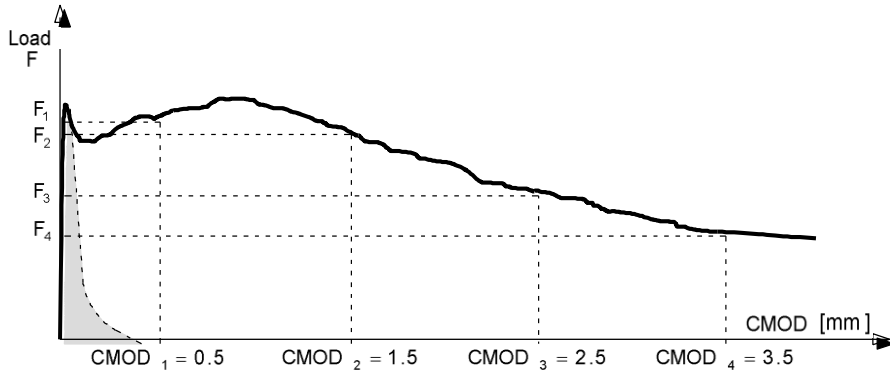


Figure 5.6-6: Inverse analysis of beam in bending performed to obtain stress – crack opening relation

In case of organic and natural fibres, post-cracking long term behaviour can be affected by an additional creep of the fibres themselves.

For high fibre contents, strain hardening materials can be obtained. To guarantee the hardening in tension, the tensile behaviour must be identified by means of uniaxial tension tests carried out on unnotched specimens.

In order to identify the constitutive law, residual flexural tensile strengths f_{Rj} are used.

$$f_{R,j} = \frac{3F_j l}{2b h_{sp}^2}$$

where:

f_{Rj} is the residual flexural tensile strength corresponding with $CMOD = CMOD_j$

F_j is the load corresponding with $CMOD = CMOD_j$

l is the span length [mm];

b is the width of the specimen [mm];

h_{sp} is the distance between the notch tip and the top of the specimen [mm], 125 mm.

Long term behaviour of cracked FRC under tension has to be properly taken into account for those materials whose long term performance is affected by creep and/or creep rupture (see subclause 5.6.5.)

Other tests can be accepted if correlation factors with the parameters of EN 14651 are proven.

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For structural applications with normal and light-strength concrete the material classification is based on the post-cracking residual strength.

For concrete with higher compressive strength (including UHPFRC) special design rules may be adopted.

For instance, a material denoted as "3b" has a strength f_{R1k} ranging between 3 and 4 MPa and the ratio f_{R3k}/f_{R1k} ranging between 0.7 and 0.9.

The Limit of Proportionality f_L, as defined in EN 14651, can be determined by applying the following equation:

$$f_L = \frac{3F_L l}{2bh_{sp}^2}$$

Apart from these requirements on the material for structural use, more requirements on the structure level are to be found in subclause 7.7.2 of the Model Code.

5.6.3 Classification

In order to classify the post-cracking strength of FRC a linear elastic behaviour can be assumed, by considering the characteristic residual strengths significant for service (f_{R1k}) and ultimate (f_{R3k}) conditions.

Post-cracking residual strength can be classified by using two parameters, namely f_{R1k} (representing the strength interval) and a letter a, b, c or d (representing the ratio f_{R3k}/f_{R1k}).

The strength interval is defined by two subsequent numbers in the series:

1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 6.0, 7.0, 8.0 [MPa]

while the letters *a*, *b*, *c*, *d* correspond to the residual strength ratios:

a if $0.5 \leq f_{R3k}/f_{R1k} \leq 0.7$

b if $0.7 \leq f_{R3k}/f_{R1k} \leq 0.9$

c if $0.9 \leq f_{R3k}/f_{R1k} \leq 1.1$ (5.6-1)

d if $1.1 \leq f_{R3k}/f_{R1k} \leq 1.3$

e if $1.3 \leq f_{R3k}/f_{R1k}$

The designer has to specify the class, the residual strength ratio and the material of the fibre.

Fibre reinforcement can substitute (also partially) conventional reinforcement at ultimate limit state if the following relationships are fulfilled:

$f_{R1k}/f_{Lk} > 0.4$ (5.6-2)

$f_{R3k}/f_{R1k} > 0.5$ (5.6-3)

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The rigid-plastic model takes the static equivalence into account as shown in Figure 5.6-8, i.e. f_{FTu} results from the assumption that the whole compressive force is concentrated in the top fibre of the section:

$$M_u = \frac{f_{R3} b h_{sp}^{\ 2}}{6} = \frac{f_{FTu} b h_{sp}^{\ 2}}{2}$$

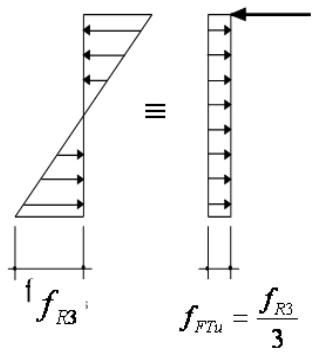


Figure 5.6-8: Simplified model adopted to compute the ultimate tensile strength in uniaxial tension f_{FTu} by means of the residual nominal bending strength f_{R3}

The equation for f_{FTu} and w_u=$CMOD_3$ is obtained, from the rotational equilibrium, with reference to ULS, when a stress block in tension along the section is taken into account as shown in Figure 5.6-9b.

5.6.4 Constitutive laws

The stress-crack opening law in uniaxial tension is defined for the post-cracking range.

Two simplified stress-crack opening constitutive laws may be deduced from the bending test results: a plastic rigid behaviour, or a linear post-cracking behaviour (hardening or softening) as schematically shown in Figure 5.6-7, where f_{Fts} represents the serviceability residual strength, defined as the post-cracking strength for serviceability crack openings, and f_{Ftu} represents the ultimate residual strength.

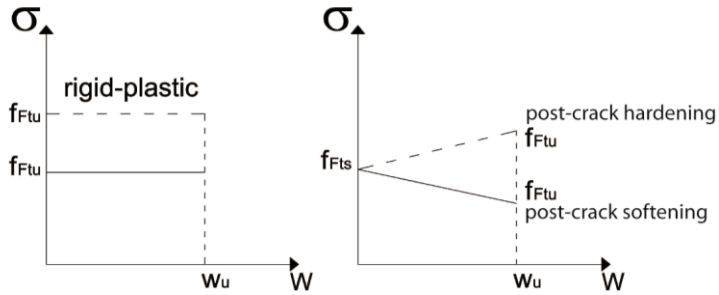


Figure 5.6-7: Simplified post-crack constitutive laws: stress-crack opening (continuous and dashed lines refer to softening and hardening post-crack behaviour respectively).

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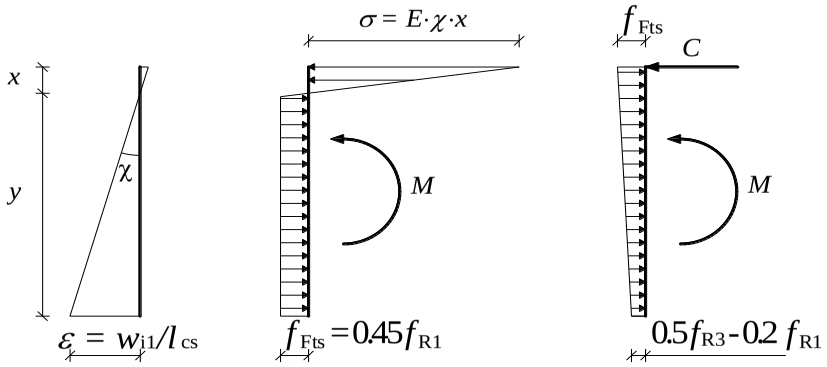


Figure 5.6-9: Simplified model adopted to compute the ultimate tensile strength in uniaxial tension f_{FTu} by means of the residual nominal bending strength f_{R3}

The equation for f_{FTu} and $w_u \neq CMOD_3$ is obtained by considering a linear constitutive law between points with abscissa $CMOD_1$ and $CMOD_3$, up to the point with abscissa w_u (Figure 5.6-10).

The stress value corresponding to the crack opening $CMOD_1$ is determined from equilibrium, with the assumption that the compressive stress distribution is linear (Figure 5.6-9a) and that the tensile behaviour is elasto-plastic until a crack opening displacement corresponding to the serviceability limit state ($CMOD_1$):

$$M(CMOD_1) = \frac{f_{R1} b h_{sp}^{\ 2}}{6}$$

The variability introduced in the results by elastic modulus is here negligible and a common value is assumed.

The stress value corresponding to the crack opening $CMOD_3$ is determined from equilibrium, with the assumption that the compressive stress

Rigid-plastic model

The rigid-plastic model identifies a unique reference value, f_{Ftu}, based on the ultimate behaviour. Such a value is determined as:

$$f_{Ftu} = \frac{f_{R3}}{3} \tag{5.6-4}$$

Linear model

The linear model identifies two reference values, f_{Fts} and f_{Ftu}.

They have to be defined through residual values of flexural strength by using the following equations:

$$f_{Fts} = 0.45 f_{R1} \tag{5.6-5}$$

$$f_{Ftu} = f_{Fts} - \frac{w_u}{CMOD_3}(f_{Fts} - 0.5 f_{R3} + 0.2 f_{R1}) \geq 0 \tag{5.6-6}$$

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resultant is applied on the extrados chord (Figure 5.6-9b) and that the tensile behaviour is rigid-linear:

$$M(CMOD_3) = \frac{f_{R3} b h_{sp}^{\ 2}}{6}$$

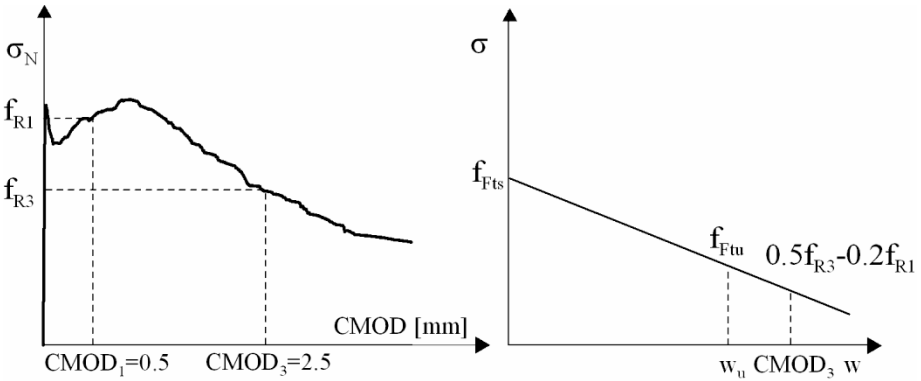


Figure 5.6-10: Typical results from a bending test on a softening material (a); linear post-cracking constitutive law (b).

The limit value w_u applies particularly for design purposes.

For numerical analyses more advanced constitutive laws are recommended, including first crack tensile strength.

w_u is the maximum crack opening accepted in structural design. Its value depends on the ductility required.

When considering softening materials, the definition of the stress-strain law is based on the identification of the crack width and on the corresponding structural characteristic length, l_{cs} of the structural element. Thus, the strain can be assumed equal to:

$$\varepsilon = w / l_{cs} \qquad (5.6\text{-}7)$$

In elements with conventional reinforcement (rebars), the characteristic length, l_{cs}, may be evaluated as:

$$l_{cs} = \min\{s_{rm}, y\} \qquad (5.6\text{-}8)$$

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where:

s_{rm} is the mean distance value between cracks;

y is the distance between neutral axis and tensile side of the cross section, evaluated in the elastic cracked phase assuming no tensile strength of the fibre reinforced concrete, and for a load configuration corresponding to the serviceability state of crack opening and crack spacing.

The ultimate tensile strength f_{Ftu} in the linear model depends on the required ductility that is related to the allowed crack width. The ultimate crack width can be calculated as $w_u = l_{cs} * \varepsilon_{Fu,}$ by assuming ε_{Fu} equal to 2% for variable strain distribution along the cross section and 1% for only tensile strain distribution along the cross section. The maximum crack width may not exceed 2.5 mm.

In sections without traditional reinforcement under bending or under combined tensile – flexural and compressive – flexural forces with resulting force external to the section, $y = h$ is assumed. The same assumption can be taken for slabs.

When considering strain hardening materials, the same ε_{Fu}, by assuming ε_{Fu} equal to 2% for variable strain distribution along the cross section and 1% for only tensile strain distribution along the cross section should be considered.

The material is regarded as strain hardening, if it shows a hardening behaviour in tension up to a ε_{Fu} =1%.

5.6.5 Stress-strain relationship for SLS

For the ULS the constitutive laws as described in subclause 5.6.4 shall be applied.

For the SLS the same constitutive relationship adopted for plain concrete in uniaxial tension is used up to the peak strength f_{ct}. In the post-cracking stage, a bilinear relation applies (Figure 5.6-10). For softening material, the residual strength (second branch) is defined by two points corresponding to (ε_{SLS}, f_{Ftsd}) and (ε_{ULS}, f_{Ftud}) where:

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The second branch suggested in the pre-peak constitutive relationship corresponds to that for plain concrete until an intersection between the residual post-cracking behaviour and the unstable crack propagation branch for plain concrete is available.

When this condition does not apply, a new second branch is proposed, as shown in Figure 5.6-11b.

A reduced safety factor may be adopted for improved control procedures.

$$\varepsilon_{SLS} = CMOD_1/l_{cs} \quad (5.6\text{-}9)$$

$$\varepsilon_{ULS} = w_u/l_{cs} = \min(\varepsilon_{Fu}, 2.5/l_{cs} = 2.5/y) \quad (5.6\text{-}10)$$

with ε_{Fu} = 2% for variable strain distribution along the cross section and 1% for only tensile strain distribution along the cross section, see subclause 5.6.4.3.

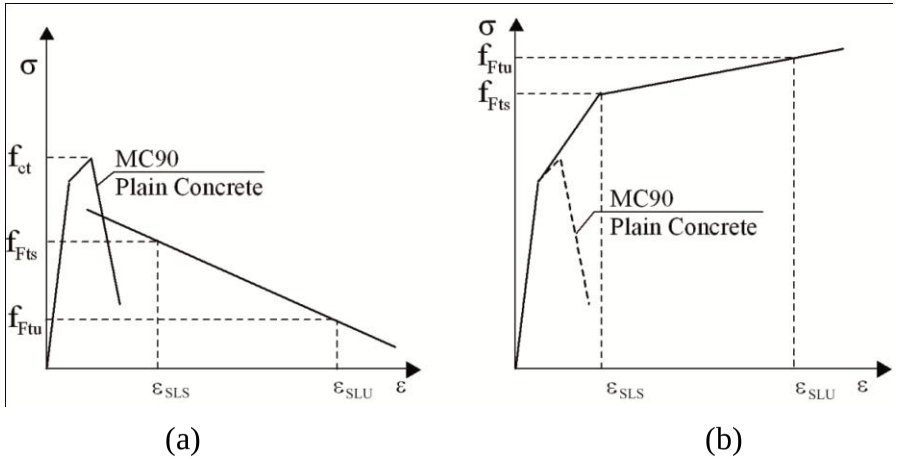


Figure 5.6-11: Stress - strain relations for the SLS for softening (a) and hardening (b) behaviour of FRC

5.6.6 Partial safety factors for ULS

For ULS-design the fibre contribution follows from (see Fig. 5.6-7):

$$f_{tsd} = f_{tsk} / \gamma_F$$

and

$$f_{tud} = f_{tuk} / \gamma_F$$

The recommended values for the partial safety factors are given in Table 5.6-1.

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The behaviour observed in the standard tests can deviate substantially (beneficial and non-beneficial) from the behaviour of the corresponding FRC in the structural element or structure. Thus the manufacturing method and the concrete consistency should be taken into account by the designer.

Special tests can be used to determine the effect of fibre orientation due to casting and compaction in real structural elements, by using structural specimens which better reproduce the material in the structural elements.

Table 5.6-1: Partial Safety Fact

Material	**Partial Safety factors**
FRC in compression	As plain concrete
FRC in tension (limit of linearity)	As plain concrete
FRC in tension (residual strength)	γ_F=1.5

For serviceability limit states (SLS), the partial factors should be taken as 1.0

5.6.7 Orientation factor

In general, isotropic fibre distribution is assumed so that the fibre orientation factor K = 1.0.

For favourable effects, an orientation factor K < 1.0 may be applied if experimentally verified.

For unfavourable effects, an orientation factor K > 1.0 must be experimentally verified and applied.

The values f_{Ftsd} and f_{Ftud} should then be modified into:

$f_{Ftsd,mod} = f_{Ftsd} / K$

$f_{Ftud,mod} = f_{Ftud}/K$

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6 Interface characteristics

6.1 Bond of embedded steel reinforcement

Bond is the term used to denote the interaction and transfer of force between reinforcement and concrete. Bond influences performance of concrete structures in several ways. At the serviceability limit state, bond influences width and spacing of transverse cracks, tension stiffening and curvature. At the ultimate limit state, bond is responsible for strength of end anchorages and lapped joints of reinforcement, and influences rotation capacity of plastic hinge regions.

6.1.1 Local bond-slip relationship

Under well-defined conditions, it is possible to consider that there is an average 'Local bond' versus 'Local slip' relationship, for short anchorage lengths, statistically acceptable.

For monotonic loading the bond stresses between concrete and reinforcing bar for pull-out and splitting failure can be calculated as a function of the relative displacement s according to Eqs. (6.1-1) to (6.1-4) (see Figure 6.1-1).

$$\tau_0 = \tau_{max} \, (s/s_1)^{\alpha} \quad \text{for} \quad 0 \le s \le s_1 \qquad (6.1\text{-}1)$$

$$\tau_0 = \tau_{max} \quad \text{for} \quad s_1 \le s \le s_2 \qquad (6.1\text{-}2)$$

$$\tau_0 = \tau_{max} \quad (\tau_{max} - \tau_f) \, (s-s_2)/(s_3-s_2) \quad \text{for} \quad s_2 \le s \le s_3 \qquad (6.1\text{-}3)$$

$$\tau_f \quad \text{for} \quad s_3 < s \qquad (6.1\text{-}4)$$

Explicit consideration of the influence of bond on performance at the serviceability limit state and on rotation capacity is not normally required for members reinforced with conventional plain or deformed steel bars. The influence is instead represented by a modification of the tension stiffening from surrounding concrete.

The bond stress-slip relationship depends on a considerable number of influencing factors like rib geometry (related rib area), concrete strength, position and orientation of the bar during casting, state of stress, boundary conditions and concrete cover.

Therefore the bond stress-slip curves for confined and unconfined concrete, presented in Figure 6.1-1, can be considered as statistical mean curves, applicable as an average formulation for a broad range of cases. Further reliability handling would be needed to derive design bond stress-slip curves.

The first curved part refers to the stage in which the ribs penetrate into the mortar matrix, characterized by local crushing and micro-cracking. The horizontal level occurs only for confined concrete, referring to advanced crushing and shearing off of the concrete between the ribs. This level represents a residual bond capacity, which is maintained by virtue of a large concrete cover or a minimum transverse reinforcement, keeping a certain degree of integrity intact. The decreasing branch refers to the reduction of bond resistance due to shearing of the concrete corbels between the ribs. In case of unconfined concrete splitting failure occurs which is reflected by a sudden drop of the bond strength before the horizontal level is reached.

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With regard to the generation of bond stresses, the following considerations apply.

Reinforcement and concrete have the same strain ($\varepsilon_s = \varepsilon_c$) in those areas of the structure where the steel is under compression and in uncracked parts of the structure under tension.

In cracked cross-sections the tension forces in the crack are transferred by the reinforcing steel. In general, the absolute displacements of the steel u_s and of the concrete u_c between two cracks or along the transmission length l_t are different.

Due to the relative displacement $s = u_s - u_c$ bond stresses are generated between the concrete and the reinforcing steel. The magnitude of these bond stresses depends predominantly on the surface of the reinforcing steel, the slip s, the concrete strength f_{ck} and the position of the reinforcing steel during concreting. Between cracks or along the transmission length l_t a part of the tension force of the reinforcing steel, acting in the crack, is transferred into the concrete by bond (tension stiffening effect).

The local decrease of the relative displacement along the transmission length l_t, is characterized by the strain difference:

$$ds/dx = \varepsilon_s - \varepsilon_c$$

Depending on the selection of the coefficient $\alpha (0 \leq \alpha \leq 1)$ in Eq. (6.1-1) all usual forms of a bond stress-slip relationship can be modelled, starting from a bond characteristic with a constant stress ($\alpha = 0$) up to a bond stress-slip relationship with linear increasing bond stress ($\alpha = 1$).

The parameters given in Table 6.1-1 are valid for ribbed reinforcing steel with a related rib area $A_{sr} \approx A_{sr,min}$ according to relevant international standards. Depending on the failure mode, pull-out or splitting, different parameters are applied. In order to take other bond conditions into consideration a further distinction of the parameters is made.

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Table 6.1-1: Parameters for defining the mean bond stress-slip relationship of deformed bars (acc. to Eqs. (6.1-1) to (6.1-4))

	1	2	3	4	5	6
	Pull-Out (PO)		**Splitting (SP)**			
	$\varepsilon_s < \varepsilon_{s,y}$		$\varepsilon_s < \varepsilon_{s,y}$			
	Good bond cond.	**All other bond cond.**	**Good bond cond.**		**All other bond cond.**	
			unconfined	**stirrups**	**unconfined**	**stirrups**
τ_{max}	$2.5\sqrt{f_{ck}}$	$1.25\sqrt{f_{ck}}$	$7.0\cdot\left(\frac{f_{ck}}{20}\right)^{0.25}$	$8.0\cdot\left(\frac{f_{ck}}{20}\right)^{0.25}$	$5.0\cdot\left(\frac{f_{ck}}{20}\right)^{0.25}$	$5.5\cdot\left(\frac{f_{ck}}{20}\right)^{0.25}$
s_1	1.0mm	1.8mm	$s(\tau_{max})$	$s(\tau_{max})$	$s(\tau_{max})$	$s(\tau_{max})$
s_2	2.0mm	3.6mm	s_1	s_1	s_1	s_1
s_3	c_{clear} [1)]	c_{clear} [1)]	$1.2s_1$	$0.5c_{clear}$ [1)]	$1.2s_1$	$0.5c_{clear}$ [1)]
α	0.4	0.4	0.4	0.4	0.4	0.4
τ_f	$0.40\,\tau_{max}$	$0.40\,\tau_{max}$	0	$0.4\,\tau_{max}$	0	$0.4\,\tau_{max}$

[1)] c_{clear} is the clear distance between ribs

The values in Table 6.1-1, columns 1 and 2 (pull-out failure) are valid for well confined concrete (concrete cover ≥ 5 Ø, clear spacing between bars ≥ 10Ø).

The values in Table 6.1-1, columns 3 to 6 (splitting failure) are valid for Ø ≤ 20mm, $c_{max}/c_{min} = 2.0$, $c_{min} = Ø$ and $K_{tr} = 2\%$ in case of stirrups. For definition of K_{tr} see Eq. (6.1-5).

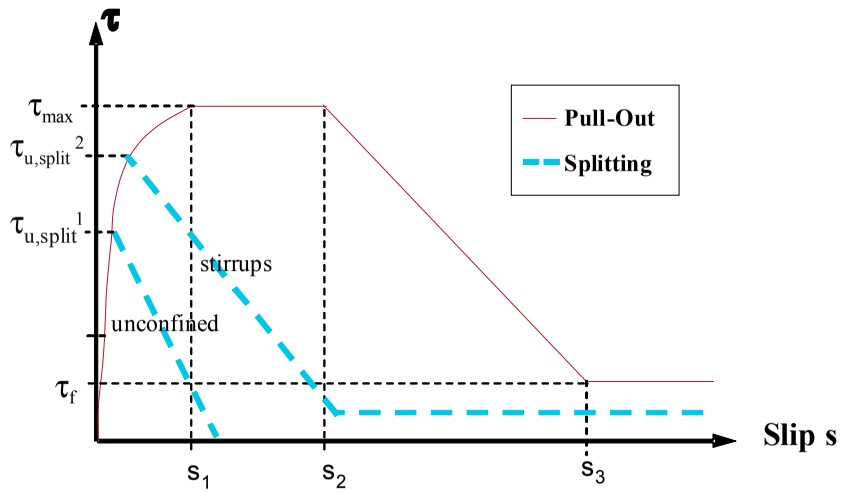


Figure 6.1-1: Analytical bond stress-slip relationship (monotonic loading)

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$$\tau_{u,split} = \eta_2 \cdot 6.54 \cdot \left(\frac{f_{ck}}{20}\right)^{0.25} \cdot \left(\frac{20}{\varnothing}\right)^{0.2} \cdot \left[\left(\frac{c_{min}}{\varnothing}\right)^{0.33} \cdot \left(\frac{c_{max}}{c_{min}}\right)^{0.1} + 8K_{tr}\right] \qquad (6.1\text{-}5)$$

where:

η_2 = 1.0 for good bond conditions,

= 0.7 for all other bond conditions

f_{ck} is characteristic cylinder concrete compressive strength (N/mm²)

Ø is diameter of the bar (mm)

c_{min} = min{c_x,c_y,c_{si}}

c_{max} = max{c_x, c_{si}}

and

K_{tr} = $n_1 A_{sv}/(n_b \varnothing s_v)$

where:

n_1 is number of legs of confining reinforcement at a section

A_{sv} is cross sectional area of one leg of a confining bar [mm²]

s_v = longitudinal spacing of confining reinforcement [mm]

n_b = number of anchored bars or pairs of lapped bars

Ø is diameter of the anchored bar or of the smaller of a pair of lapped bars [mm]

For those parts of the reinforcing bar which are at a distance $x \leq 2$ Ø from a transverse crack, the bond stress τ and the slip s are to be reduced by the factor λ where:

$$\lambda = 0.5\, x/\varnothing \leq 1 \qquad (6.1\text{-}6)$$

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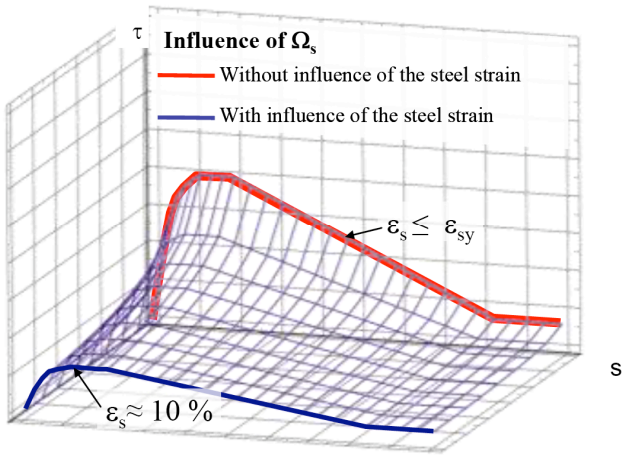


Figure 6.1-2: Influence of steel strains on local bond-stress slip relationship in case of pull-out failure

The bond stress-slip curve is considerably influenced by reinforcement yielding and by transverse pressure. Taking into account these phenomena the bond stress according to Eqs. (6.1-1) to (6.1-4) should be modified by the factor Ω_y in case of bar yielding and $\Omega_{p,tr}$ in case of lateral pressure:

$$\tau_{bm} = \tau_0 \cdot \Omega_y \cdot \Omega_{p,tr} \qquad (6.1\text{-}7)$$

where:

τ_{bm} is bond stress according to the modified bond stress-slip curve

τ_0 is bond stress according to the bond stress – slip curve defined by Eq. (6.1-1) to (6.1-4)

If yielding of reinforcement occurs along the embedment length, the corresponding reduction of the bond stress is given by the factor Ω_y according to Eq. (6.1-8).

$$\Omega_y = 1.0 \qquad \text{for } \varepsilon_s \le \varepsilon_{sy} \qquad (6.1\text{-}8a)$$

$$\Omega_y = 1.0 - [0.85 \cdot (1 - e^{-5a^b}) \qquad \text{for } \varepsilon_{sy} < \varepsilon_s \le \varepsilon_{su} \qquad (6.1\text{-}8b)$$

with

$$a = \frac{\varepsilon_s - \varepsilon_{sy}}{\varepsilon_{su} - \varepsilon_{sy}} \; ; \quad b = \left[2 - \frac{f_t}{f_y} \right]^2 \qquad (6.1\text{-}8c)$$

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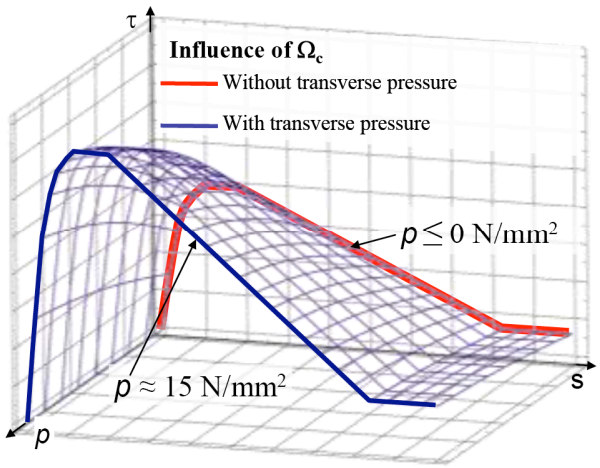


Figure 6.1-3: Influence of transverse pressure on local bond-stress slip relationship in case of pull-out failure

The parameters given in Table 6.1-2 are valid for smooth reinforcing steel, depending on the main influencing factors: roughness of the bar surface, bond conditions and concrete strength. They are valid for confined and unconfined concrete. They are applicable only in loading states for which the concrete is not subjected to lateral tension, in the elastic range of the reinforcement and for those parts of the reinforcing bar with the distance x > 2Ø from a transverse crack. For those parts of the reinforcing bars with a distance x ≤ 2Ø from a transverse crack, the bond stress τ and the slip s are to be reduced by the factor λ according to Eq. (6.1-6)

If transverse pressure is present the bond stress slip curve for pull-out failure should be modified by the factor $\Omega_{p,tr}$ according to Eq. (6.1-9).

$$\Omega_{p,tr} = 1,0 \qquad \text{for } \sigma_c \geq 0 \qquad (6.1\text{-}9a)$$

$$\Omega_{p,tr} = 1,0 - \tanh\left[0.2 \cdot \left(\frac{\overline{\sigma}_{\perp}}{0,1 \cdot f_c}\right)\right] \qquad \text{for } \sigma_c \leq 0 \qquad (6.1\text{-}9b)$$

where $\overline{\sigma}_{\perp}$ is the mean compressive stress (orthogonal to the bar axis) averaged over a volume around the bar with a diameter of 3 Ø.
f_c is the mean concrete compressive strength.

If there is any tension stress around the bar then Ω_c=1,0.

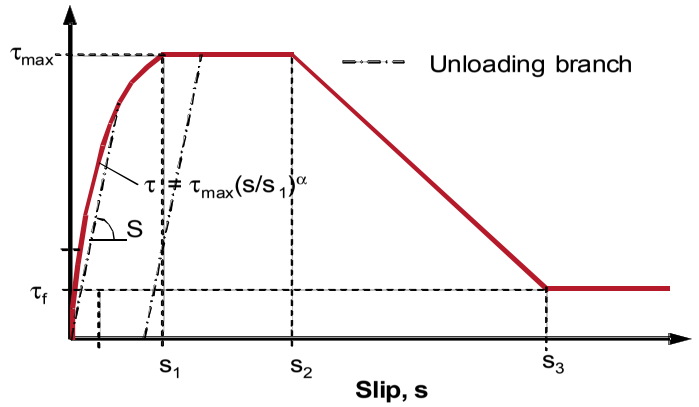


Figure 6.1-4: Unloading branch of the τ-s relationship

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Table 6.1-2: Parameters for defining the bond stress-slip relationship of smooth bars (according to Eqs (6.1.-1) to (6.1.-4))

	Cold drawn wire		Hot rolled bars	
	Good bond conditions	All other bond cond.	Good bond conditions	All other bond cond.
$s_1 = s_2 = s_3$	0.01mm	0.01mm	0.1mm	0.1mm
α	0.5	0.5	0.5	0.5
$\tau_{max}=\tau_f$	$0.1\sqrt{f_{ck}}$	$0.05\sqrt{f_{ck}}$	$0.3\sqrt{f_{ck}}$	$0.15\sqrt{f_{ck}}$

The parameters given in Tables 6.1-1 and 6.1-2 are mean values.

It has to be kept in mind that the scatter of different test series is considerable, especially for small values of slip. For a given value of the slip the coefficient of variation of the bond stresses may amount to approx. 30%. The scatter is due to the use of different test specimens and the resulting differences in the state of stress in the concrete surrounding the reinforcing bar, to the different measuring techniques, and to the different loading and deformation velocities. The heterogeneity of the concrete and the geometry of the reinforcing bars (related rib area, diameters) also have a significant influence on the τ - s relationship. The designer should take account of this scatter as far as possible, at least in the cases where a more accurate design is necessary.

Rules for serviceability behaviour have been derived for ribbed bars with a relative rib area f_r within the range 0.05-0.07. More highly ribbed bars may be capable of developing higher bond stiffness, while coatings may cause a reduction. Coefficients β representing the influence of bond in Eqs. (7.6-5) and (7.6-17) in clause 7.6 may be modified in proportion to measured bond stiffness.

In the absence of test data, the β coefficient for plain round and for epoxy coated ribbed bars should be taken as 50% of that for a 'normal' ribbed bar.

The unloading branch of the bond stress-slip relationship is linear and valid for the increasing and horizontal part of the diagram. The slope S (see Figure 6.1-4) is independent of the slip value s, and has an average value of $S = 200$ N/mm³.

6.1.2 Influence on serviceability

The influence of bond on crack control and tension stiffening may be estimated from the secant modulus of local bond-slip behaviour measured in accordance with RILEM pullout test procedures at a free end slip of 0.01mm.

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Only one of the additional components may be considered to contribute to anchorage at a particular location.

Note that the resistance of two components acting in combination will generally be less than the sum of their resistances determined individually. The rules provided here make allowance for interaction effects.

6.1.3 Anchorage and lapped joints of reinforcement

Reinforcement may be anchored by bond alone, or by a combination of bond along the straight portion of a bar together with a resistance provided by one of:

- a hook or bend (but only for bars in tension and compression bars within a footing);
- welded transverse bar(s);
- a head welded to the end of the bar;
- in the case of bars in compression, by the bearing of the end of the bar on the concrete.

Lapped joints may alternatively be made by welding or by mechanical couplers.

The stress in reinforcement to be anchored by bond over the straight portion of a bar is:

$$\sigma_{sd} = \alpha_1 f_{yd.} - F_h / A_b \qquad (6.1\text{-}10)$$

where:

$\alpha_1 = A_{s,cal} / A_{s,ef}$

$A_{s,cal}$ is the calculated area of reinforcement required by the design

$A_{s,ef}$ is the area of reinforcement provided

A_b is the cross sectional area of the bar considered

F_h is the force developed by the other measures listed above. $F_h = 0$ in the case of straight tension bars.

Minimum lap and anchorage lengths are specified in subclause 6.1.3.3.

6.1.3.1 Basic bond strength

Bond strength $f_{b,0}$ is considered as an average stress on the nominal surface of the bar over the bond length l_b. The basic bond strength $f_{b,0}$ is:

$$f_{b,0} = \eta_1 \eta_2 \eta_3 \eta_4 (f_{ck} / 20)^{0.5} / \gamma_c \qquad (6.1\text{-}11)$$

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Bond strength for ribbed bars in 'good' casting position is derived from the following semi-empirical expression for reinforcement stress f_{stm}, which has been calibrated against results from over 800 tests

$$f_{stm} = 54\left(\frac{f_c}{20}\right)^{0.25}\left(\frac{20}{\phi}\right)^{0.2}\left(\frac{l_b}{\phi}\right)^{0.55}\left[\left(\frac{c_{\min}}{\phi}\right)^{0.33}\left(\frac{c_{\max}}{c_{\min}}\right)^{0.1} + 8K_{tr}\right] \qquad (6.1\text{-}12)$$

with $f_{stm} \le f_y$ and $f_{stm} \le 2.5\sqrt{f_c}.4\left(l_b/\phi\right)$

The expression is valid for : 15MPa $< f_c <$ 110MPa, $0.5 < c_{min} / Ø < 3.0$, $1.0 < c_{max} / c_{min} < 5.0$ and $K_{tr} < 4\%$.

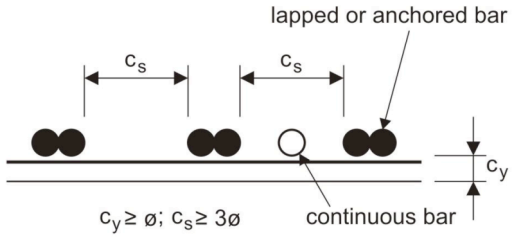
f_{stm} is a mean value, and may not be directly used in design.

Eq. (6.1-12) has been derived from tests on bars with a relative rib area f_r within the range 0.05-0.07. More highly ribbed bars may be capable of developing higher bond strengths. The appropriate η_1 value should be determined from tests.

Bond of bars that are not in a 'good' casting position may be impaired by consolidation of the fluid concrete under the bar. Measures to minimise plastic settlement cracking will also be effective in minimising the reduction in bond where the casting position is not classified as 'good'.

Eq. (6.1-12) is valid where bar spacing and/or transverse reinforcement complies with conditions (c), (d) and (e), and either condition (a) or (b) below.

(a) Minimum cover not less than one bar diameter, clear spacing between the anchored or lapped bars not less than 3 bar diameters.



where :

η_1 is a coefficient taken as 1.8 for ribbed bars (including galvanised and stainless reinforcement), 1.4 for fusion bonded epoxy coated ribbed bars, 0.90 for plain bars and 1.1 for indented bars

η_2 represents the casting position of the bar during concreting: η_2 =1.0 when good bond conditions are obtained, as for:

- all bars with an inclination of 45° - 90° to the horizontal during concreting and
- all bars with an inclination less than 45° to the horizontal which are up to 250mm from the bottom or at least 300mm from the top of the concrete layer during concreting.

(but see also 'special circumstances' section later)

η_2 = 0.7 for all other cases where ribbed bars are used, or

η_2 = 0.5 where plain bars are used

η_3 represents the bar diameter.

η_3 = 1.0 for Ø ≤20mm

$\eta_3 = (20/ Ø)^{0.3}$ for Ø >20mm (Ø in mm)

η_4 represents the characteristic strength of steel reinforcement being anchored or lapped.

η_4 = 1.0 for f_{yk}=500MPa

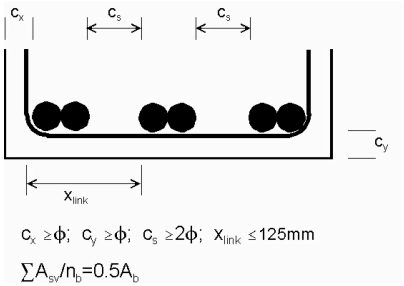
η_4 = 1.2 for f_{yk}=400MPa

η_4 = 0.85 for f_{yk}=600MPa

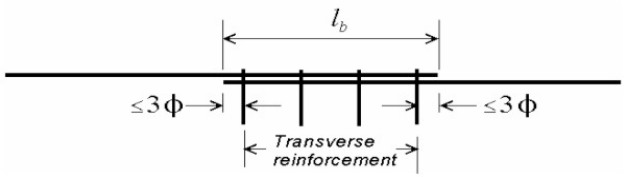
η_4 = 0.75 for f_{yk}=700MPa

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(b) Minimum cover not less than one bar diameter, clear spacing between bars of not less than 2 bar diameters, an area of transverse reinforcement of at least 50% of the total cross sectional area of the bars being lapped or anchored, and no lapped or anchored bar more further than the lesser of 5Ø or 125 mm from a vertical leg of a link.



(c) At least 1 piece of transverse reinforcement should be positioned within a lap no further than 3Ø from the end of the bars.



(d) No more than 50% of bars are lapped at the same section. *Note: Condition (d) does not apply to distribution reinforcement*

(e) The section is not subjected to applied tensile stress (other than that generated by the splitting action of bond) on a plane passing through the bars.

Where these conditions are not satisfied, more detailed guidance may be found in Structural Concrete, Journal of the *fib*, "Derivation of design rules for embedded reinforcement in the New Model Code" (to be published).

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Transverse pressure has a strongly beneficial influence on bond strength where it restrains a splitting failure mode. Once confinement from cover, transverse reinforcement and transverse pressure is sufficient to restrain splitting, the rate of increase reduces.

Provided minimum detailing provisions are satisfied, α_2 and α_3 may conservatively be taken as 1.0.

It will usually be advisable to include the influence of transverse pressure when considering anchorage of reinforcement in deep beams and corbels.

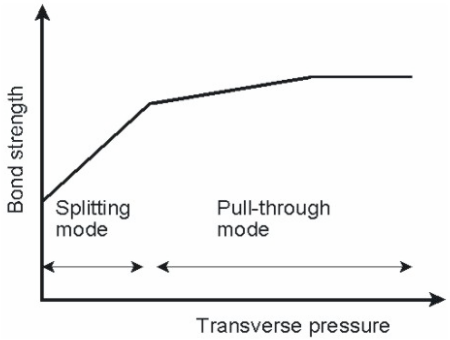


Figure 6.1-5: Influence of transverse pressure

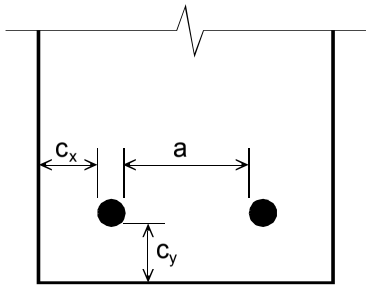


Figure 6.1-6: Notation for bar spacing and cover: straight bars

6.1.3.2 Design bond strength

The design ultimate bond strength f_{bd} of ribbed bars may be increased above the basic value where concrete cover, bar spacing or transverse reinforcement are above their respective minima or where the bar is subjected to transverse compression:

$$f_{bd} = (\alpha_{2+}\alpha_3).f_{b,0} + 2p_{tr} < 2.0\, f_{b,0} + 0.4p_{tr} < 2.5\sqrt{f_{ck}} \qquad (6.1\text{-}13)$$

where:

α_2 and α_3 represent the influence of passive confinement from cover (α_2) and from transverse reinforcement (α_3) in excess of their respective permissible minima.

p_{tr} is the mean compression stress perpendicular to the potential splitting failure surface at the ultimate limit state; where transverse compression perpendicular to the bar axis acts over a portion of the bond length, bond strength may be increased over that portion.

Passive confinement from cover: straight reinforcing bars

Ribbed bars: $\alpha_2 = (c_{min} / Ø))^{0.5}.(c_{max} / c_{min})^{0.15}$

Epoxy coated bars : $\alpha_2 = (c_{min} / Ø)^{0.7}.(c_{max} / c_{min})^{0.15}$

$0.5 \le c_{min} / Ø \le 3$, $c_{max} / c_{min} \le 5$

α_2 is taken as 1.0 for plain surface bars.

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Minimum transverse reinforcement at laps and anchorages is to be provided to prevent a brittle mode of anchorage or lap failure. Reinforcement provided for other purposes (e.g. for shear resistance) may be included in computation of $\Sigma A_{st,min}$.

At least 1 piece of transverse reinforcement should be positioned within the lap or anchorage length no further than 3Ø from the end of the bar.

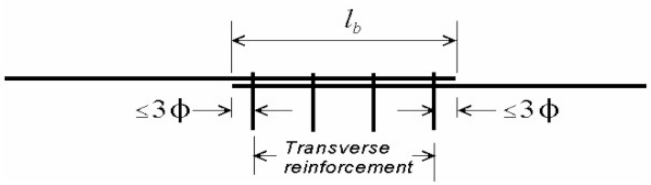


Figure 6.1-7: Location of links near ends of lap or anchorage

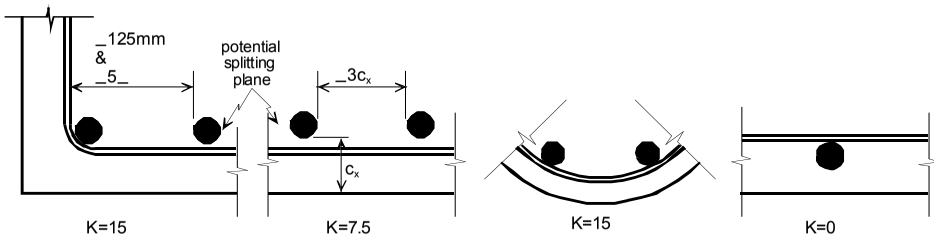


Figure 6.1-8: Transverse reinforcement factor k

A minimum quantity of transverse reinforcement $\Sigma A_{st,min}$ is to be provided at laps and anchorages in beams and columns of structural significance, and in slabs or walls where the spacing of anchored or lapped bars is less than 3 times c_x. Minimum transverse reinforcement should also be provided in slabs of Grades C60/75 and above. $\Sigma A_{st,min}=0$ in other cases.

$$\sum A_{st,\min} = \frac{n_l A_{sv} l_b}{n_b s_v} \geq 0.5\alpha_1 \sum A_b \qquad (6.1\text{-}14)$$

Passive confinement from transverse reinforcement

$$\alpha_3 = k\, K_{tr} \qquad (6.1\text{-}15)$$

where:

$K_{tr} = n_l.A_{sv}/(n_b\, Ø\, s_v)$ is the density of transverse reinforcement.

n_l is the number of legs of confining reinforcement at a section

A_{st} is the cross sectional area of one leg of a confining bar (mm^2.)

s_v is the longitudinal spacing of confining reinforcement (mm)

n_b is the number of anchored bars or pairs of lapped bars

Ø is the diameter of the anchored bar or of the smaller of a pair of lapped bars (mm).

k is an effectiveness factor dependent on the reinforcement detail.

k = 15 where the legs of a link are perpendicular to the splitting plane and for helical confining reinforcement, provided no anchored bar or pair of lapped bars are further than 150m from where the leg crosses the splitting plane

k = 7.5 where bars are confined by straight bars within the cover thickness, provided clear spacing is at least 3 times cover.

k = 0 in other circumstances

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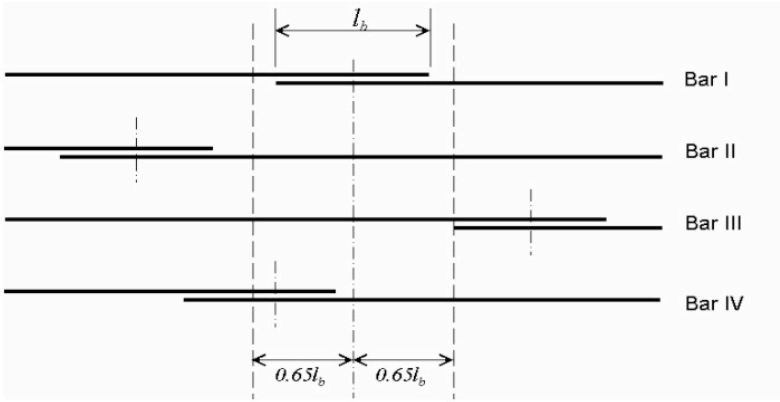


Figure 6.1-9: l_b is the lap length of Bar I to be designed. II and III are situated outside the section. Lap IV is situated within the section. Percentage = 50% and α_4=1.0.

For transverse distribution reinforcement α_4 can be taken as 0.75.

Transverse tension

Design bond strengths calculated by Eq.(6.1-13) should be reduced by 30% where the anchorage length lies wholly within a zone in which concrete is cracked parallel to the bar axis as a result of structural loadings on the member under consideration (tensions generated by the anchorage of the bar itself should not be included here).

6.1.3.3 Design bond length

The design anchorage or lap length may be calculated from :

$$l_b = \alpha_4 \frac{\phi.\sigma_{sd}}{4 f_{bd}} \geq l_{b,\min} \qquad (6.1\text{-}16)$$

where

α_4 is a function of the percentage of reinforcement lapped or anchored within 0.65 l_b from the centre of the anchorage or lap length considered:

% of bars lapped	>50%	>34%	>25%	<25%
α_4	1.1	1.0	0.85	0.75

The clear distance between lapped bars should not be greater than 4Ø or 50 mm, otherwise the lap length should be increased by a length equal to the clear space where it exceeds 4Ø or 50 mm

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Lapped joints should be located away from regions of high stress whenever possible. If this is not possible, particular attention should be paid to ensuring robustness of the joint.

With the exception of compression bars of columns or walls in footings, bends and hooks are not permitted for compression reinforcement.

Eq. (6.1-18) is based on the standard bend radius. For larger bend radii, see subclause 7.13.2.

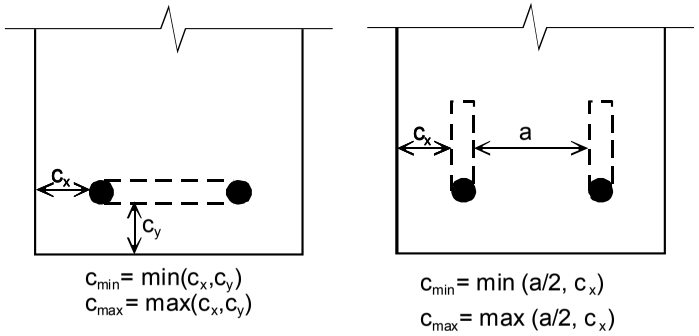


Figure 6.1-10: Notation for bar spacing and cover: hooks and bends

End bearing should only be considered to contribute to strength of laps and anchorages of bars in compression where the end of the bar is no closer than 3.5Ø (measured parallel to the bar axis) from an unsupported face.

Minimum lap or anchorage length

$$l_{b,min} > \max\{0.3 l_b f_{yd}/\sigma_{sd}; 15 d_b; 200 \text{ mm}\} \quad (6.1\text{-}17)$$

6.1.3.4 Contribution of hooks and bends

End bends or hooks contribute to transfer of force for bars in tension. The value of F_h in Eq. (6.1-18) for a standard bend or hook may be taken as:

$$F_h = 50 f_{bd} A_b \quad (6.1\text{-}18)$$

6.1.3.5 Laps and anchorages of bars in compression

Bearing of ends of bars in compression augments bond in the transfer of force between bar and concrete. Provided that the bar terminates a distance of at least 3.5Ø from an unsupported face, the value of F_h in Eq. (6.1-19) may be taken as:

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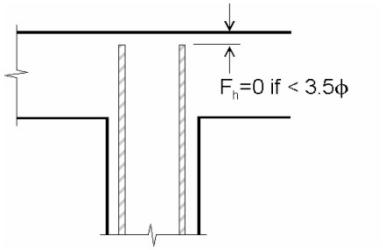


Figure 6.1-11: Minimum end cover to compression bars

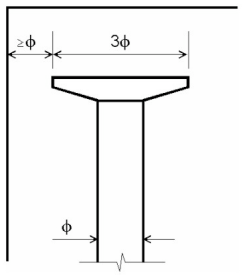


Figure 6.1-12: Condition (a), where the full yield capacity of the bar must be developed at the head

While it is evident that the strength of an anchorage comprising a straight portion of bar plus a welded head is less than the sum of the strengths of the two components acting independently, there not as yet a consensus model for condition (b).

$$F_h = 40 f_{bd} A_b \qquad (6.1\text{-}19)$$

Minimum covers perpendicular to the bar axis are described in subclause 6.1.3.2.

6.1.3.6 Headed reinforcement

Two conditions may be considered:

a) where the full yield capacity of the bar must be developed at the head, for example in double headed studs for shear or as wall ties, and concrete is uncracked perpendicular to the axis of the bar.

b) other circumstances in which the reinforcement is anchored by a combination of anchorage by the head and bond along the length of the bar.

Condition (a) may be satisfied by a stud with head diameter 3 times that of the bar, where the minimum cover to the side of the head is not less than 1.5 bar diameters and concrete is uncracked perpendicular to the axis of the bar.

For condition (b), anchorage capacity may be determined in one of three ways:

i) the headed bar may be treated as if it were a bar terminated by a hook or bend (subclause 6.1.3.4) provided the net projected area of the

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Option (ii) for condition (b) represents a conservative approach.

For more details reference is made to CEB Design Guide, "Design of fastenings in concrete". Thomas Telford, London, 1997. ISBN 978-0-7277-3566-9

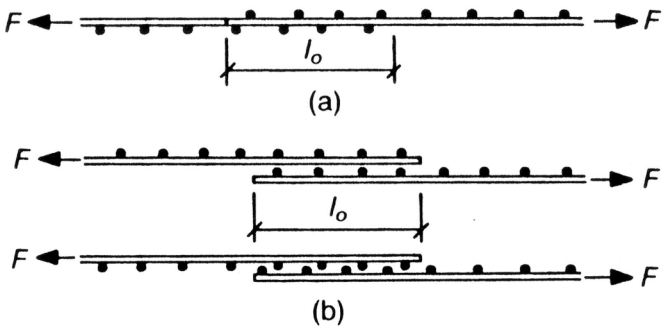


Figure 6.1-13: Lapping of welded fabric: (a) intermeshed fabric (longitudinal section); (b) layered fabric (longitudinal section)

Splicing of welded fabric in structures assessed for fatigue loads should be done with intermeshed fabrics.

head is equal to that of a standard bend and the weld between hook and bend is able to develop the full yield strength of the bar,

ii) as the capacity of the head alone with no contribution from bond along the straight portion of the bar, using the procedures for cast-in-place headed anchors in tension in the CEB Design Guide 'Design of Fastenings in Concrete'.

iii) anchorage capacity may be determined by test.

In all cases the head must have sufficient embedment beyond the most highly stressed point of the bar to preclude a premature concrete cone type failure.

6.1.4 Anchorage and lapped joints of welded fabric

6.1.4.1 Design lap length of welded fabric in tension

6.1.4.1.1 Lap of the main reinforcement

The minimum number of welded cross wires over the lap length is:

n = 1 for fabric made of ribbed wires

n = 5 ($A_{s,\ calc}$ / $A_{s,\ ef}$) for fabric made of plain indented wires (n to be rounded up to the next whole number).

The design lap length is:

- with intermeshed fabrics (Figure 6.1-13), according to Eq. (6.1-16)
- with layered fabric

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For welded fabric placed in more than one layer, the values of l_0 from Eq. (6.9-8) can be reduced by 20% for the fabric further from a surface.

$$l_0 \geq \alpha_5 \frac{Ø}{4} \frac{f_{yd}}{f_{bd}} \frac{A_{s,calc}}{A_{s,ef}} \geq l_{0,\min} \tag{6.1-20}$$

where

$$\alpha_5 = 0.5 + \frac{A_s / s}{A_{s,ef}} \geq l_{0,\min} \quad \text{and} \quad 1.0 \leq \alpha_5 \leq 2.0 \tag{6.1-21}$$

and

$$l_{o,\min} > \max \{0.75 \frac{Ø}{4} \frac{f_{yd}}{f_{bd}}; 15Ø; s; 200\text{mm}\} \tag{6.1-22}$$

A_s /s corresponds to specific cross-section area of the fabric (mm^2 /m)

6.1.4.1.2 Laps in the transverse direction: secondary reinforcement

For intermeshed fabrics provisions as for the main welded fabric reinforcement apply (see subclause 6.1.4.1.1).

For layered fabrics, the length of lap is chosen from Table 6.1-3:

Table 6.1-3: Required lap lengths for splices of the secondary reinforcement (layered fabrics)

Diameter of wires (mm)	Lap lengths	
Ø ≤ 6	≥ 150 mm; at least 1 wire pitch with the lap length	
6 <	Ø ≤ 8.5	≥ 250 mm; at least 2 wire pitch
8.5 <	Ø ≤ 12	≥ 350 mm; at least 2 wire pitch

6.1.4.2 Design lap length of welded fabric in compression

For the main reinforcement the design lap length should comply with the condition:

$$l_0 \geq \frac{Ø}{4} \frac{f_{yd}}{f_{bd}} \tag{6.1-23}$$

For the secondary reinforcement, subclause 6.1.4.1.2 applies.

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The suitability of the systems has to be proved by an independent approval process. Full provisions are given in **ETAG TR023**: Technical Report for Post-Installed Rebar Connections' (2006) to ETAG 001: Guideline for European Technical Approval of Metal Anchors for Use in Concrete, Part 5: Bonded Anchors, EOTA (2008).

6.1.5 Special circumstances

6.1.5.1 Slipform construction

The value of coefficient η_2, Eq. 5.1.2, should be taken as 0.7 for bars in structural parts built using slipform construction.

6.1.5.2 Bentonite walling

The value of coefficient η_2, Eq. 5.1.2, should be taken as 0.7 for bars in concrete cast under bentonite or polymer drilling fluids.

6.1.5.3 Post-installed reinforcement

Rebar connections using post-installed rebars are permissible for all applications where straight cast-in-place rebars are allowed . The design can be performed in a simplified way using the provisions for cast-in-place rebars in the codes for reinforced concrete. However, the following restrictions must be considered:

- larger minimum concrete cover,
- larger minimum clear bar spacing,
- limited design compression strength,
- special requirements for fire safety.

6.1.5.4 ECE (electrochemical extraction of chlorides)

The correct application of electrochemical techniques for chloride extraction from or re-alkalisation of concrete does not cause any significant long-term reduction in bond strength, except where:.

- AAR is induced in concrete containing susceptible aggregates by the electrochemical process.
- an appreciable amount of corrosion has occurred prior to treatment. Plain round bars are likely to be particularly susceptible to loss of bond in these circumstances, see subclause 6.1.7.1

There may be a temporary reduction in bond strength while treatment is underway. The reduction does not exceed 50%, and strength is restored within a few days of cessation of treatment.

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Both bond strength and stiffness increase at temperatures below -80°C. The increase is influenced by a number of factors, and hence is too complex for a Code type formulation. The approach given in subclause 6.1.6.1 is conservative.

Changes in bond strength and stiffness with increasing temperature are influenced by a number of factors, and hence are too complex for a Code type formulation. The approach given in subclause 6.1.6.2 represents a reasonable simplification.

Most data on bond resistance of corroded reinforcement is obtained from tests in which corrosion activity has been accelerated, and corrosion rates are in excess, or well in excess, of those measured in field exposure. Consequently, experimental data must be interpreted with caution.

The magnitude of the reduction in residual bond strength is highly dependent on the confinement to the bar, and is also affected by concrete quality and environment. The following values may be taken as indicative, but detailed guidance should be sought in cases where residual strength of a corroding structure is of concern.

6.1.6 Conditions of service

6.1.6.1 Cryogenic conditions

The basic bond strength given by Eq. (6.1-11) may safely be used in conditions of low temperature.

6.1.6.2 Elevated temperatures

The reduction in bond strength of ribbed bars at elevated temperatures may be taken as similar to that for tensile strength of concrete. Note that explosive spalling of HSC may reduce concrete cover.

Bond strength of plain round bars at 300° and 500° may be taken as 50% and 10% respectively of strength at normal temperatures.

6.1.7 Degradation

6.1.7.1 Corrosion

Corrosion of reinforcement embedded in concrete may affect residual capacity of reinforced concrete structures. The effects of corrosion in hardened concrete differ from those associated with corrosion prior to concreting.

Small amounts of corrosion, up to the level required to induce longitudinal cracking, do not cause loss of bond capacity, and may even augment bond strength to a modest degree, particularly where the bar is in a ‘poor’ casting position.

At greater levels of corrosion, residual bond strength is strongly influenced by the degree of confinement provided by secondary reinforcement in the form of links and by the surrounding structure. Transverse pressure from support reactions augments bond . Links play a valuable role in maintaining residual strength of anchorages and lapped joints.

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Table 6.1-4: The magnitude of the reduction in residual bond strength for corroded reinforcement

Corrosion penetration (mm)	**Equivalent surface crack** (mm)	**Confinement**	**Residual capacity** % Bar type	
			Ribbed	Plain
0.05	0.2-0.4	No links	50-70	70-90
0.10	0.4-0.8		40-50	50-60
0.25	1.0-2.0		25-40	30-40
0.05	0.2-0.4	Links	95-100	95-100
0.10	0.4-0.8		70-80	95-100
0.25	1.0-2.0		60-75	90-100

It should be appreciated that residual strength of concrete structures is also affected by cross section loss of both steel and concrete.

Alkali aggregate reactions (AAR) occur in concrete when hydroxyl ions in the pore water react with certain components of the aggregate. The reaction product is a highly expandable gel which imbibes water and swells. If sufficient expansion takes place the result is cracking and expansion of the surrounding concrete. The surface of the concrete does not expand to the same extent as the interior and this causes tensile stresses to be set up in the surface of the concrete, these can, in turn lead to surface macro cracks. The most common form of AAR is alkali silica reaction (ASR) with other less common forms being alkali silicate reaction and alkali carbonate reaction. The guidance in this section is appropriate to ASR.

The residual capacity of anchorages and lapped splices should be checked at the ultimate limit state at locations of high reinforcement stress where longitudinal cracking develops. Away from anchorages and laps, a substantial loss of bond may be tolerated without ultimate strength being affected. Rotation capacity at plastic hinges is likely to increase with corrosion, as the length of bar over which plastic strains develop will increase as longitudinal cracking develops and bond is reduced.

For the purpose of assessing performance at the serviceability limit state, it is likely a) that the influence of bond on deflections will be small compared to that of loss of reinforcement and of concrete cross section, b) the width of corrosion induced longitudinal crack widths is likely to exceed that of flexural cracks by the time a significant change in flexural crack widths would be observed , and c) by this stage of deterioration the serviceability limit state of durability will in any case be the more critical.

6.1.7.2 ASR

When laps and anchorages are restrained by links expansion within the limits 450 με restrained and 4500 με unrestrained induced by ASR has been shown not to have a significant effect on bond strength.

Where transverse reinforcement is not present then bond strength may be reduced by up to 50%.

Care is necessary in the assessment of residual resistance of anchorage zones of prestressed elements, particularly of pre-tensioned members in which the tendons are not contained by links.

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Surface scaling is generally found where the surface of the concrete is subjected to weak solutions of salt or urea, typically for de-icing purposes.

Internal freeze-thaw damage results from expansive stresses generated by water on freezing when the pore structure of the concrete is saturated above a critical value, and leads to internal microcracking. Internal damage is likely only in concrete subjected to long-term wet/saturated conditions. Care needs to be taken in diagnosis for internal frost damage as visual observation is not conclusive.

Tensile strength undergoes greater reductions than compressive strength under frost attack, and it is not acceptable to use estimated tensile strengths based on compressive strength measurements.

For more details reference is made to CONTECVET (2001) A validated Users Manual for assessing the residual service life of concrete structures. Manual for assessing structures affected by frost. Geocisa, Madrid. Available on CD from BCA, Crowthorne, Berskshire, UK

6.1.7.3 Frost

Two types of damage to concrete may occur as a result of freezing and thawing:

- surface scaling
- internal damage

Surface scaling leads to a reduction in concrete cover to reinforcement. The effect on bond may be allowed for by use of the reduced cover in Eq. (6.1-13).

Residual bond capacity of ribbed bars not confined by links in freeze-thaw damaged concrete may be assessed using splitting tensile strength measurements on cores taken from the affected structure. The concrete compressive strength f_{ck} in Eq. (6.1-11) may be substituted with $f_{ck,ft}$ given by:

$$f_{ck,ft} = 3.3 f_{ctk,is}^{1.5} \quad (6.1\text{-}24)$$

where:

$f_{ckt,is}$ is the characteristic measured in-situ tensile strength.

Bond strength of ribbed bars is not degraded as severely where bars are confined by secondary reinforcement detailed in compliance with the requirements of this Code. Residual bond strength is expected to be at least 70% of that 'as constructed'

6.1.7.4 Fire

The reduction in residual bond strength of ribbed bars in fire damaged concrete structures may be taken as similar to that for tensile strength of concrete. Where surface spalling has occurred, the reduced concrete cover should be used.

Residual bond strength of plain round bars after heating to 300° and 500° may be taken as 50% and 10% respectively of strength at normal temperatures.

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Two different bond situations should be considered due to the transverse deformations of the tendon. A 'push-in' along the transmission length, where the tendons become thicker at release. and a 'pull-out', which refers to the anchorage length where the opposite occurs when the steel stress is increased due to loading.

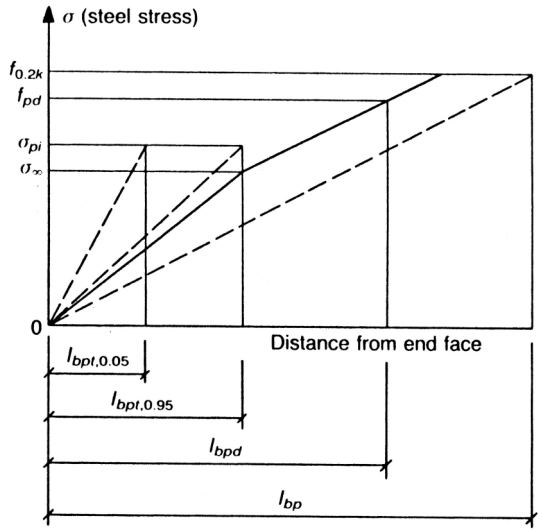


Figure 6.1-14: Variation in steel stress along the anchorage zone of a pretensioned member

6.1.8 Anchorage of prestressing tendons

6.1.8.1 General

The bond strength of pretensioned prestressing tendons depends on the loading case. The highest value applies to the transmission length – length to introduce the prestressing force. Beyond that length a lower bond strength has to be taken into account. This results in a bilinear diagram for the embedment length that is required to develop the design steel stress (see Figure 6.1-14).

6.1.8.2 Design bond strength

The design value of the bond strength for prestressing tendons is:

$$f_{bpd} = \eta_{p1}\eta_{p2} f_{ctd} \qquad (6.1\text{-}25)$$

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where:

f_{ctd} $= f_{ctk}\ (t)$ /1.50 is the lower design concrete tensile strength; for the transmission length the strength at the time of release, for the anchorage length the strength at 28 days

η_{p1} takes into account the type of prestressing tendon:

$\eta_{p1} = 1.4$ for indented and crimped wires

$\eta_{p1} = 1.2$ for 7-wire strands

η_{p2} takes into account the position of the tendon:

$\eta_{p2} = 1.0$ for all tendons with an inclination of 45°-90° with respect to the horizontal during concreting

$\eta_{p2} = 1.0$ for all horizontal tendons which are up to 250m from the bottom or at least 300m below the top of the concrete section during concreting

$\eta_{p2} = 0.7$ for all other cases

6.1.8.3 Basic anchorage length

The basic anchorage length of an individual pretensioned tendon is:

$$l_{bp} = \frac{A_{sp}}{\text{Ø}\pi}\frac{f_{ptd}}{f_{bpd}} \qquad (6.1\text{-}26)$$

where:

f_{ptd} $= f_{ptk}$ /1.15 and f_{ptk} is defined in subclause 5.3.5.1.

6.1.8.4 Transmission length

The transmission length of a pretensioned tendon is:

$$l_{bpt} = \alpha_{p1}\alpha_{p2}\alpha_{p3}\frac{\sigma_{pi}}{f_{ptd}} \qquad (6.1\text{-}27)$$

The basic anchorage length defines the length that is required to develop the full strength in an untensioned tendon.

The factor A_{sp} / Ø π depends on the type of tendon:

$\frac{A_{sp}}{\phi\pi} = \frac{\text{Ø}}{4}$ for tendons with a circular cross-section

$\frac{A_{sp}}{\phi\pi} = \frac{7}{36}\text{Ø}$ for 7-wire strands

The use of narrow spaced stirrups or helices around the tendons and transverse prestressing may result in shorter transmission length . This is not considered due to lack of experimental data.

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Tendon release that is obtained by sawing through the concrete and the steel should be considered as gradual release.

The transmission length can be estimated from the draw-in value (δ_e) of the tendons at the end face of the concrete member. However, when the concrete member is sawn from a longer production unit, the draw-in cannot be estimated properly. Assuming a linear steel stress along the transmission length, this draw-in shall be:

$$\delta_e < 0.5 \frac{\sigma_{pi}}{E_p} l_{bpt}$$

with α_{p2}= 1.0 in the Eq. (6.1-27) for l_{bpt} ; E_p is the modulus of elasticity of the prestressing steel.

See commentary to subclause 6.1.8.1 for different bond situations. The basic anchorage length is related to "pull-out". The transmission length is connected to "push-in".

If necessary, the required anchorage capacity may be obtained by additional end anchorages or non-prestressed reinforcement.

where:

σ_{pi} is the steel stress just after release

α_{p1} considers the type of release

α_{p1}= 1.0 for gradual release

α_{p1}= 1.25 for sudden release

α_{p2} considers the action effect to be verified

α_{p2}= 1.0 for calculation of anchorage length when moment and shear capacity is considered

α_{p2}= 0.5 for verification of transverse stress in anchorage zone

α_{p3} considers the influence of bond situation

α_{p3}= 0.5 for strands

α_{p3}= 0.7 for indented or crimped wires

6.1.8.5 Design anchorage length

The design anchorage length of a pretensioned prestressing tendon is:

$$l_{bpd} = l_{bpt} + l_{bp} \frac{\sigma_{pd} - \sigma_{pcs}}{f_{ptd}} \qquad (6.1\text{-}28)$$

where:

σ_{pd} is the tendon stress under design load ($\sigma_{pd} \le f_{ptd}$)

σ_{pi} is the tendon stress due to prestress including all losses.

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For non-rectangular sections the development length can be found in a similar way as assumed for post-tensioning.

6.1.8.6 Development length

The development length is the distance from the end face to the concrete cross-section beyond which the distribution of the longitudinal stresses is considered linear.

For a rectangular cross-section and straight tendons situated near the bottom edge of the concrete section the development length is:

$$l_p = \sqrt{[h^2 + (0.6 l_{bpt})^2]} > l_{bpt} \qquad (6.1\text{-}29)$$

where:

h is the total depth of the concrete section.

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6.2 Bond of non-metallic reinforcement

Bond of non-metallic reinforcement is the term used to denote the interaction and transfer of force between fibre reinforced polymer (FRP) reinforcement and concrete. At the serviceability limit state, bond influences width and spacing of transverse cracks, tension stiffening and curvature. At the ultimate limit state, bond is responsible for strength at end anchorages or at intermediate regions (the latter in the case of externally bonded reinforcement).

6.2.1 Local bond stress-slip model

The bond stresses between concrete and non-metallic reinforcement can be calculated as a function of the relative displacement s.

6.2.1.1 Local bond stress-slip model for FRP rebars

The bond stress can be calculated according to the following equations (see also Figure 6.2-1):

$$\tau = \tau_m (s/s_m)^{\alpha} \qquad \text{for } 0 \le s \le \qquad (6.2\text{-}1a)$$

$$\tau = \tau_m - \tau_m p(s-s_m)/s_m \qquad \text{for } s_m \le s \le s_u \qquad (6.2\text{-}1b)$$

The bond behaviour of FRP reinforcement to concrete depends mainly on the reinforcement geometry, application type (e.g. internal or externally bonded) and surface characteristics. It varies from that of conventional steel reinforcement, given for example the following aspects:

- the modulus of elasticity of FRP is generally lower than that of steel, especially in the transverse direction.
- the shear stiffness of FRP is significantly lower than that of steel.
- the surface deformations relate to the resin matrix, which has lower shear strength than steel.

It is generally possible to obtain bond strengths for non-metallic reinforcement of similar or greater magnitude than for steel reinforcement.

The bond stress-slip relationship (monotonic loading) for deformed steel rebars given in subclause 6.1.1 is applicable for FRP reinforcement, provided that the use of model parameters is calibrated on the basis of experimental results. Generally, a modified bond stress-slip relationship is assumed with an ascending and descending branch and whereas the constant shear stress branches are not applicable in the case of FRP. Further details are given in subclauses 6.2.1.1 and 6.2.1.2

The constitutive model of Figure 6.2-1 has been proposed by Cosenza, Manfredi and Realfonzo (1995) and is also discussed in *fib* Bulletins 10 and 40. The parameters have to be calibrated on the basis of experimental results.

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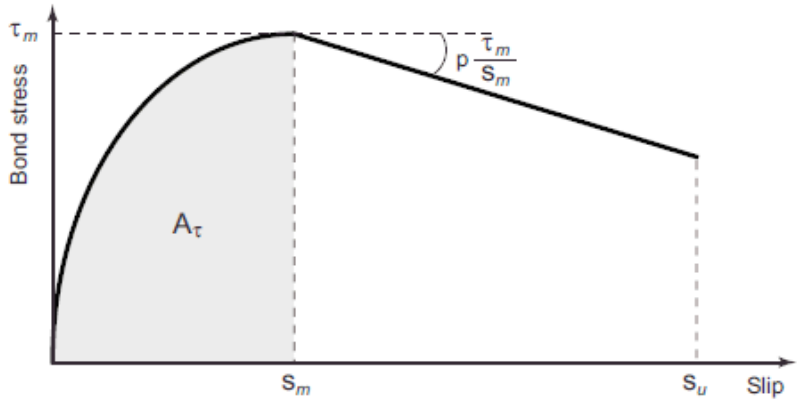


Figure 6.2-1: Analytical bond stress-slip relationship (embedded FRP reinforcement)

6.2.1.2 Local bond stress-slip model for externally bonded FRP

The bond stress can be calculated according to the following equations (see also Figure 6.2-2):

$\tau = \tau_m(s/s_m)$ for $0 \le s \le s_m$ (6.2-2a)

$\tau = \tau_m\text{-}\tau_m(s\text{-}s_m)/(s_u\text{-}s_m)$ for $s_m \le s \le s_u$ (6.2-2b)

The constitutive bond model for externally bonded FRP reinforcement is assumed bilinear. It has been proposed by various authors among which Holzenkämpfer (1994) and is also discussed in *fib* Bulletin 14. The model parameters basically relate to the fracture energy GF of the glued joint connection between the external reinforcement and the concrete and have to be calibrated on the basis of experimental results.

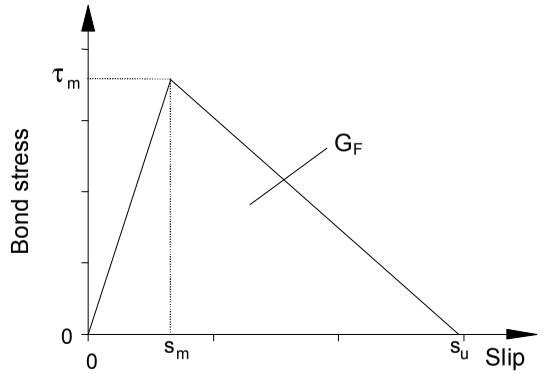


Figure 6.2-2: Analytical bond stress-slip relationship (externally bonded FRP reinforcement)

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For a discussion on available models for the basic development length and lap splices of non-prestressed FRP reinforcement and the transfer length of prestressed FRP reinforcement, reference is made to *fib* Bulletins 10 and 40. Bulletin 40 provided also modelling with respect to tension stiffening, deflections and crack opening, taking into account bond interaction between FRP reinforcement and concrete.

Models are currently being developed by *fib* TG9.3.

A further discussion on bond modelling of EBR is given in *fib* Bulletin 14.

6.2.2 Bond and anchorage of internal FRP reinforcement

Bond of plain (smooth) bars is governed by the adhesion between the bar surface and the concrete, provided that the interlaminar shear strength between the fibres is higher. The bond strength of plain bars is generally low and splitting bond forces can be neglected. Their use is limited as they need to be combined with other anchoring devices such as bends and transverse bars.

Bond of deformed (surface treated) bars is often governed by the shear strength of the deformations (provided that the interlaminar shear strength between the fibres is higher). In this case, influence of concrete strength is limited compared to bond of steel bars. For high strength deformations, concrete shear failure similar to deformed steel bars is more predominant.

The bond strength of deformed FRP bars is similar or superior to that of steel bars. Splitting bond forces can govern in case insufficient confinement by the surrounding concrete is provided. Depending on the surface texture, splitting tendency of FRP bars is lower or higher compared to deformed steel bars. Analytical modelling of bond splitting is provided in Appendix A of *fib* Bulletin 40.

6.2.3 Bond and anchorage of externally bonded FRP reinforcement

In the case of externally bonded reinforcement (EBR) the bond behaviour relates to the glued joint between the external FRP reinforcement and the concrete. As part of any flexural or shear strengthening design, the evaluation of the maximum force that may be transferred from the concrete to the external reinforcement, as well as the evaluation of shear and normal stresses at the concrete-FRP interface is required. The former is necessary when designing at the ULS; the latter when designing at the SLS.

Debonding of EBR can be predicted by considering the different bond failure modes which can occur, as outlined in the following clauses.

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Depending on the cohesive and adhesive strengths of the concrete, adhesive (glue) and FRP, the bond failure may occur at different interfaces, as shown in Figure 6.2-3. Given the high shear strength of the adhesive and the FRP, the failure plane is generally located in the concrete layer near the adhesive.

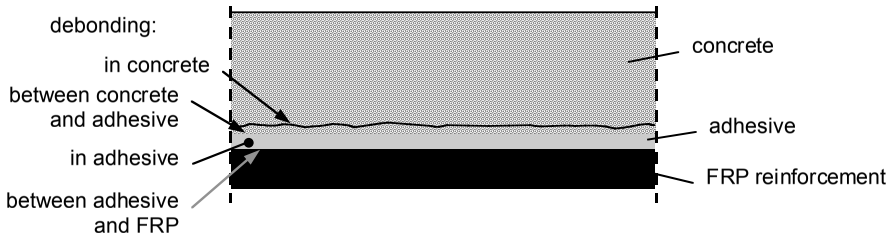


Figure 6.2-3: Bond interface

6.2.3.1 Bond-critical failure modes

Depending on the starting point of the debonding process, the bond-critical failure modes in flexural and shear strengthening with EBR can be classified into two main categories:

- end debonding,
- intermediate crack debonding.

End debonding

If insufficient anchorage capacity is provided interfacial end debonding occurs as shown in Figure 6.2-4 (top). A specific form of end debonding is concrete cover separation or concrete rip-off, see Figure 6.2-4 (bottom). The latter failure mode is obtained when a shear crack at the end of the FRP reinforcement propagates into a debonding mode at the level of the internal reinforcement.

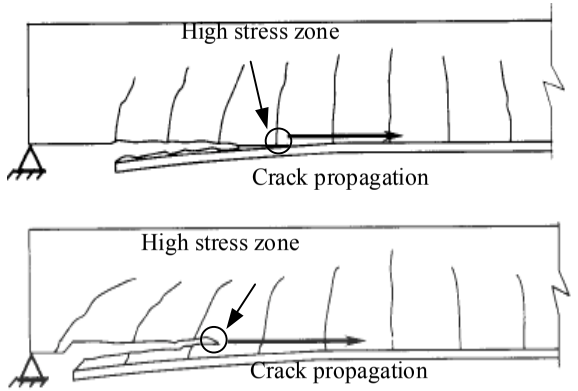


Figure 6.2-4: Anchorage (top) and concrete rip-off (bottom) failure.

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Intermediate crack debonding

Debonding of FRP can also be caused by bridging of intermediate cracks as illustrated in Figure 6.2-5.

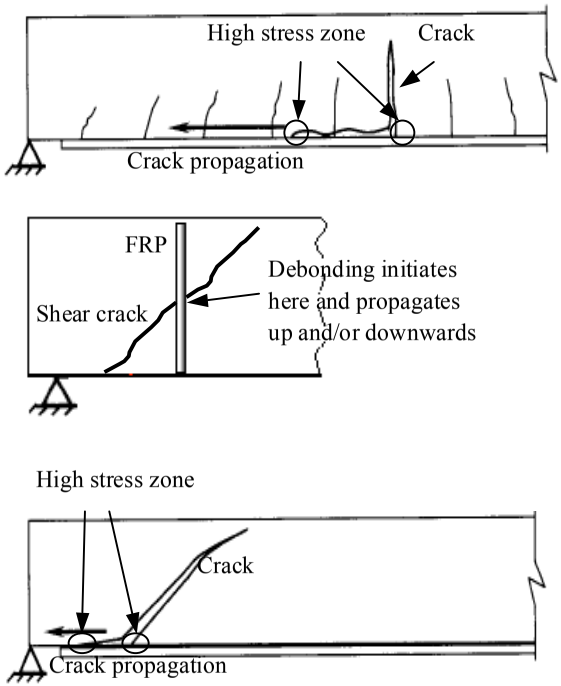


Figure 6.2-5: Intermediate crack debonding failure aspects

6.2.3.2 Maximum bonded length

With reference to a typical bond test, as represented in Figure 6.2-6, the ultimate value of the force transferred to the FRP system prior to debonding depends on the length, ℓ_b, of the bonded area. The maximum bonded length, $\ell_{b,max}$, is defined as the length that, if exceeded, there would be no increase in the force transferred between concrete and EBR.

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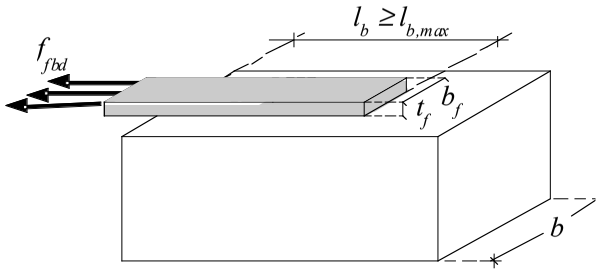


Figure 6.2-6: FRP pure shear bond test configuration

The maximum bonded length may be estimated as follows:

$$\ell_e = 0.6\sqrt{\frac{E_f t_f}{\sqrt{k_b f_{ctm}}}} \qquad \text{(lengths in mm, stresses in MPa)} \qquad (6.2\text{-}3)$$

where:

E_f is the elastic modulus in the stress direction of the FRP,

t_f is the thickness of the FRP,

f_{ctm} is the mean tensile strength of concrete substrate,

k_b is a shape factor, equal to:

$$k_b = \sqrt{\frac{2 - b_f / b}{1 + b_f / b}} \geq 1 \qquad (6.2\text{-}4)$$

b_f and b being the FRP and concrete section width, respectively.

6.2.3.3 Ultimate strength for end debonding – anchorage capacity

The mean and the design ultimate bond strength, that is the maximum stress transferable from concrete to EBR is:

The maximum bond anchorage capacity is valid for bond lengths equal or higher than the maximum bond length. If smaller bond lengths are provided, the bond anchorage capacity is reduced assuming a parabolic relationship between the anchorage capacity and the bond length, as expressed by the factor β_ℓ.

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$$f_{fbm} = k_m \cdot \beta_\ell \cdot k_b \sqrt{\frac{2 \cdot E_f}{t_f} \cdot \sqrt{f_{cm} \cdot f_{ctm}}} \qquad (6.2\text{-}5a)$$

(lengths in mm, stresses in MPa)

$$f_{fbd} = \frac{k_k}{\gamma_{f,b}} \cdot \beta_\ell \cdot k_b \sqrt{\frac{2 \cdot E_f}{t_f} \cdot \sqrt{f_{cm} \cdot f_{ctm}}} \qquad (6.2\text{-}5b)$$

(lengths in mm, stresses in MPa)

where:

f_{cm} is the mean cylinder compressive strength of concrete,

$\gamma_{f,b}$ is the FRP partial safety factor for debonding (equal to 1.5),

β_ℓ is the length factor, defined as:

$$\beta_\ell = \frac{\ell_b}{\ell_{b,\max}} \cdot \left(2 - \frac{\ell_b}{\ell_{b,\max}}\right) \text{ if } \ell_b \le \ell_{b,max}, \ \beta_\ell = 1 \text{ otherwise} \qquad (6.2\text{-}6)$$

On the basis of calibration with experimental results, for epoxy bonded CFRP systems, k_m = 0.21 and under the hypothesis that the debonding load has a lognormal distribution k_k = 0.12.

6.2.3.4 Ultimate strength for end debonding – concrete rip-off

This debonding mode can be avoided by providing shear strengthening at the end of the FRP. The design of the shear strengthening aims at extending the existing shear links up to the level of the EBR.

In case a shear crack occurs at the end of the FRP, this shear crack can propagate into concrete cover separation, related to the missing tension link between the internal and external reinforcement when considering the classical truss analogy for shear capacity of beams. Prediction of occurrence of a shear crack at the end of the FRP is provided in *fib* Bulletin 14, and acts as a lower bound prediction model for concrete rip-off.

6.2.3.5 Ultimate strength for intermediate debonding

According to a simplified procedure, the design ultimate bond strength is obtained by multiplying f_{fbd} by a multiplication factor k_c. If specific experimental data are not available, k_c may be taken equal to 3.

For more detailed modelling of intermediate debonding, reference is made to for example *fib* Bulletin 14.

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Bond stresses (shear and normal) at serviceability limit state can be calculated on the basis of linear elastic analysis.

6.2.3.6 Interfacial stresses for the serviceability limit state

It is assumed that bond interface crack initiation will not occur under service load, provided proper detailing and limitation of deflections and crack opening.

6.2.4 Mechanical anchorages for externally bonded FRP reinforcement

Debonding at the ends of the EBR could be avoided, or an enhancement of the debonding load could be achieved, using anchorage systems. Various solutions are available and can be designed for the specific case employing the fibres themselves with suitable configurations (e.g. transverse wrapping, spike anchors) or additional devices such as bolts, plates, etc.

The ultimate value of the force transferred to the EBR system prior to debonding depends on the efficiency of the anchorage system. In the presence of mechanical anchorage, the design bond strength f_{fad} may be taken equal to:

$$f_{fad} = k_a f_{fd} \tag{6.2-7}$$

where:

f_{fd} is the design tensile strength of the FRP

k_a is the effectiveness coefficient of the specific anchorage system ($k_a \leq 1.0$), provided on the basis of experimental results.

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In practice the following aspects have to be considered:

– contamination of the concrete surface just before casting the overlay may reduce bonding significantly; likewise unfavourable climatic conditions (like strong dry winds or solar radiation) can dry up the concrete surface and reduce bond strength,
– inappropriate roughening methods may harm the top surface of the concrete (leading e.g. to micro-cracking),
– insufficient quality of the overlay concrete may cause larger constraint forces (e.g. due to shrinkage) and reduce bonding,
– edge zones (where significant internal tensile and shear stresses may occur due to forces of constraint) have to be sufficiently secured.

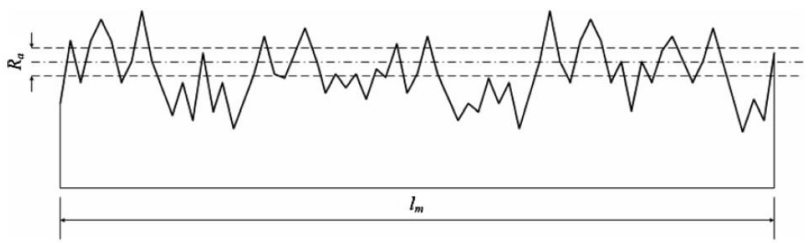


Figure 6.3-1: Average roughness R_a

6.3 Concrete to concrete

6.3.1 Definitions and scope

Concrete-to-concrete load transfer across interfaces has to be considered when two concretes are cast against each other at different times, i.e. when the hardening process of the older concrete is already finished. While tensile loads have to be transferred via reinforcement to design for the ULS, shear forces with their load direction parallel to the interface or normal forces perpendicular to the interface or a combination of shear and normal forces are to some extent transferred directly from concrete to concrete.

The topic is relevant in practice in the following situations:

– repair and strengthening of existing RC-members by means of new concrete layers;
– supplement of precast elements with additional concrete cast on the site;
– all situations on the site where due to interruptions in the erection process new concrete is cast against already completely hardened concrete;
– post-installed concrete elements (e.g. corbels) attached to existing members for introduction of loads.

6.3.2 Interface roughness characteristics

There are several indicators to describe and quantify the roughness of a concrete surface. The most commonly used parameter is the average roughness R_a which represents the average deviation of the profile from a mean line. It is determined as the mean value of profile heights along a certain length l_m:

$$R_a = \frac{1}{l_m} \cdot \int_0^{l_m} y(x) \cdot dx \approx \frac{1}{n} \sum_{i=1}^{n} y_i \qquad (6.3\text{-}1)$$

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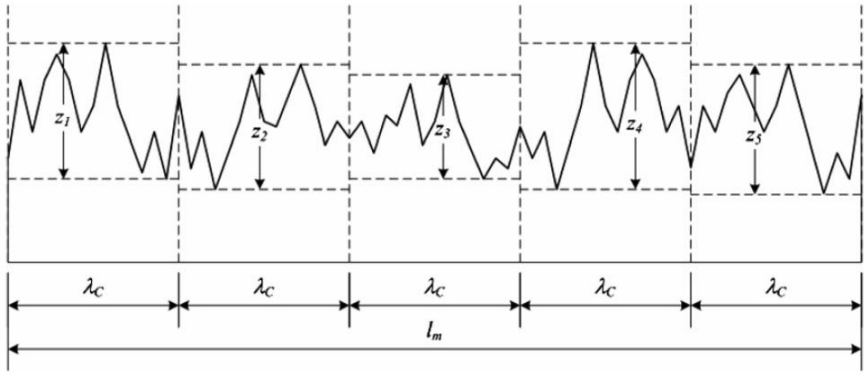


Figure 6.3-2: Mean peak-to-valley height $R_{z\,(DIN)}$

There are several methods to measure and specify the roughness of a concrete surface. A simple method widely used on the site is the sand patch method: A defined volume of fine sand is spread on the surface; depending on the diameter of the circle the average roughness can be estimated as the mean height of the sand cylinder. The advantage of this method is its simplicity and quickness. The disadvantages are that it is not very exact, only possible on horizontal or slightly inclined surfaces and that only the average roughness R_a can be determined.

Other advanced methods are due to their increased complexity more used for laboratory investigations. The advantage of these methods is that they allow for exact measurements of roughness profiles and therefore deliver a more detailed specification of the surface topography. Examples are as follows:

- contact methods:
  - mechanical contact profilometer;

where:

l_m is the evaluation length,

$y(x)$ is the profile height at position x.

Another frequently used parameter is the mean peak-to-valley height $R_{z\,(DIN)}$, representing the average of maximum valley-to-peak-heights z_i within a certain number of assessment lengths:

$$R_{z(DIN)} = \frac{1}{5} \cdot \sum_{i=1}^{5} z_i \qquad (6.3\text{-}2)$$

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– non-contact methods:
  – laser triangulation;
  – photogrammetry – digital imaging.

The simplified classification into the 4 categories very smooth, smooth, rough and very rough on the basis of the average roughness R_a is not an exact method of categorization, but useful for practical design. However, the designer should be aware that identical values of the same average roughness R_a might still lead to different shear resistances due to differences in the actual surface topography and therefore always engineering judgement is required when determining the appropriate roughness category.

For design the surface roughness has to be classified in different categories. Depending on the average roughness R_a and the applied roughening method the following categories can be defined:

Category:	R_a [mm]
Very smooth (e.g. cast against steel formwork)	not measureable
Smooth (e.g. untreated, cast against wooden formwork)	< 1.5 mm
Rough (e.g. sand blasted, high pressure water blasted etc.)	≥ 1.5 mm
Very rough (e.g. high pressure water jetting, indented)	≥ 3 mm

6.3.3 Mechanisms of shear transfer

Substantial experimental investigations have been performed since about 1960. The results show a very large scatter for the following reasons:

1. The test setup affects the flow of forces within the specimen and a clear determination of the shear force distribution along the interface may be difficult even with small scale specimens.
2. Depending on whether or not reinforcement / connectors cross the interface the load bearing behaviour may be completely different.
3. Bond-slip and ultimate load are strongly influenced by the bond strength which can on the one hand under laboratory conditions be very good or may on the other hand, in such tests be intentionally minimized by pre-cracking the specimen or reduced by the use of de-bonding agents in order to account e.g. for possible contamination at the site.

The shear strength of interfaces between concretes cast at different ages can be investigated experimentally at large scale members like beams or slabs or with small scale specimens. There is a variety of small-scale test-setups like slant shear test, pull-off and push-off tests etc. The main parameters decisive for the actual load bearing capacity observed in tests are:

– interface roughness;
– cleanliness of surface;
– concrete strength and concrete quality;
– eccentricity / inclination of shear force;
– strong bond / pre-cracking / de-bonding before testing;
– ratio of reinforcement crossing the interface.

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4. In the case of an uncracked joint without reinforcement the shear stresses are transferred primarily in the edge zones so that the size and shape of the shear interface affect the ultimate shear strength τ_u.

Shear-slip characteristics of adhesive bonding: very stiff, only effective at shear slips smaller than ~0.05 mm

Parameters influencing adhesive bonding are amongst others:

- preparation (roughness, moisture etc.) and cleanliness of interface,
- strength class of old and new concrete,
- quality of top layer of old concrete,
- porosity and moisture content of old concrete,
- quality and composition of new concrete,
- degree of shrinkage of new concrete,
- age of concretes.

High pressure water jetting may for example lead to undercut effects and keying of protruding aggregates so that mechanical interlocking plays an important role, even at slips larger than 0.05 mm when adhesive bonding is already broken.

The main contributions to the overall shear resistance result from the following mechanisms:

- mechanical interlocking and adhesive bonding,
- friction due to:
  - external compression forces perpendicular to the interface,
  - clamping forces due to reinforcement / connectors,
- dowel action of reinforcement / connectors crossing the interface.

Adhesive bonding:

Adhesive bonding, i.e. adhesive forces due to chemical and physical bonding, can significantly contribute to the overall resistance provided the bond effects are not reduced by contaminants along the interface. Taking into account the effect of adhesive bonding for the ultimate load implies that the related slip at failure is very small, otherwise this effect will collapse.

Compared to the other mechanisms contributing to interface shear transfer adhesive bonding is more sensitive to any contamination of the surface.

Provided good bonding is achieved by appropriate measures (clean surface, appropriate roughening, good concrete quality etc.) representative mean values for the mean shear resistance are for concrete grades ≤ C50/60 in the following ranges:

Rough interface (e.g. sand blasted): ~ 1.5 - 2.5 N/mm²

Very rough interface (e.g. high pressure water jetted): ~ 2.5 - 3.5 N/mm²

Mechanical interlocking:

In addition to adhesive bond forces appropriate roughening and a resulting irregular surface contour may in addition lead to strong mechanical interlocking effects.

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Shear-slip characteristics of aggregate interlock: very stiff, decreases with increasing slip

The friction effect in reinforced shear joints can be explained most simply by means of the so-called 'shear friction theory'. In this respect, the joint is described by a simple saw-tooth model. According to this model, when a surface is rough, a shear force causes not only parallel displacements, but also forces the joint to open, which generates tensile stresses in the reinforcement crossing the interface. These, in turn, create equalizing compressive stresses in the joint, permitting frictional forces to develop.

Shear-slip characteristics for friction show a slight decrease with increasing slip due to interface deterioration.

Shear-slip characteristics of dowel action: increases with increasing slip

Dowel action of the reinforcement / connectors crossing the interface means first of all the bending resistance which develops its maximum at shear slips of up to approximately 0.1-0.2 times the bar diameter. At large slips the so called kinking-effect can be observed: In the crossing-zone with the interface the inclined bar at large slips increasingly provides a contribution to the shear resistance with the horizontal component of the tensile force in the bar.

Representative mean values for the interlocking effect are for concrete grades ≤ C50/60 in the following range:

High pressure water jetted interface: ~ 1.5 - 2.0 N/mm²

Friction:

In the case of compression forces perpendicular to the interface, a frictional resistance can develop depending on the degree of interface roughness. Reinforcement or connectors crossing the interface may also lead to the activation of frictional forces, provided the surface has been sufficiently roughened (so that a shear slip is linked to an opening of the joint).

Representative mean values for the coefficient of friction μ are for concrete grades ≤ C50/60 in the following ranges:

Smooth interface:	0.5 – 0.7
Rough interface:	0.7 – 1.0
Very rough interface:	1.0 – 1.4

Dowel action

Dowel action refers to the bending resistance of reinforcing bars or connectors crossing the interface: A shear slip along an interface leads to a lateral displacement between upper and lower connector ends, thereby inducing bending stresses in these bars which are superimposed by axial tensile forces due to the opening of the joint.

These tensile forces, on the other hand, do not allow for the full bending resistance resulting from the plastic moment of the bar to develop.

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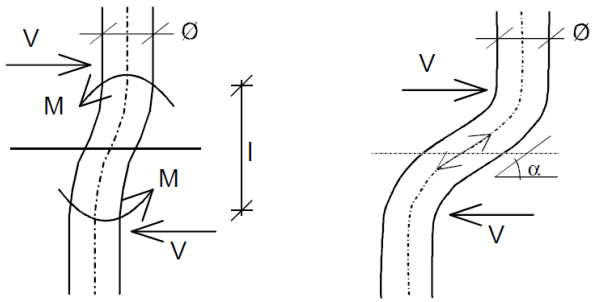


Figure 6.3-3: Dowel action: Bending and kinking effect at large slips

Interaction:

In a real structure subject to shear loading the various mechanisms (i.e. adhesive bonding, mechanical interlocking, friction and dowel action) interact, thereby affecting each other as a function of the shear slip: After failure of the adhesive bond, with increasing shear slip also the aggregate interlock effect decreases quickly when protruding aggregates crush. While also the frictional resistance declines somewhat, the bending resistance of the connectors increases with larger slips.

If there is no interface reinforcement present, the behaviour of the joint is quite brittle: typically failure occurs with the loss of adhesion at slips below 0.05 mm.

Reinforced joints show a more ductile behaviour, depending on the degree of reinforcement and the interface roughness failure occurs typically at much larger slips (~0.5 – 1.5 mm).

6.3.4 Modelling and design

The ultimate resistance of an interface subject to shear forces can be approached by superposition of the single mechanisms of adhesion and interlocking, friction and dowel action. As these mechanisms interact with each other and reach their maxima at different slips it is not possible to add them all together with their maximum values.

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A "rigid" bond-slip behaviour is to be expected when in the case of smooth interfaces no reinforcement or in the case of rough interfaces no or only small amounts of reinforcement crossing the interface are present. A noticeable amount of reinforcement (≥ 0.05%) on the other hand allows for larger deformations until the ultimate failure load is reached, i.e. a "non-rigid" bond-slip behaviour where friction forces and dowel action are activated due to the slip.

For post-installed reinforcement or connectors with circular cross-sections the interaction factor κ may be assumed equal to 0.5 for rough and very rough interfaces (for background information reference is made to *Randl, Wicke, Beton- und Stahlbetonbau 08/2000*).

Depending on bond strength, roughness of the interface and degree of reinforcement either adhesive bonding and mechanical interlocking or the other mechanisms of friction and dowel action will dominate the overall load-bearing behaviour. Therefore two situations can be distinguished in general:

- strong adhesive bonding → "rigid" bond – slip behaviour:
  adhesive bonding and interlocking are the main contributing mechanisms to shear resistance
- weak adhesive bonding → "non-rigid" bond – slip behaviour:
  friction and dowel action are the main contributing mechanisms to shear resistance

The single mechanisms can be summarized and approached with the following formulae:

Adhesion / interlocking + friction:

$$\tau_{u,1} = \tau_c + \mu \cdot \left(\sigma + \kappa \cdot \rho \cdot f_y\right) \qquad (6.3\text{-}3)$$

where:

κ is interaction ("effectiveness") factor

The tensile force in the reinforcement / connectors may be limited due to simultaneous bending and / or reduced anchorage of the bars: $\kappa = \sigma_s / f_y \leq 1.0$

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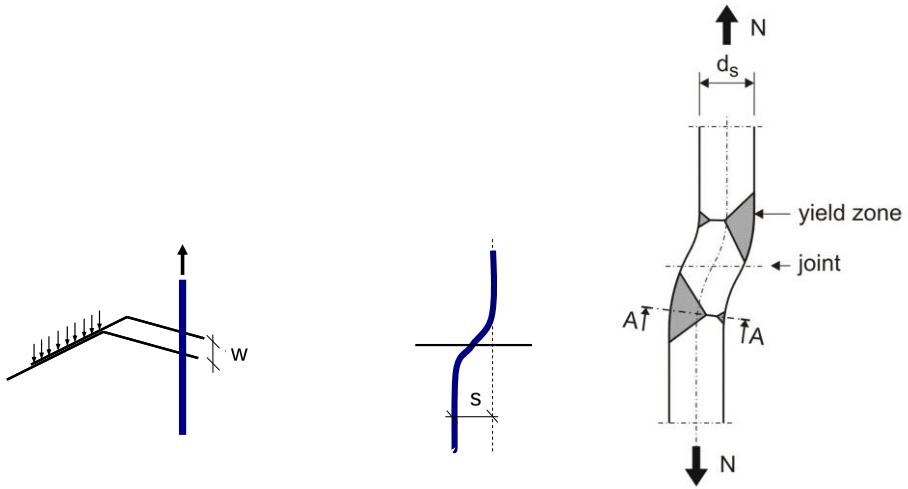


Figure 6.3-4: Clamping force and dowel action of reinforcement / connectors

Depending on the strength of adhesive bonding, the degree of reinforcement and the kind and intensity of roughening either τ_c is decisive (rigid bond-slip characteristics) or the other mechanisms (friction, dowel action) may deliver the main contribution.

Dowel action

The bending-resistance of the reinforcement or connectors can be approximated as follows:

$$F(s) \approx F_{0,\max} \cdot \left(\frac{s}{s_{\max}}\right)^{0,5} = k \cdot A_s \cdot \sqrt{f_{cc}} \cdot \sqrt{f_y} \cdot \left(\frac{s}{s_{\max}}\right)^{0,5} \qquad (6.3\text{-}4)$$

where:

s_{max} is slip when $F_{0,max}$ is reached: $s_{max} \approx 0.10 - 0.20\ d_s$

$k \sim 1.6$ for circular cross-sections, C ≤ C50/60

Interaction of tensile forces and bending leads to a reduction of the maximum possible dowel action, especially when the surface is rough so that substantial tensile forces are generated in the reinforcement / connectors. The reduction of the shear strength can be approached by multiplying $F_{0,max}$ with an interaction factor taking into account the degree of utilization (κ):

$$F_{\max} = F_{0,\max} \cdot \sqrt{1 - \left(\frac{\sigma_s}{f_y}\right)^2} = F_{0,\max} \cdot \sqrt{1 - \kappa^2} \qquad (6.3\text{-}5)$$

Superposition

When superposing the different mechanisms the following aspects have to be taken into account:

- the different mechanisms interrelate and thereby influence each other,
- interaction of tension and bending in the connectors leads to an interrelationship between clamping force and dowel action,
- maximum contributions of different mechanisms occur at different slips.

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For design this implies that either τ_c is the main contributor in the interaction formula, or τ_c becomes rather small or even disappears compared to the other mechanisms of friction and dowel action.

For background information to Eq. 6.3-6 and appropriate parameters depending on different interface roughness characteristics reference is made to *Randl & Wicke, Beton- und Stahlbetonbau 08/2000*.

Tests performed with large scale specimen (e.g. beam tests) indicate that with rough interfaces (good bonding provided) under dynamic loading usually no significant strength reduction occurs compared to monolithic beams subject to fatigue loading.

The ultimate shear stress at the interface resulting from the different single mechanisms can be described in an overall simplified approach as follows:

$$\tau_u = \underbrace{\tau_c}_{\text{Adhesion / interlocking}} + \underbrace{\mu \cdot \left(\rho \cdot \kappa \cdot f_y + \sigma_n\right)}_{\text{friction}} + \underbrace{\alpha \cdot \rho \cdot \sqrt{f_y \cdot f_{cc}}}_{\text{Dowel action}} \leq \beta \cdot \nu \cdot f_{cc} \quad (6.3\text{-}6)$$

where

ρ is degree of reinforcement crossing the interface ($\rho = A_s/A_2$).

The interaction factors κ and α in Eq. (6.3-6) take into account that

– the reinforcement or connectors are subject to bending and axial forces simultaneously,
– the maximum values of the different contributions occur at different slips.

Depending on the roughness and expected deformation characteristics as well as the anchorage of the reinforcement / connectors the following values can be adopted:

For cast-in reinforcement with appropriate end anchorage in the old and the new concrete the tensile loading of the bars may be considered as predominant, i.e. $0.5 < \kappa \leq 1.0$; the dowel action capacity in turn reduces. For simplified design purposes the following limiting values may be used: $\kappa \leq 1.0$ and $\alpha = 0$ (corresponding design values are given in subclause 7.3.3.6).

For post-installed reinforcement or connectors a factor $\kappa = 0.5$ shall be taken when the interface has been roughened. For smooth interfaces dowel action is the main resistance mechanism resulting from reinforcement or connectors, i.e. $\kappa = 0$ and $\alpha = 1.5$ (corresponding design values are given in subclause 7.3.3.6).

Additional design recommendations for interfaces subject to fatigue loading:

The main question concerning interface shear behaviour under fatigue loading is whether or not cracks along the interface are to be expected. In the case of monolithic behaviour (good bonding provided) the overall behaviour of the member can be judged according to subclause 7.4.1. However, as a

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However, if the adhesive bond resistance is exceeded under characteristic load (or likewise 50% under fatigue loading), cracks are likely to occur. In that case due to the relative displacements in the crack more significant deteriorations have to be considered under fatigue loading:

- reduction of mechanical interlocking due to crushing of protruding aggregates,
- reduction of friction due to an increasing amount of fine grains,
- local plasticizing and deterioration of concrete around reinforcing bars / connectors crossing the interface.

In the case of slabs a total overall collapse of adhesion and interlocking effects does not have to be taken into account so that the minimum reinforcement may be reduced to ~50%.

simple approximation a reduction of τ_c to 50% under fatigue loading may be assumed.

As soon as cracks appear, they tend to cause more significant deteriorations along the interface under dynamic loading than observed with monolithic concrete members. When cyclic shear loading is expected an overall reduction of all contributing mechanisms to about 40% of the static resistance according to Eq. (6.3-6) is recommended.

6.3.5 Detailing

Arrangement of reinforcement / connectors

Reinforcement / connectors crossing the interface are required if the shear load in the interface under consideration cannot be resisted by the adhesive / interlocking effect of the roughened joint surface alone. A stepped distribution of the reinforcement / connectors with respect to the interface shear load distribution may be used. In the case of smooth surfaces the flexibility of the reinforcement / connectors even allows for a redistribution of forces and thus a uniform distribution of the connectors.

Minimum reinforcement

If reinforcement / connectors are required because the shear load in the interface under consideration cannot be resisted by adhesive bonding and interlocking effects alone a minimum amount of reinforcement $\rho_{min} = A_{s,min}/A$ should be foreseen in order to prevent brittle failure at loss of adhesion:

beam members: $\rho_{min} = 0.20\, f_{ctm}/f_{yk} \geq 0.001$

slabs: $\rho_{min} = 0.12\, f_{ctm}/f_{yk} \geq 0.0005$

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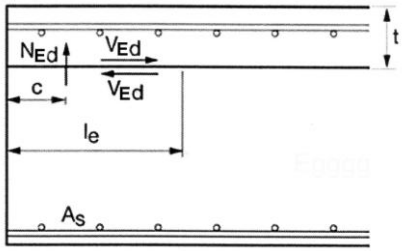


Figure 6.3-5: Edge reinforcement

At the perimeter of a new concrete layer the concrete dries out and tends to contract, thereby producing tensile forces and delaminating perpendicular to the interface, leading eventually to cracking of the concrete. On the load side normally restraint forces along the perimeter are not taken into account because the determination of the effects of differential shrinkage and temperature gradients in a realistic way is difficult without knowing all influencing factors. Therefore appropriate edge reinforcement is highly recommended.

The thickness of the additional concrete overlay is usually limited and lower than the height of the existing concrete member. Moreover, especially in bridge design, the requirements concerning the rather large concrete covers for bridges have to be fulfilled. Therefore the connectors normally need a special kind of end anchorage at the upper end. This end anchorage has to be designed in such a way that premature pull-out or concrete cone breakout can be excluded.

Edge reinforcement

In order to counteract possible delaminating of the concrete overlay, reinforcement / connectors shall be installed all along the perimeter if other adequate structural provisions are not taken. In absence of more exact calculations, as an upper limit the tensile cracking force of the new concrete overlay may be introduced as the maximum shear force to be expected along the edges, and the reinforcement / connectors may be designed accordingly:

$$V_{Ed} = t \cdot b \cdot f_{ct,eff} \qquad (6.3\text{-}7)$$

where:

t is thickness of the new concrete layer,

b is the width of the interface.

Anchorage of reinforcement / connectors

The connectors have to be anchored appropriately in the old as well as in the new concrete in order to avoid premature pull-out failure.

If post-installed reinforcement or connectors are used in combination with the design approach based on the provisions given in subclause 6.3.4 and subclause 7.3.3.6 the tensile force to be anchored may be assumed as:

$$N_{Ed} = \kappa \cdot A_s \cdot f_{yd} \qquad (6.3\text{-}8)$$

For detailed design of the anchorage loaded by N_{Ed} the possible failure modes known for bonded anchors have to be checked (see also chapter 6.4).

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6.4 Concrete to steel

6.4.1 Classification of interaction mechanisms

Use of members with components made of concrete and steel, both structural and cold formed, leads to take properly account of interaction mechanisms.

A variety of details exist depending on the type of members to be connected, the actions to be transmitted and the design performance requirements.

A fundamental classification can be made according to the nature of interaction that is needed:

- mutual restraint between steel and concrete members and/or substructures;
- interconnection between components which allows steel and concrete components to behave as a single structural member.

In former case, steel members are connected to concrete via mechanical devices and activate localised actions in the concrete matrix.

In the latter, stiffness and strength of interface leads to activate the composite behaviour of the member and/or of the structure; both distributed and local interaction are of interest.

Interaction between concrete and steel components can be classified as follows:

- adhesion – pure bond – between two materials;
- frictional interlock provided by peculiar shapes of the interface profile;
- mechanical interlock provided by specific treatments and deformations of the steel interface (i.e. indentations and embossments);
- dowel action provided by anchor devices and systems.

Reference to different types of interaction mechanisms depends on the type of members and/or structure and on the type of loading.

Relevant standards can be referred for design under gravity and horizontal forces.

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6.4.2 Bond of metal sheeting and profiles

Combination of concrete and steel members requires development of composite action that involves extended interface surfaces.

This is the case of composite encased members and of slabs made of metal sheeting and reinforced concrete slabs.

Interaction is aimed to prevent interface slips and ensure the fulfilment of elastic and/or plastic analysis of cross sections under both flexural and axial forces; thus it strongly influences the overall structural response of members either in the load introduction zones or in the critical regions, where high internal forces develop.

6.4.2.1 Metal sheeting

Use of metal sheeting is common to build composite slabs and take advantage of the load bearing capacity of cold-formed steel section during concrete hardening phase.

Relevant standards apply to manufacture and design steel sheeting as shuttering; relevant rules also for definition of minimum nominal thickness of steel sheeting.

Pure bond is not suitable to bear shear forces to develop composite action, thus plain sheeting is basically inappropriate for composite application.

Conversely, mechanical interlock provided by deformations in the steel profile, i.e. indentations and embossments, can be used.

An alternative solution that fits design requirements of composite members is represented by sheeting with re-entrant cross section, so that frictional interface stresses can take advantage of lateral confinement actions due to shrinkage and flexural deformation.

Mechanical connecting devices can be used in order to prevent end relative slip between the two components; deformation of end steel webs can be used as an alternative to additional steel devices when re-entrant profiles are used.

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6.4.2.2 Steel profiles

A variety of solution can be used to increase performance of steel profile under generalised (axial force, moment and shear) external actions.

Structural performances of members can be increased either fully encasing steel profile or filling hollow steel sections.

In both cases, effectiveness of steel-concrete interface is critical in view of composite action development. Adequate structural performances are in fact needed in order to ensure a composite behaviour of the members and prevent local buckling of steel components.

Relevant standard can be used to fit requirements concerning the aspect ratio of steel components and/or flanges depending on the steel grade.

Structural demand on steel and concrete interface depends on the type of cross section, design level of composite interaction, and on the load introduction mechanisms and details adopted.

Pure bond is not suitable in the case of partially encased members under flexure. Compressive members and composite columns can take advantage of such an interaction mechanism, thus only segments inside the load transfer regions are critical.

6.4.2.3 Interface strength

Interface strength is dependent upon the type of active mechanisms: chemical adhesion, interface interlock and active friction.

Chemical adhesive bond between concrete and steel has a strength around 0.1 MPa and occurs over the total area between steel profile and the adjacent concrete. This contact area is higher in the case of trapezoidal ribs and low deformed ribs. Adhesion obviously drops as slippage starts and cannot be recovered, due to this characteristic elastic full interaction theory can be used to perform interface stress estimations and checks, if needed.

Interface interlock is related to local passive restraint given by embossments and indentations of steel profile. Strength is then strongly influenced by the shape of ribs, thickness of the sheeting, size and frequency of embossments. Strength of this mechanism can range virtually from zero

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for smooth trapezoidal ribs without embossment to 0.8 MPa for deformed ribs with embossments. Strength levels are related to the area of steel rib that is in contact with the concrete.

Active friction provides a contribution to shear strength depending on the normal forces applied across the interface and by the surface treatments that governs the value of the friction coefficient. Coefficient of friction between steel and concrete can range from very low values, virtually zero, to 0.6 when the interface is not greased.

Commonly values of active frictional strength are about 0.003 MPa, much lower than values given provided by adhesion and interface interlock. However, its contribution generally is not negligible, since it develops all over the interface surface.

Relevant codes give detailed data concerning characteristic strength values to be used in specific design contexts.

6.4.2.4 Shear stress-slip relationships

Interaction shear stress-slip relationship is dependent upon surface properties of the steel-concrete interface.

If indentations and surface treatments are not present, constitutive relationship is very similar to a rigid plastic behaviour, in fact as soon as chemical bond is exceeded, large slip start with a residual stress related to development of friction.

When indented interfaces are concerned, typical shear bond interaction exhibits a strength increase until the ultimate (peak) shear stress value and a consequent descending curve until a stabilised stress value is reached due to friction interaction again. During this phase steel deck stiffness (related to thickness and rib height) plays an important role.

For design purposes, on the analogy with similar interaction phenomena (i.e. bond of smooth reinforcing bars), theoretical stress strain relationships can neglect peak stress response and aim to set a plastic plateau at the residual stress level.

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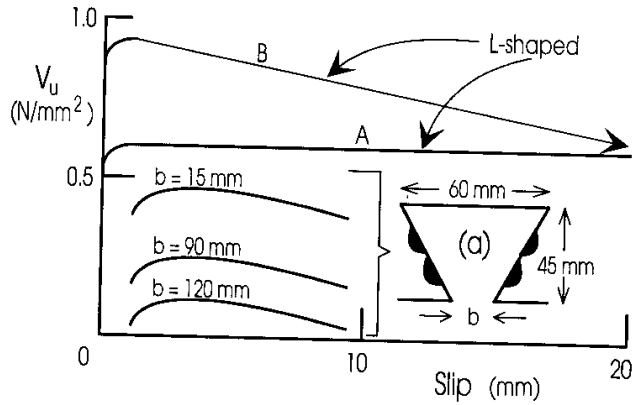


Figure 6.4-1: Shear stress-slip relationship for different types of sheeting

6.4.2.5 Influence of the type of loading

Interface structural performances are influenced by the type of loading. Assessment of actual influence of repeated loading and sustained loading are not fully established.

Long term phenomena that develop in the concrete components should be taken into consideration, since can influence the performances of interface, especially when concrete filled members are used.

Repeated loading, i.e. fatigue, seismic, lead to stress and slip demands at interface level and should be taken into careful consideration.

Damage tolerance at interface level becomes critical for specific applications.

6.4.2.6 Determination of properties by testing

Shear stress-slip constitutive relationships can be obtained from two main types of experimental tests:

- full-scale members flexural tests;
- shear bond elemental tests.

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Relevant codes can be referred in order to fulfil experimental requirements on type 1 assessment of interface structural performances, i.e. steel and concrete composite slabs.

As shear bond elemental tests are concerned, experimental setup are similar to those used for connection devices. Average forces and slip measured at the steel-concrete interface enable to define local constitutive relationship. The latter depends on the interface normal stresses depending on the type of the composite member, thus specific force controlled devices are to be used in order to reproduce realistic interface stress conditions.

6.4.3 Mechanical interlock

Mechanical interlock is generally used load transfer into concrete components and structures or to connect element together.

Anchors can act either as single component or as groups when common fixtures are used. Load transfer mechanisms can be either statically determined or statically undetermined.

Relevant codes of practice are to be referred for design and constructional detailing of fixtures.

Reliable fastening applications require qualified and experienced designers, installation carried out by experienced and skilled personnel, proper maintenance of the structure, of the fixtures and of anchorages. Furthermore, specified use of fastenings should not be changed for the worse during its intended service life unless recalculation.

Construction drawings should clearly show at least the following information:

- location of the anchorage in the structure including tolerances;
- number and type of anchors, including embedment length;
- spacing and edge distance of the anchors, including (positive) tolerances;
- position of the attachments on the fixture including tolerances;
- maximum thickness of eventual non bearing layers below the fixture;
- any special installation instruction.

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6.4.3.1 Classification of devices

Fastening of steel to concrete may be classified as follows:

– cast-in-place devices;

– post-installed devices.

Cast-in-place are positioned in the formwork before the concrete is cast and thus can be used in members with dense reinforcement.

Post-installed systems may either be installed into drilled holes (drill installation) or be driven into the base material with impact energy (direct installation).

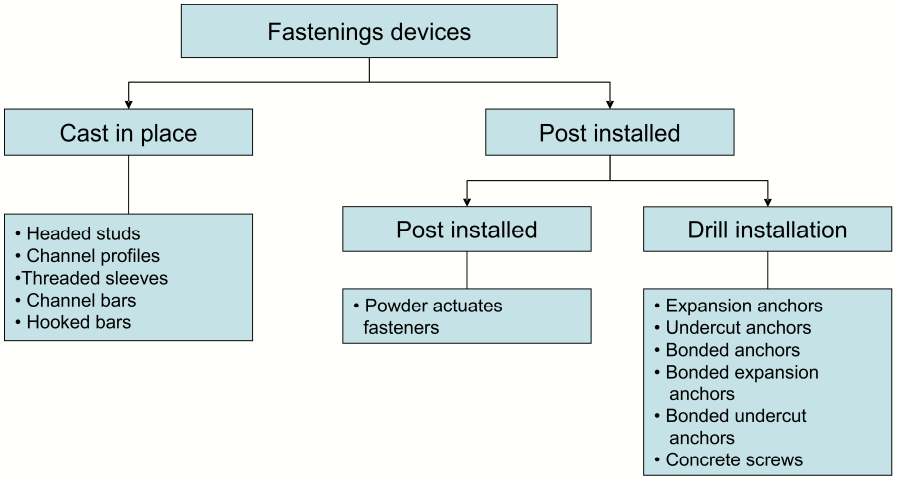


Figure 6.4-2: Summary of fastening techniques in concrete.

Different mechanisms can be activated to transfer tensile loads from the steel anchor to the surrounding concrete matrix:

– friction;

– mechanical interlock;

– dowel action;

– bond.

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Cast-in-place systems like headed studs, hooked bars and channel bars transfer loads mainly by mechanical interlock. Similar mechanisms are activated in the case of undercut anchors, while in the case of expansion anchors, tensile forces are transferred to concrete basically to friction.

6.4.3.2 Strength evaluation

Strength of anchors depends basically on the type of loading. Axial loads are induced by fixtures subjected to axial forces and bending moments.

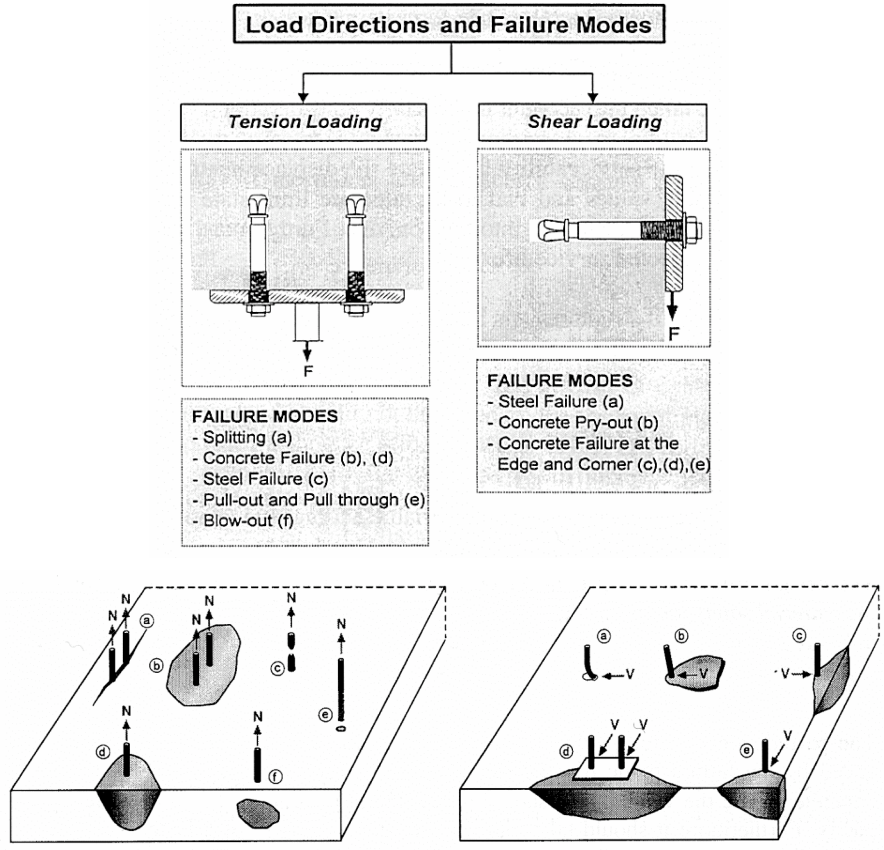


Figure 6.4-3: The most relevant failure modes of anchors.

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A summary of the most relevant failure modes of anchors is reported in Figure 6.4-3.

There are several types of failure modes which are exhibited by anchors when they are loaded beyond their capacity:

- steel breakage;
- concrete cone failure;
- concrete splitting:
- edge failure;
- bond failure, or pullout (including slip or pull through).

Strength and failure mode of anchors are related to the concrete strength, depth of embedment, loading type, loading direction, edge distance or anchor spacing.

Due to several causes tensile stresses in concrete can exceed the corresponding strength, thus fasteners and connectors are often installed in cracked concrete regions. The influence of concrete cracking on the anchor performances depends on the type of anchor.

Only fasteners with a predictable behaviour in cracked concrete have to be used; the anchor suitability has to be proven on experimental basis by means of pre-qualification tests carried out according to relevant codes.

Characteristic resistance of anchors can be based on the computation or test evaluation of the steel tensile and shear resistance, the concrete breakout tensile and shear resistance, the concrete splitting resistance and the tensile pull-out resistance of the anchors.

The models should take into account factors affecting strength such as embedment depth, spacing and edge distance, depth of structural members as well as the presence or the absence of concrete cracking. Limits on edge distance and anchor spacing in the design model should be consistent with the set of reference tests.

Interaction of tensile and shear loads should be considered in design using interaction curves that results in prediction of strength in substantial agreement with results of comprehensive tests.

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Design equations given by relevant codes are to be properly used depending on the type of anchors and the nature of the reinforced concrete components or members to be connected.

Connection devices are widely used in composite constructions, where structural combination of concrete and steel is made basically through shear type (parallel to the interface) force exchange. As a result, commonly fastenings in composite construction are defined 'shear connectors'.

Actually force components perpendicular to the interface arise during loading process of composite members, but can play a secondary role if specific detailing of devices and connection system are provided.

Shear strength of headed stud connectors, which represent the most common device for composite construction, is basically derived from empirical correlations between relevant parameters and results of experimental test programs:

$$P = P(A, f_t\ , f_c\ , E_s\ , E_c\ ) \tag{6.4-1}$$

where:

A is the connector equivalent cross section

f_c is the cylindrical compressive strength of concrete

E_c is the concrete elastic modulus

f_t the steel ultimate tensile stress

E_s is the steel elastic modulus

Due to moderate variation of E_s, the connector shear strength depends basically on three parameters.

Statistical analysis of data give the following characteristic shear strength of headed stud connectors in solid slabs:

$$P_k = \left(5.3 - \frac{1.3}{\sqrt{n}}\right) \cdot A \cdot f_c^{0.35} \cdot f_t^{0.65} \cdot \left(\frac{E_c}{E_s}\right)^{0.4} \tag{6.4-2}$$

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It is worth noting that strength of headed studs is influenced by:

- ultimate axial resistance of studs ($A f_t$)
- interaction between steel and concrete via the ratio between elastic moduli and resistances;
- the number of connectors; in fact factor $\left(5.3 - 1.3 / \sqrt{n}\right)$ depends on the number n of the studs in the group. Assuming a very large number of connectors (n tends to infinity) the factor 5.3 gives the mean shear strength of connectors.

The following limits of the parameter affect the equation reported above:

$10000 \text{ MPa} \leq E_c \leq 33000 \text{ MPa}$

$430 \text{ MPa} \leq f_t \leq 640 \text{ MPa}$

$24 \text{ MPa} \leq f_c \leq 81 \text{ MPa}$

Headed studs are furthermore characterised by an aspect ratio $h_c / d \geq d$, and a height of the weld equal to $0.31\, d_c$. Collapse of concrete due to cracking and pull-out are not covered.

If composite decks are used, the studs are placed within a rib, and their performance is fairly different than in the previous case of solid concrete slab. Completely different are the concrete stiffness, degree of confinement, and the resistance mechanism of the studs, which are loaded eccentrically.

The prime parameters affecting the stud behaviour are:

- the orientation of the ribs relative to the beam span;
- the rib geometry as characterized by the b_r / h_r ratio;
- the stud height H_{sc} relative to the rib height h_r;.

Available data are not suitable for a comprehensive design method to be developed.

In codes, the effects of the main factors are accounted for via a suitable reduction factor that ranges between 0.4 and 1.0 depending on the geometry of sheeting and studs dimensions and location.

Relevant codes provide limits to use of above relationship in terms of b_r / h_r ratio and height of the headed stud compared to height of profiled sheeting.

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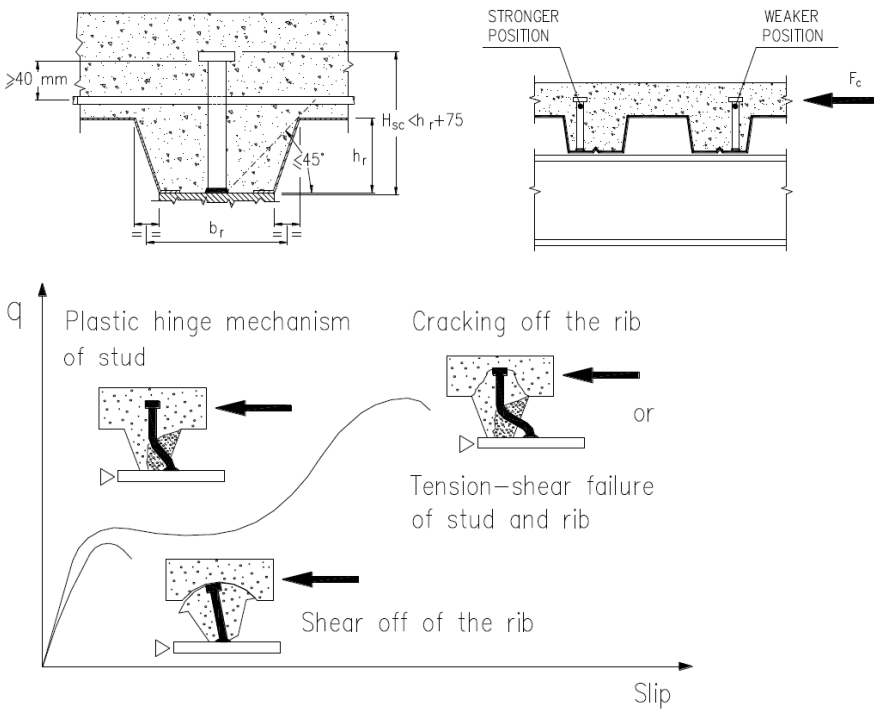


Figure 6.4-4: Sheeting geometry vs. headed studs performances.

6.4.3.3 Force slip constitutive relationships

Strength of connection between concrete and steel members may be influenced by the capacity of the anchors to redistribute loads among the devices connected by fixtures.

This circumstance points out the relevance of force-slip relationships on the overall performances of the composite sub-structure.

Structural relevant aspects related to force-slip relationship are:

– Stiffness: a shear connection realizes either full interaction (the connection is "rigid" and no slip occurs under stress at the steel-concrete interface) or partial interaction (the connection is flexible and interface slip occurs).

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– Resistance: strength is required to the single connector or to the group of connectors in order to enable in a plastic approach the full transfer of forces related the assumed structural mechanism.

  If composite constructions are of interest, shear strength sufficient to make the composite structural element (beam or slab) to develop its ultimate flexural resistance before collapse is achieved gives the so-called full connection condition.

  Conversely, the connection is a partial connection. A structural element with full shear connection is a fully composite structural element. A structural element with partial shear connection is a partially composite structural element. The ratio F_c / F between the resistance of the shear connection and the minimum resistance required by the full connection condition defines the degree of shear connection.

– Ductility: a connection is ductile if its deformation (shear and/or pull-out) capacity is adequate for a complete redistribution of the forces acting on the individual connector. In composite constructions, the ductility demand depends on the member length and the degree of shear connection.

Figure 6.4-5 reports a number of force-slip relationships of shear connectors commonly used in composite construction.

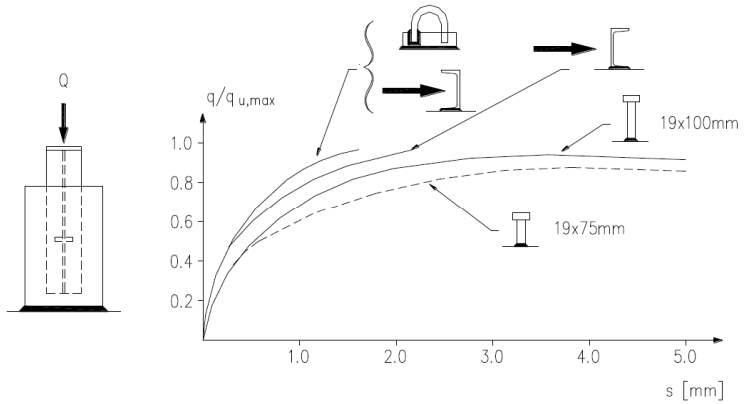


Figure 6.4-5: Non-dimensional force slip relationships for shear connectors

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Generally, all types of mechanical shear connectors possess a limited deformation capacity. However, in several instances, the associated slip is sufficient to make the design flexural resistance and rotation capacity of the composite section to be developed.

Relevant codes give provisions about ductility requirements related to span and the degree of shear connection and to the characteristic values of slip capacity for design.

Modelling of force-slip relationship can be carried out evaluating specific tests on connectors that can be influenced by the interface greasing of the steel profiles. In fact, due to chemical adhesion, force-slip relationships can show an initial rigid (zero slip) branch and then a very steep curve. Conversely, presence of greasing at interface enables to generate a response completely depending on the dowel action.

If deformation of headed connectors in solid slabs are concerned, four relevant parameters can indicated:

- strength of the stud, examined in the previous section;
- ultimate slip s_u, corresponding to the maximum load P of the stud;
- service slip s_s, corresponding to a half of the maximum load;
- slip capacity s_c evaluated on the post-peak descending branch of the relationship at 95 % of the peak shear load;

Statistical analyses, on the analogy with results discussed for stud resistance, give the following relationship providing the ultimate slip and the slip capacity respectively:

$$s_u = (0.389 - 0.0023 \cdot f_c) \cdot d_c \qquad (6.4\text{-}3)$$

$$s_c = (0.453 - 0.0018 \cdot f_c) \cdot d_c \qquad (6.4\text{-}4)$$

Results are relevant for a range of concrete strength between 20 MPa and 70 MPa, d_c being the stud nominal diameter.

Refined modelling of composite construction can take advantage of theoretical curves that relate force and slip relationships of studs:

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$$P = P_{\max} \cdot \left(1 - e^{-\beta \cdot s} \right)^{\alpha} \tag{6.4-5}$$

where:

– s is the generic slip (mm);
– $P_{\max}$ is the stud strength;
– α and β are the constants that enable to characterise the curve.

In particular, the first parameter α governs the initial stiffness, and the second one β influences the shape of the curve by a proper scaling of the slip.

The first one ranger between zero and unity.

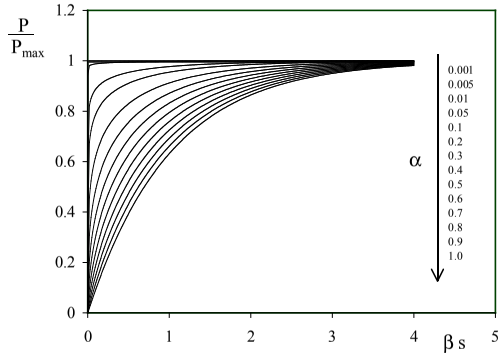


Figure 6.4-6: Force-slip relationship depending on α

Based on experimental results, α ranges between 0.5 and 1, while β ranges between 0.7 and 1.5.

An alternative formulation can be derived from the well-known formulation of bond-slip relationship of reinforcement in concrete, that fits properly the basic requirements of experimental force-slip relationships:

$$P(s) = P_{\max} \cdot \left(\frac{s}{s_u} \right)^{\gamma} \tag{6.4-6}$$

The shape of the curve is governed by g exponent, that plays a role similar to the parameter a used in the preceding formulation and by the slip at peak load, that is conceptually analogous to the product $\beta\, s$, namely an equivalent slip s_{eq}, but enables an easier interpretation.

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If serviceability and deflection analysis of composite members are of interest, a linear approximation of the stress slip relationship can be used. Different definitions of linear stiffness of connectors can be referred; a possible evaluation can be based on the secant stiffness evaluated in correspondence of the slip and the shear force under serviceability conditions, generally about 50-60% of the resistance.

6.4.3.4 Influence of the type of loading

Fatigue loading of the structural members serving as base material or of the anchorage may be allowed for certain anchors, if this is stated in the relevant approval certificate or if it has been shown in the prequalification procedure that fatigue loads can be taken up by the anchor. In both cases the corresponding conditions (e.g. permanent pre-stressing force or sufficient magnitude) and the allowable load should be met in the design.

6.4.3.5 Determination of properties by testing

Relevant standards can cover need of design equation that relate material and geometrical parameters to strength of mechanical devices.

As equation do not cover selected anchors, experimental tests can supply design data.

The variables to be investigated include the geometry and the mechanical properties of the concrete slab, the connectors and the reinforcement.

The resistance to loading, other than fatigue, may be determined by specific tests in accordance with relevant standards.

In general, the test rigs should allow the formation of an unrestricted rupture cone. For this reason the clear distance between the support reaction and an anchor (single anchor) or an outer anchor (anchor group) shall be at least twice the effective depth of the anchor or twice the distance between the anchor and the edge in load direction. Only in shear tests without edge influence where steel failure is expected.

During all tests, the load shall be applied to the anchor by a fixture representing the conditions found in practice.

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In tests on single anchors without edge and spacing influences the centre-to-centre distance and the distances from free edges shall be large enough to allow the formation of an unrestricted rupture cone of vertex angle 120° in the concrete.

In the case of shear connectors for composite constructions, specific push tests should be carried out such that the slabs and the reinforcement are suitably dimensioned in comparison with the beams for which the test is designed.

Thus, the length l of each slab should be related to the longitudinal spacing of the connectors in the composite structure; the width of each slab should not exceed the effective width of the beam's slab; the thickness of each slab should not exceed the minimum thickness of the slab in the member.

Measure of applied load and relative displacements between the single anchor and the base concrete component should be taken. The slip should be measured at least until the load has dropped to 20% below the maximum load. Average displacements can be used as reference force-slip relationships.

The slip capacity of a specimen should be taken as the maximum slip measured at the characteristic load level. The characteristic slip capacity should be taken as the minimum test value of slip capacity reduced by 5% or determined by statistical evaluation from all the test results.

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