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Subject to priorities defined by the Technical Council and the Presidium, the results of ***fib***'s work in Commissions and Task Groups are published in a series of technical publications called 'Bulletins'.

category	minimum approval procedure required prior to publication
Technical Report	approved by a Task Group and the Chairpersons of the Commission
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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.
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* winning structure, 2006 ***fib*** Awards for Outstanding Concrete Structures
** winning structure, 2010 ***fib*** Awards for Outstanding Concrete Structures

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Preface

The International Federation for Structural Concrete (***fib***) is a pre-normative organization. "Pre-normative" implies pioneering work in codification. This work has now been realized with the ***fib*** Model Code 2010. Earlier Model Codes from the ***fib***'s parent organizations were published as CEB-FIP Model Codes 1978 and 1990. The objectives of the ***fib*** Model Code 2010 are to (a) serve as a basis for future codes for concrete structures, and (b) present new developments with regard to concrete structures, structural materials and new ideas in order to achieve optimum behaviour.

Structural concrete is more than a continuously developing material. It also represents a remarkable development in design concepts and strategies. Requirements for concrete structures have often been formulated as follows: concrete structures should be resistant, serviceable, durable, economic and aesthetic. Today, several further requirements or expectations regarding concrete structures have to be met, for example: they should be robust enough to avoid progressive collapse, should need only minimal maintenance, should be able to embed waste materials, should provide protection against accidents, should provide barriers against or following hazards, should be reusable or at least recyclable, should support sustainability in all possible ways, and in addition, provide adequate fire and earthquake resistance and be environmentally compatible.

The ***fib*** Model Code 2010 includes the whole life cycle of a concrete structure, from design and construction to conservation (assessment, maintenance, strengthening) and dismantlement, in one code for buildings, bridges and other civil engineering structures. Design is largely based on performance requirements. The chapter on materials is particularly extended with new types of concrete and reinforcement (such as fibres and non-metallic reinforcements).

The ***fib*** Model Code 2010 – like the previous Model Codes − does not only specify requirements but also gives the corresponding explanations in a separate column of the document. Additionally, MC2010 is supported by background documents that have already been (or will soon be) published in ***fib*** Bulletins and journal articles.

The ***fib*** Model Code 2010 (MC2010) was produced during the last ten years through an exceptional effort by 44 countries from five continents: *Argentina, Australia, Austria, Belgium, Belarus, Brazil, Canada, China, Croatia, Cyprus, Czech Republic, Denmark, Egypt, Estonia, Finland, France, Germany, Greece, Hungary, India, Iran, Israel, Italy, Japan, Luxembourg, the Netherlands, New Zealand, Norway, Poland, Portugal, Romania, Russia, Serbia, Slovakia, Slovenia, South Africa, South Korea, Spain, Sweden, Switzerland, Turkey, Ukraine, United Kingdom, USA*. The General Assembly of ***fib*** accepted Model Code 2010 on 29 October 2011 in Lausanne, Switzerland.

On behalf of ***fib*** we would like to acknowledge the efforts of all those who contributed to the drafting, correcting or editing of the MC2010 text, including the members of the ***fib*** Special Activity Group 5, *New Model Code*, and also the ***fib*** Commissions and Task Groups (see the names listed on the following pages). Special thanks are owed to Agnieszka Bigaj-van Vliet for her work as technical secretary and to Laura Thommen-Vidale for her editorial help.

We believe that the ***fib*** Model Code 2010 provides an extraordinary contribution to the advancement of knowledge and technical developments in the field of design and assessment of concrete structures.

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In addition to the work realized by the members of ***fib*** Special Activity Group 5 (listed on page ii), the members of the other ***fib*** Commissions, Task Groups and Special Activity Groups have made important contributions to the content of the Model Code 2010 during the past years. The current members of these groups are given below.

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SAG 6, Composite steel-concrete construction

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Notations

Meaning of Roman capital letters

A	area
C	torsional moment of inertia; serviceability constraints
D	fatigue damage factor; diffusion coefficient
E	modulus of elasticity; earthquake action; load (action) effect
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
M	bending moment; coefficient of water absorption; safety margin
N	axial force
P	force
Q	variable action
R	resistance; strength (resisting load effect); reaction at a support; resultant
S	static moment of a plane area
T	torsional moment; temperature
V	shear force, volume
W	modulus of inertia
X	material or soil properties in general; 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; sets of loads (actions)
f	strength
g	distributed permanent load; acceleration due to gravity; limit state function
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
p	prestressing
q	distributed variable load
r	radius; resistance variables; resistance function
s	spacing; standard deviation of a sample
t	time; torsional moment per unit length or width; thickness of thin elements
u	perimeter
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

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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
epsilon	ε	strain
zeta	ζ	coefficient
eta	η	coefficient
theta	θ	rotation
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
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
phi	φ	coefficient
chi	χ	coefficient
psi	ψ	coefficient; ratio
omega	ω	mechanical percentage of reinforcement

Mathematical symbols and special symbols

S	sum
Δ	difference; increment (enlargement)
Ø	nominal 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.

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

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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; yield
z	linear co-ordinate
1, 2, 3	particular values of quantities
cc	conventional asymptotic value

*: 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
cs	shrinkage of concrete
ep	earth pressure
ex	explosion; blast
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

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
ext	external
fat	fatigue
inf	inferior
int	internal
lat	lateral
lim	limit

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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
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
$1/r_{0\,(g+q)}$	instantaneous (elastic) curvature due to g and q
$1/r_I$	curvature of an uncracked concrete section (state I)
$1/r_{Ir}$	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	geometrical quantity in general; deformation; deflection
a_d	design values of geometrical quantity
a_0	elastic deflection (calculated with rigidity $E_c\, I_e$)
b	breadth of compression zone or flange, width of concrete section
b_f	width of FRP section
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	concentration of a substance in a volume element; concrete cover; coefficient for shear resistance due to adhesive bond
c_r	coefficient for shear resistance due to aggregate interlock
c_1	column dimension parallel to the eccentricity of the load
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})

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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	strength
f_{bd}	design bond strength
$f_{bd,0}$	basic bond strength
f_{bpd}	design bond strength for prestressing tendon
f_c	cylinder compressive strength of concrete
f_{lc}	cylinder compressive strength of lightweight aggregate 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
$f_{c,\,imp,\,k}$	characteristic compressive strength under high rates of loading
f_{ck}	characteristic value of compressive strength of concrete
$f_{ck,c}$	value of f_{ck} of confined concrete
$f_{ck.cube}$	characteristic value of cube compressive strength of concrete
$f_{ck,fat}$	characteristic value of fatigue reference compressive strength
$f_{ck,ft}$	characteristic value of concrete compressive strength after freeze-thaw attack
f_{cm}	mean value of compressive strength of concrete
$f_{cm,sus}(t,t_0)$	mean value of compressive strength of concrete at time t when subjected to a high sustained compressive stress at an age at loading t_0
f_{ct}	axial tensile strength of concrete
f_{ctd}	design value of f_{ct}
$f_{ct,\,imp,\,k}$	characteristic tensile strength under high rates of loading
f_{ctk}	characteristic value of f_{ct}
$f_{ctk,\,is}$	characteristic measured in-situ tensile strength
$f_{ctk,\,\max}$	upper lower bound value of the characteristic tensile strength of concrete
$f_{ctk,\,\min}$	lower bound value of the characteristic tensile strength of concrete
$f_{ctk,\,sus}$	characteristic tensile strength of concrete under sustained loading
f_{ctm}	mean value of axial tensile strength of concrete
$f_{ct,fl}$	flexural tensile strength (at T = 20°C)
$f_{ctm,fl}$	mean flexural tensile strength (at T = 20°C)
$f_{ct,sp}$	splitting tensile strength
$f_{ctm,sp}$	mean splitting tensile strength
f_d	design value of material or product property; design value of strength

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f_f	tensile strength of non-metallic reinforcement
f_{fad}	design bond strength in the presence of anchorage
f_{fbd}	design value of tension stress in the non-metallic reinforcement limited by bond to concrete
f_{fbm}	mean value of tension stress in the non-metallic reinforcement limited by bond to concrete
f_{fd}	design tensile strength of non-metallic reinforcement
f_{fk}	characteristic value of tensile strength of non-metallic reinforcement
f_{Fts}	serviceability residual strength (post-cracking strength for serviceability crack opening)
f_{Ftsd}	design value of post-cracking strength for serviceability crack opening
f_{Ftu}	ultimate residual strength (post-cracking strength for ultimate crack opening)
f_{Ftud}	design value of post-cracking strength for ultimate crack opening
f_k	characteristic value of material or product property; characteristic value of strength
f_L	Limit of Proportionality
f_{Lk}	characteristic value of Limit of Proportionality
f_{lck}	characteristic value of compressive strength of lightweight aggregate concrete
f_{lcm}	mean value of compressive strength of lightweight aggregate concrete
$f_{lctk,\,\max}$	upper lower bound value of the characteristic tensile strength of lightweight aggregate concrete
$f_{lctk,\,\min}$	lower bound value of the characteristic tensile strength of lightweight aggregate concrete
f_{lctm}	mean value of axial tensile strength of lightweight aggregate concrete
$f_{p0.1}$	0.1% proof strength of prestressing steel
$f_{p0.2}$	0.2% proof strength of prestressing steel
$f_{p0.1k}$	characteristic 0.1% proof strength of prestressing steel
$f_{p0.2k}$	characteristic 0.2% proof strength of prestressing stel
f_{pt}	tensile strength of prestressing steel; UTS (Ultimate Tensile Strength) of prestressing steel
f_{ptd}	design tensile strength of prestressing steel
f_{ptk}	characteristic value of tensile strength of prestressing steel; characteristic value of UTS (Ultimate Tensile Strength) of prestressing steel
f_{py}	tension yield stress of prestressing steel
f_{pyd}	design value of tension yield stress of prestressing steel
f_{pyk}	characteristic value of tension yield stress of prestressing steel
f_r	relative (or projected) rib area
$f_{R,j}$	residual flexural tensile strength of fiber reinforced concrete corresponding to Crack Mouth Opening Displacement ($CMOD$) = $CMOD_j$
f_{R1k}	characteristic residual strength of fiber reinforced concrete significant for serviceability conditions
f_{R3k}	characteristic residual strength of fiber reinforced concrete significant for ultimate conditions
$f_{sp,\theta}$	proportional limit of reinforcing steel at temperature θ
$f_{sy,\theta}$	maximum stress of reinforcing steel at temperature θ

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$f_{0.2}$	0.2% proof strength of reinforcing steel
$f_{0.2k}$	characteristic value of 0.2% proof strength of reinforcing steel
f_t	tensile strength of reinforcing steel
f_{tk}	characteristic value of tensile strength of reinforcing steel
f_{tm}	mean value of tensile strength of reinforcing steel
f_y	yield strength of reinforcing steel in tension
$f_{y,act}$	actual yield strength of reinforcing steel in compression
f_{yc}	yield strength of reinforcing steel in compression
f_{ycd}	design yield strength of reinforcing steel in compression
f_{yd}	design yield strength of reinforcing steel in tension
f_{yk}	characteristic value of yield strength of reinforcing steel in tension
f_{ym}	mean value of yield strength of reinforcing steel in tension
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
h_{sp}	distance between the notch tip and the top of the specimen
Δh_w	height of water column
i	radius of gyration
k	plasticity number; unintentional angular displacement
k_a	effectiveness coefficient of anchorage system
k_b	shape factor
k_{bl}	bond length calibration factor
k_c	coefficient
k_d	effectiveness factor dependent on the reinforcement detail
k_l	stress-strength ratio
k_m	coefficient of confinement from transverse reinforcement
k_n	displacement factor for repeated constant amplitude loading
k_t	displacement factor for permanent load
l	design span, effective span, length of an element, thickness of a penetrated section
Δl	change in distance between two measuring points
l_0	design lap length, effective length (of columns); distance between measuring points
l_b	design anchorage length; design lap length
l_{bp}	basic anchorage length of bonded pretensioned reinforcement
l_{bpd}	design anchorage length of bonded pretensioned reinforcement
l_{bpt}	transmission length of bonded pretensioned reinforcement
$l_{b,\min}$	minimum anchorage length; minimum lap length
$l_{bd,net}$	design anchorage length
l_{cs}	characteristic length (fracture parameter)
l_p	development length for bonded prestressing reinforcement
Δl_{pl}	residual elongation after unloading

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$l_{p,\max}$	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; moisture content
n	number of bars, number of load cycles; force per unit width (in-plane-loading)
n_b	number of anchored bars or pairs of lapped bars in the potential splitting surface; number of bars in the bundle
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
n_t	number of legs of confining reinforcement crossing a potential splitting failure surface at a section
p	local gas pressure; overall steel ductility parameter
p_m	mean pressure
p_{tr}	transverse pressure perpendicular to the bar axis; mean compression stress perpendicular to the potential splitting failure surface at the ultimate limit state
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; coefficient which depends on the strength class of cement
s_m	slip at maximum bond stress
$s_{n,t}$	slip due to permanent or repeated loading
$s_{\max}$	maximum bar spacing
s_r	distance between cracks; radial spacing of layers of shear reinforcement
$s_{r,m}$	mean spacing between cracks
s_t	longitudinal spacing of confining reinforcement
s_u	ultimate slip
t	time, age, duration; thickness of thin elements
t_0	age at first loading
t_1	age of the concrete when its temperature returns to ambient temperature
t_f	thickness of non-metalic reinforcement
t_{eq}	equivalent time interval for calculation of relaxation losses
t_{p1}	mean duration of the heating cycle
t_R	reference period
t_s	concrete age at the beginning of shrinkage or swelling
t_T	temperature adjusted concrete age
u	length of a perimeter; component of displacement of a point
u_0	length of the periphery of the column or distribution area of 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

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w_{lim}	nominal limit value of crack width
w_u	maximum crack opening accepted in structural design: its value depends on the ductility required
x	depth of compression zone; distance; parameter
$x_c(t)$	carbonation depth at the time t
x_d	design value of parameter x
z	internal lever arm

Greek lower case letters

α	coefficient; reduction factor; inclination of reinforcement crossing an interface; sum of the angular displacements
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_{fl}	conversion factor (= f_{ctm} / $f_{ctm,\,fl}$)
α_p	coefficient of thermal expansion of prestressing reinforcement
α_{spl}	conversion factor (= f_{ctm} / $f_{ctm,\,spl}$)
α_{sT}	coefficient of thermal expansion for steel
α_T	coefficient of thermal expansion in general
α_1	coefficient representing the influence of reinforcement provided
α_2	coefficient representing the influence of passive confinement from cover
α_3	coefficient representing the influence of passive confinement from transverse reinforcement
β	coefficient characterizing the bond quality of reinforcing bars, coefficient for the compressive strength of a strut across and interface
β_c	coefficient for the compressive strength of a strut across an interface
$\beta_{bc}(t,t_0)$	coefficient to describe the development of basic creep with time after loading
$\beta_{dc}(t,t_0)$	coefficient to describe the development of drying creep with time after loading
$\beta_{cc}(t)$	coefficient to describe the development of strength of concrete with time
$\beta_{c,sus}(t,t_0)$	coefficient to describe the decrease of strength with time under sustained load
$\beta_E(t)$	coefficient to describe the development of modulus of elasticity of concrete with time
$\beta_{lcc}(t)$	coefficient to describe the development of strength of lightweight aggregate concrete with time
$\beta_{H,\,T}$	coefficient to describe the effect of temperature on the time development of creep
γ	safety factor
γ_c	partial safety factor for concrete material properties
γ_{cb}	partial safety factor for bond
$\gamma_{c,fat}$	partial safety factor for concrete material properties under fatigue loading
γ_d	partial safety factor for partial factors for model uncertainties
γ_f	partial safety factor for the tensile strength of non-metallic reinforcement
γ_F	partial safety factor for actions; partial safety factor for fibre reinforced concrete
γ_G	partial safety factor for permanent actions
γ_m	partial safety factor for material properties
γ_M	partial safety factor for material properties partial safety accounting for the model uncertainties and geometrical uncertainties
γ_Q	partial safety factor for variable actions
γ_{Rd}	partial safety factor associated with the uncertainty of the model and geometrical uncertainties

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γ_s	partial safety factor for the material properties of reinforcing and prestressing steel
$\gamma_{s,fat}$	partial safety factor for the material properties of reinforcing and prestressing steel under fatigue loading
γ_{Sd}	partial safety factor accounting for model uncertainty
δ	shear displacement
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_{c1}	concrete strain at maximum compressive stress
$\varepsilon_{c1,\,imp}$	impact concrete strain at maximum load
$\varepsilon_{cc}(t)$	concrete creep strain at concrete age $t > t_0$
$\varepsilon_{ci}(t_0)$	stress dependent initial strain of concrete at the time of first loading
ε_{cf}	strain at maximum stress due to repeated loads
$\varepsilon_{cn}(t)$	stress independent strain at a concrete age t
$\varepsilon_{cs}(t)$	shrinkage or swelling strain at concrete age t
$\varepsilon_{c\sigma}(t)$	stress dependent strain at a concrete age t
ε_{ct}	concrete tensile strain
$\varepsilon_{cT}(t)$	thermal strain at a concrete age t
ε_{clim}	ultimate strain of concrete in compression
ε_{pd0}	strain of prestressed reinforcement corresponding to P_{d0}
ε_f	strain of non-metallic reinforcement
ε_{fu}	strain of non-metallic reinforcement at maximum force in tension
ε_{fuk}	characteristic value of strain of non-metallic reinforcement at maximum force in tension
ε_{lc1}	lightweight aggregate concrete strain at maximum compressive stress
ε_{lclim}	ultimate strain of lightweight aggregate concrete in compression
ε_{pu}	strain of prestressing steel at maximum force
ε_{puk}	characteristic value of strain of prestressing steel at maximum force
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 due to crack formation in the section
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 reinforcing steel at maximum load
$\Delta\varepsilon_{ts}$	increase of strain by the effect of tension stiffening
ε_u	limit strain value; strain of reinforcing steel at maximum force
ε_{uk}	characteristic value of reinforcing steel strain at maximum force
ε_{yd}	design yield strain of reinforcing steel ($= f_{yd} / E_s$)
ε_v	transverse contraction
ζ	ratio of bond strength of prestressing steel and high-bond reinforcing steel
η	viscosity of gas
η_1	coefficient representing the type of reinforcing bar being anchored or lapped
η_2	coefficient representing the casting position of the bar during concreting

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η_3	coefficient representing the bar diameter
η_4	coefficient representing the characteristic strength of steel reinforcement being anchored or lapped
η_{p1}	coefficient representing the type of prestressing tendon
η_{p2}	coefficient representing the casting position of the tendon
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
κ	coefficient
κ_1	coefficient for axial force in interface connectors
κ_2	coefficient for dowel action resistance of interface connectors
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} / ($A_c f_{cd}$))
ξ	creep induced stress distribution after modification of restraint conditions
ρ	ratio of (longitudinal) tension reinforcement (= A_s / (bd)); density
$\rho_{s,ef}$	effective reinforcement ratio (= A_s / $A_{c,ef}$)
ρ_t	relaxation after t hours
$\rho_t(T)$	relaxation after t hours at temperature T
ρ_{100}	relaxation after 100 hours
ρ_{1000}	relaxation after 1000 hours
ρ_w	ratio of web reinforcement (= A_{sw} / (b_{ws} sinα))
σ	stress
σ_1, σ_2, σ_3	principal stresses
σ_c	concrete compression stress
σ_{cd}	design concrete compression stress
σ_{ct}	concrete tensile stress
$\sigma_{c,c}$	compression stress of confined concrete
$\sigma_{c,\max}$	maximum compressive stress
$\sigma_{c,\min}$	minimum compressive stress
$\sigma_{ct,\max}$	maximum tensile stress
σ_f	stress in non-metallic reinforcement
σ_n	(lowest) compressive stress resulting from normal force acting on the interface
$\sigma_{p0}(x)$	initial stress in prestressing steel at a distance x from anchorage device
$\sigma_{p0,\max}$	maximum tensile stress in prestressing steel at tensioning
σ_{pcs}	stress in prestressing steel after all losses (including creep and shrinkage)
σ_{pd}	tendon stress under design load
$\Delta\sigma$	stress range relevant to fatigue of reinforcement
$\Delta\sigma_{Rsk}(n)$	stress range relevant to n cycles obtained from a characteristic fatigue strength function
σ_s	steel stress
σ_{sd}	steel stress be anchored by bond over the distance l_b
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_0	bond stress according to the bond stress – slip curve
τ_a	ultimate shear capacity due to adhesion or interlocking

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τ_b	local bond stress
$\tau_{b,m}$	bond stress modified in case of bar yielding, transverse pressure and cracking parallel to the bar axis and cyclic loading
$\tau_{fu,d}$	ultimate design shear friction capacity
τ_{max}	maximum value of bond stress
τ_{Rd}	design value of shear strength
τ_{Sd}	applied shear stress (design value)
τ_u	ultimate shear friction capacity
$\tau_{u,split}$	peak value of bond strength in a splitting failure
$\varphi(t,t_0)$	creep coefficient
φ_0	basic creep coefficient
$\varphi_{0,dc}$	drying creep coefficient
$\varphi_{0,k}$	nonlinear notional creep coefficient
φ_l	basic creep coefficient for lightweight aggregate concrete
$\varphi_{RH,T}$	temperature dependent creep coefficient
$\Delta\varphi_{T,trans}$	transient thermal creep coefficient which occurs at the time of the temperature increase
χ	aging coefficient in the evaluation of creep structural effects
$\psi(t,t_0)$	relaxation coefficient
ω	mechanical reinforcement ratio
ω_{sw}	mechanical ratio of shear 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_b	area of single bar
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_d	design value of accidental action
A_{Ed}	design value of seismic action
A_{Ek}	representative value of seismic action
A_{ef}	area enclosed by the centre-lines of a shell resisting torsion
A_p	area of prestressing steel
A_s	area of reinforcement
$A_{s'}$	area of compression reinforcement
A_{sh}	area of hoop reinforcement for torsion
A_{sl}	area of longitudinal reinforcement
A_{sp}	cross sectional area of the tendon
A_{st}	area of transverse reinforcement; cross sectional area of one leg of a confining bar
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
AF	amplification factor
C	serviceability constraints

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C_0	initial chloride content of concrete
C_f	aggregate effectivity factor
$C_{S,\Delta x}$	chloride content at a depth of Δx
D	fatigue damage; diffusion coefficient; deformation
D_{app}	apparent diffusion coefficient of a substance in concrete
D_{eff}	effective diffusion coefficient of a substance in concrete
D_{lim}	limiting fatigue damage
D_{RCM}	chloride migration coefficient
E	modulus of elasticity; load (action) effect; cumulative leaching
E_c	reduced modulus of elasticity for concrete
$E_c(t_0)$	modulus of elasticity of concrete at the time of loading t_0
E_{ci}	tangent modulus of elasticity of concrete at a stress σ_i
$E_{cl}(t)$	modulus of elasticity of concrete at an age $t \neq 28$ days
$E_{c,1}$	secant modulus from the origin to the peak compressive stress
$E_{c,\mathrm{imp}}$	modulus of elasticity of concrete for impact loading
E_d	design action-effect
E_f	modulus of elasticity for non-metallic reinforcement
E_{lc}	reduced modulus of elasticity for lightweight aggregate concrete
E_{lci}	tangent modulus of elasticity of lightweight aggregate concrete at a stress σ_i
E_p	modulus of elasticity of prestressing steel
E_s	modulus of elasticity of reinforcing steel
$E_{s,\theta}$	modulus of elasticity of reinforcing steel at temperature θ
$E_{s,sec}$	secant modulus of elasticity of steel
F	action in general; 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_j	load corresponding to Crack Mouth Opening Displacement ($CMOD$) = $CMOD_j$
F_{pt}	tensile load of prestressing steel
$F_{p,0,\max}$	maximum tensile force in the prestressing steel reinforcement at tensioning
$F_{p0.1}$	characteristic 0.1% proof load
F_{pkT}	characteristic long-term tensile strength of the tendon for declared design life
F_{rep}	representative value of the actions
$F_{Sd,ef}$	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}	basic value of fracture energy (depending on maximum aggregate size)
G_{inf}	favourable part of permanent action
G_{sup}	unfavourable part of permanent action
H	humidity; 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 (excluding reinforcement)
I_e	second moment of area for short-term loading

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$J(t,t_0)$	creep or compliance function representing the total stress dependent strain per unit stress
K	orientation factor
K_g	coefficient of gas permeability
K_{tr}	density of transverse reinforcement
K_s	dissociation constant
K_w	coefficient of water permeability
L	span, length of an element
L_{pl}	plastic length (region in which tensile strain is larger than yield strain)
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_w	coefficient of water absorption
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
Q	variable single action; volume of a transported substance (gas or liquid)
Q_k	characteristic value of variable action
R	resistance (strength); bending radius; universal gas constant
R_a	average roughness
R_{AAC}	inverse effective carbonation resistance of dry concrete determined using the accelerated carbonation test ACC
R_d	design value of resistance
R_k	characteristic value of resistance
R_m	mean value of resistance
R_{NAC}	inverse effective carbonation resistance of dry concrete determined using the normal carbonation test NAC
R_t	peak-to-meanline height (derived from sand patch method)
R_z	mean peak-to-valley height
$R(t,t_0)$	relaxation function, representing the stress response to a unit imposed strain
RH	ambient relative humidity
RH_0	100% relative humidity
S	absorption coefficient
ΔS_c	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)
$S_{ct,max}$	maximum tensile stress level (fatigue loading)
S_d	design load effect (M, N, V, T)

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S_s	slope of the unloading branch of the bond-slip relationship
T	temperature; torsional moment
$T(t)$	temperature at time t
ΔT	temperature change
T_g	glass transition temperature
$T_{\max}$	maximum temperature of the concrete during heat treatment
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_F	resistance of reinforcement to shear force
$V_{F,max}$	maximum resistance of reinforcement to shear force
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 (excluding reinforcement)
$W_{c,c}$	volume of confined concrete
W_e	external work
W_i	internal work
$W_{s,trans}$	volume of closed stirrups or cross-ties
X	value of material and soil properties in general
X_d	design value of material and soil properties in general

Others

ℓ_b	length of bonded area
$\ell_{b,\max}$	value of ℓ_b that, if exceeded, there would be no increase in the force transferred between concrete and non-metallic reinforcement
$Ø$	nominal diameter of 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 reinforcing bars
Ψ_0	coefficient for the combination value of a variable action
Ψ_1	coefficient for the frequent value of a variable action
Ψ_2	coefficient for the quasi-permanent value of a variable action
Ω_{cr}	factor for modified bond in case of cracking parallel to the bar axis
Ω_{cyc}	factor for modified bond in case of cyclic loading
$\Omega_{p,tr}$	factor for modified bond in case of transverse pressure
Ω_y	factor for modified bond in case of bar yielding
Λ_{cyc}	dissipated energy during cyclic loading
Λ_0	dissipated energy during monotonic loading

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

Greek lower case letters:

α	sensitivity factor
β	reliability index
γ	(partial) safety factor
μ	mean value
σ_x^2	scattering or variance
σ_x	standard deviation
σ_R	standard deviation of resistance
σ_S	standard deviation of action
δ_R	coefficient of variation of the parameter under consideration

Roman capital letters:

$F_r(r)$	probability distribution function (of log-normal distribution)
$F_x(x)$	probability distribution function (of normal distribution)
P_f	failure probability
R	resistance
E	action (load) effect
M	safety margin
V	coefficient of variation

Others

$\Phi(k)$	normalized function
θ	variables which account for the model uncertainties
θ_d	design values of the variables which account for model uncertainties

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Acronyms

AAEM	Age Adjusted Effective Modulus (for creep calculations)
AAR	Alkali Aggregate Reaction
ACI	American Concrete Institute
AFRP	Aramide Fibre Reinforced Plastic
ASR	Alkali Silica Reaction
ASTM	American Society for Testing and Materials
BCD	Birth Certificate Document
CCL	Condition Control Level
CCP	Condition Control Plan
CEB	Commission Euro-Internationale du Béton
CEN	European Commission for Normalization
CEM	Indication for cement type
CFRP	Carbon Fibre Reinforced Plastic
CMOD	Crack Mode Opening Displacement
CTE	Coefficient of Thermal Expansion
DIN	German institution for normalization
ECE	Electrochemical Chloride Extraction
EDC	Equivalent Durability Concept
EE	Embodied Energy
EIC	Environmental Impact Calculation
EN	European Norm
ETA	European Technical Approval
ETAG	European Technology Assessment Group
fib	fédération internationale du béton / International Concrete Federation (created from the merger between CEB and FIP)
FIP	International Federation for Prestressed Concrete
FRC	Fibre Reinforced Concrete
FRP	Fibre Reinforced Plastics
GFRP	Glass Fibre Reinforced Plastic
GHG	Green House Gas
GWP	Global Warming Potential
Hz	Herz
IABSE	International Association for Bridges and Shell Structures
ISO	International Organization for Standardization
JCSS	Joint Commission on Structural Safety
JSCE	Japanese Society of Civil Engineers
JSSC	Japanese Society of Steel Construction
LC	Indication for Lightweight Concrete strength class
LCC	Life Cycle Cost

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LCF	Life Cycle File
LCM	Life Cycle Management
LoA	Level of Approximation
LWAC	Light Weight Aggregate Concrete
MC	Model Code
MPa	Mega Pascal
PC	Prestressed Concrete
PL	Protection Level
PQP	Project Quality Plan
QM	Quality Management
RC	Reinforced Concrete
SIA	Social Impact Assessment, Swiss Union of Engineers and Architects
SFRC	Steel Fibre Reinforced Concrete
SLD	Service Life Design
RH	Relative Humidity
SETRA	French Road and Motorway Technical Studies Department
SCA	Service Criteria Agreement
SCC	Self Compacting (Consolidating) Concrete
SLS	Serviceability Limit State
RILEM	International Union of Laboratories and Experts in Construction Materials, Systems and Structures
UFC	Unified Facilities Criteria (Code for Military Structures)
UHPFRC	Ultra High Performance Fibre Reinforced Concrete
ULS	Ultimate Limit State
UTS	Ultimate Tensile Strength

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7 Design

7.1 Conceptual design

7.1.1 General

The conceptual design stage is the period during which identified needs are examined, requirements for potential solutions are defined, potential solutions are evaluated and a suitable structural concept for further design is developed.

The conceptual design stage is the most important phase of a project. Without an idea, without a proper solution to the problem under study there is no established safety concept, no adequately defined behaviour and essentially no solution to the defined problem, without which a successful construction project cannot be realized.

Many iterations of the design process are commonly required to refine the design concepts to accord with the functional requirements and associated financial/other constraints. The analytic tools applied at this stage to the investigation of the problem and evaluation of potential options may be relatively crude.

The basic approach to design relies on decomposition and integration. Since design problems are large and complex, they have to be decomposed into sub-problems that are small enough to solve. There are numerous alternative ways to decompose design problems, such as decomposition by functions of the facility, by spatial locations of its parts, or by links among various functions or parts. Solutions to sub-problems must be integrated into an overall solution. The integration and rationalisation process often creates conceptual conflicts which must be identified and resolved.

Various ideas for solving the problem under study, taking into account the owners program and the stakeholders' expectations, are produced during the conceptual design stage, with one that complies in an optimal manner with the specified requirements. These ideas, even though lacking in detail, must describe the solution from the points of view of functionality, environmental integration (physical, social and historical), structural adequacy, sustainability, construction, economy, etc. This phase should identify the more critical aspects which need to be more thoroughly developed in the following stages of the design process.

7.1.2 Methodology

Conceptual design is a creative act for which it is not easy to establish a methodology. Figure 7.1-1 illustrates a process which may provide some insight and be of assistance for carrying out this activity.

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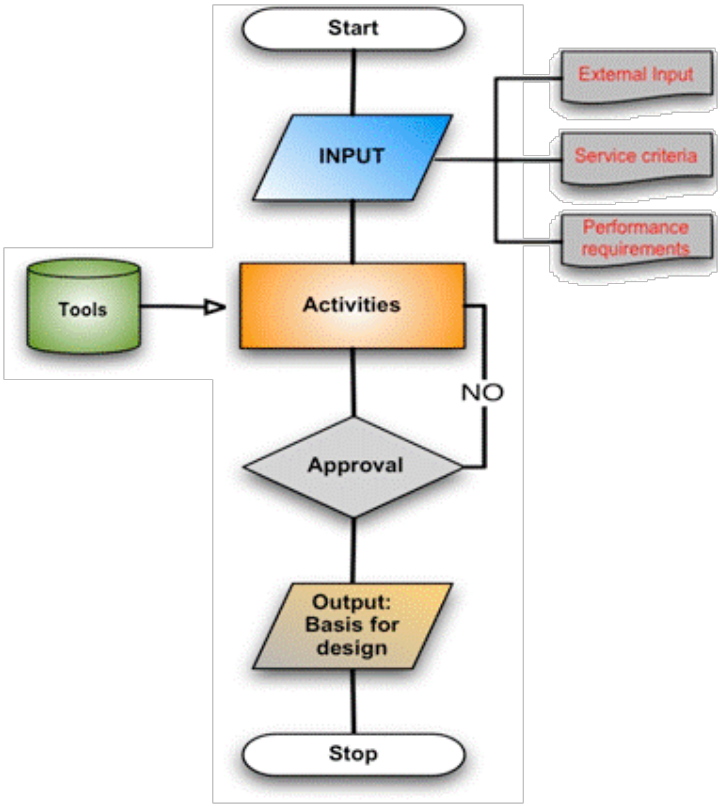


Figure 7.1-1: Methodological flowchart for conceptual design

7.1.2.1 Input

Initial information must be established with regard to:

- general criteria;
- basic external input data;
- service criteria;
- performance requirements.

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If the basic external input data is not available to the designer, a process will need to be established so that it can be obtained either from the owner, the architect, the authorities or some other source, or via an appropriate process instigated by the designer. Basic data shall be clearly specified in the Service Criteria Agreement, see subclause 3.5.3.5.

The service criteria shall be discussed and established with the owner or the architect; it shall be approved by all and shall be clearly specified in the Service Criteria Agreement, see subclause 3.5.3.5.

The performance requirements shall be established, proposed and explained by the designer, in conjunction with the owner, and shall be clearly specified in the Service Criteria Agreement, see subclause 3.5.3.5.

General criteria

- owner and stakeholder requirements and expectations;
- functionality;
- aesthetics;
- environmental integration (physical, social and historical);
- sustainability;
- structural adequacy;
- feasibility;
- economy;
- etc.

Basic external input data

- basic data applicable, including third party interactions (geotechnical data, metocean data, topographical and bathymetrical data, climatological data, environmental data (earthquake, hurricanes), material properties, accessibility and transport facilities, local construction rules, in the case of buildings architectural requirements, e.g. for form, mass/volume, surface texture and colour, etc.)

Service criteria

- general aims for the use of the construction works (efficiency, comfort, safety, etc.);
- operational and maintenance requirements (efficiency, economy, etc.);
- special requirements of the stakeholders (upgrading, replacement, etc.);
- objectives of protection and special risks;
- loadings and loading combinations;
- environmental conditions;
- codes and regulatory requirements.

Performance requirements

- performance criteria for serviceability and safety (including durability and robustness);
- service life constraints (temporary, replaceable, evolutive, long term);

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In general, activities performed during the stage of conceptual design of construction works are related to:

- constraint analysis and classification;
- environment analysis (including local politics and local traditions);
- general conception;
- choice of materials (considering economy and energy consumption for production and elimination);
- structural concept (structural logic, dimensions);
- integration and aesthetics (legibility, simplicity, proportions, equilibrium, shapes, detail philosophy);
- construction method (sequences);
- rough cost estimate;
- comparison of alternatives;
- successive presentation, explanation and discussions with the owner (architect);
- after acceptance by the owner – preparation of the basis for design (drawings, notes, reports).

Attributes and tools such as the following may be employed during the conceptual design stage:

- experience, plus insight from background, feedback, database sources;
- intuition, feeling, sensitivity for the circumstances;
- creativity, imagination, capacity of simultaneous analysis and integration of diverse criteria and constraints taking into account their relative weights;
- reliability constraints;
- performance requirements for sustainability.

7.1.2.2 Activities

The conceptual design process can be characterized by a series of interactive activities, described as follows:

- formulation, which refers to the definition or description of a design problem in broad terms, through the synthesis of ideas describing alternative concepts;
- analysis, which refines the problem definition or description by separating important from peripheral information and by pulling together the essential detail. Interpretation and prediction are usually required as part of the analysis;
- search, which involves gathering a set of potential solutions for performing the specified functions and satisfying the user requirements;
- decision, which means that each of the potential solutions is evaluated and compared to the alternatives until the best solution is obtained;
- specification, which is to describe the chosen solution in a form which is detailed enough for implementation;
- modification, which refers to the change in the solution or re-design if the solution is found to be insufficient or if new information is discovered in the process of design.

7.1.2.3 The role of expertise, insight and tools

The conceptual design process aims to find acceptable solutions for the defined requirements, constraints and the associated opportunities provided by the circumstances. The process is guided by the experience gathered in comparable construction works, along with insight and intuition obtained in other relevant circumstances. A variety of tools and aids may be used to assist the process, including those for visualisation of candidate schemes and alternative options, basic dimensioning of elements, preliminary evaluation of economic outcomes, etc.

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– quick pre-design methods,
– development of ideas, concepts and design details by sketching (ranging from rough freehand sketches to accurate drawings);
– visualization tools.

It is always interesting to involve experts from other relevant fields, depending on the type of structures (architects, urban planners, landscape artists, archaeologists, historians, etc.).

The extent and content of the basis of design shall be adapted to the importance of the construction works and the associated hazards and environmental risks, but it must always exist no matter how minor the project might be considered to be.

7.1.3 Structural Concept and Basis for Design

The Structural Concept derived from the conceptual design includes:

– the chosen structural system;
– information on the most important dimensions, construction material properties and construction details;
– comments on the envisaged methods of construction.

The Structural Concept derived from the conceptual design shall be described in the Basis of Design, including the bases and requirements for the subsequent design, execution, use and conservation.

The Basis for Design describes:

– the design working life;
– the service situations considered;
– the hazard scenarios considered;
– the requirements of structural safety, serviceability and durability together with the measures needed to guarantee them, including division of responsibilities, processes, controls and corrective mechanisms;
– the assumed ground conditions;
– the important assumptions in the structural and analytical models;
– the accepted risks;
– other conditions relevant to the design.

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In particular cases structural models may be based on experiments carried out in relation to the particular design or on a combination of testing and analytical calculations.

Typical D-regions are regions with static and/or geometric discontinuities, where plane sections do not remain plane after deformation, such as deep beams, beams/columns/slabs with geometrical discontinuities, bridge diaphragms, foundations and pile caps. Other examples of typical D-regions are the areas where structural components are connected (e.g. beam column, load introduction areas and supports).

Imposed deformations can result from differential settlements, temperature gradients or differences in humidity or from seismic actions.

With regard to the theory of plasticity both the upper bound and the lower bound theorems of plasticity can be applied.

The application of the lower theorem of plasticity implies that a safe bearing mode is found, if a statically admissible bearing system applies in which, under the actions defined, the admissible stresses are nowhere exceeded. Examples of such systems are strut and tie models and the strip method, used for the design of slabs. The solutions found can be more or less economic, but represent a lower bound for the bearing capacity.

7.2 Structural analysis and dimensioning

7.2.1 General

Structural analysis starts with a general determination of the actions in the structure, tracing the flow of forces and moments through the structure as a whole.

Structural analysis, both on a general and a more detailed level, comprises the determination of action effects such as internal stress fields, forces and moments, support reactions and deformations carried out on the basis of a structural model. To that aim, the structure can be subdivided into components, like beams, slabs, walls and shells and connecting areas, like B- and D- regions. In B-regions the forces and moments vary gradually. The assumption that the sections remain plane after deformation (hypothesis of Bernouilli) is valid. In D-regions the forces and moments vary discontinuously, so the hypothesis of Bernouilli is no longer valid.

Analyses shall be carried out using idealizations of both the geometry and the behaviour of the structure. Idealizations shall be appropriate to the case considered.

The effect of geometry and the properties of the structure and its behaviour at each stage of construction and service shall be considered in the design. Second order effects shall be taken into account where they are likely to affect the overall stability of a structure significantly and for the attainment of the ultimate limit state at critical sections.

The internal forces and moments in a structure follow from a system of loads or from imposed deformations or from a combination of both.

Internal forces, moments and deformations in statically indeterminate structures may be determined based on:

- theory of linear elasticity;
- theory of linear elasticity with limited redistribution;
- theory of plasticity;
- nonlinear methods.

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The application of the upper theorem of plasticity requires the adoption of a pattern of yield lines, generating a kinematic mechanism. The pattern that fails at the lowest load represents the bearing capacity. This method is particularly valuable for finding the bearing capacity of existing structures.

An example of a case where a detailed investigation is necessary in the SLS is crack-width control for leakage in a statically indeterminate structure. Examples of cases for the ULS are the control of rotation capacity and the determination of the magnification factor for verifying the stability of a structure.

Deviations in cross-sectional dimensions are normally taken into account in the material safety factors. Therefore these need not be included in structural analysis.

The effect of time dependant behaviour of concrete (creep and shrinkage) shall be accounted for according to the guidelines of subclause 7.2.4. Proper consideration shall be given to the effect of relaxation of steel.

Except for those due to the seismic action, the effect of imposed deformations may be neglected in verifying structural safety if an adequate deformation capacity is ensured for all parts of the structure.

If detailed investigations are necessary for the determination of forces and moments in the serviceability limit state or the ultimate limit state, an analysis can be carried out with adequately reduced stiffness of structural areas due to cracking.

7.2.2 Structural modelling

7.2.2.1 General

The static and geometrical boundary conditions as well as the transmission of support reactions shall be taken into account when idealizing and delimiting the system.

Soil structure interaction shall be considered appropriately.

7.2.2.2 Geometric imperfections

The unfavourable effects of possible deviations in the geometry of the structure and the position of the loads shall be taken into account in the analysis of members and structures.

Imperfections shall be taken into account for the verification of the ultimate limit state for persistent and accidental design situations. In the case of slender compression members, the second order effects and the influences of creep of concrete shall be taken into account (subclause 7.3.7).

Imperfections need not be considered for the verification of the serviceability limit state.

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In the case of bridge piers or highly stressed building columns, the inclination resulting from the base rotation can be of importance for the dimensioning of the bracing structural members (e.g. floor slabs, bracings of buildings, bridge bearings). The effect of the misalignment shall be estimated and if necessary taken into consideration in the calculations.

The definition of l and m in Eq. (7.2-1) and Eq. (7.2-2) depend on the effect considered, for which three principal cases can be considered:

- effect on the single element: l = real length of structural member, m = 1
- effect on stabilizing structure: l = height of building, m = number of vertical structural members that contribute to the horizontal force on the stabilizing structure
- effect on floors or roofs which transmit horizontal loads: l = storey height, m = number of vertical elements in the storeys that contribute to the total horizontal force on the floor considered

Unless specified otherwise in the basis of design, the unintended base rotation of vertical compression members amounts to

$$\frac{1}{200} \geq \alpha_i = \frac{0.01}{\sqrt{l}} \geq \frac{1}{300} \qquad (l \text{ in m}) \qquad (7.2\text{-}1)$$

where:

l denotes the height of the compression member or compression members standing on top of one another.

In buildings, the average misalignment α_{im} of a group of vertical compression members can be estimated with the equation

$$\alpha_{im} = \alpha_i \sqrt{0.5(1.0 + \frac{1.0}{m})} \qquad (7.2\text{-}2)$$

where:

m denotes the number of compression members which have to be included in determining the effect of the misalignment; see Figure 7.2-1.

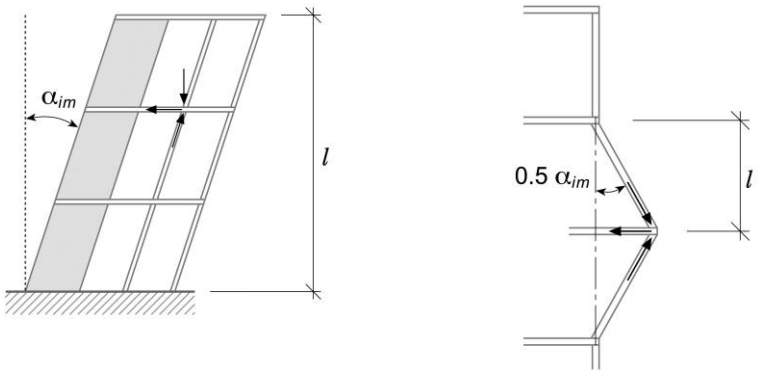


Figure 7.2-1: Geometrical imperfections

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7.2.2.3 Structural geometry

For the structural analysis, the structure shall be idealized using suitable models; examples are plane or space frames and B- and D-regions of structural components.

In the case of T-beams, the effective slab width depends on the web and the flange dimensions, the type of action, the span and the support conditions. The effective slab width may be estimated with the equation (Figure 7.2-2):

$$b_{eff} = \Sigma b_{eff,i} + b_w \leq b \tag{7.2-3}$$

where

$$b_{eff,i} = 0.2 b_i + 0.1 l_0 \leq 0.2 l_0 \tag{7.2-4}$$

The distance l_0 between the points of zero moment may be determined for usual cases according to Figure 7.2-3, based on the following assumptions:

- the cantilever length is smaller than half the adjacent span;
- the ratio between adjacent spans is between 2/3 and 1.5.

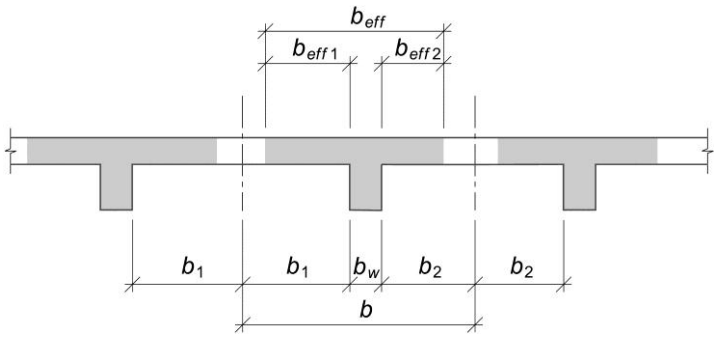


Figure 7.2-2: Effective slab width

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This approach implies that the "response relationship" is linear, and the assumption of reversible deformations is retained. The results are realistic only under the assumption that actions are low and members are uncracked.

For ULS verifications existing practice allows the use of linear elastic analysis without direct verification of sufficient ductility. This is based on the assumption that there is ductility enough to balance the lack of compatibility. The method is normally used with the gross-section of concrete members; therefore it requires definition of geometry of the structure, but not necessarily of the reinforcement.

Cracked cross-sections may, however, be used if in the limit state under consideration a fully developed crack pattern can be expected. The results of a linear analysis are also used in the verification for the serviceability limit state.

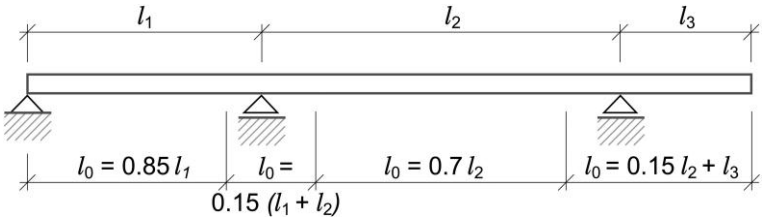


Figure 7.2-3: Relevant distances l_0 for the determination of the effective slab width

7.2.2.4 Calculation methods

7.2.2.4.1 Analysis based on linear elasticity

Analysis of elements based on the theory of linear elasticity may be used for both the serviceability and the ultimate limit states.

For the determination of the action effects, linear elastic analysis may be carried out assuming:

- uncracked cross-sections,
- linear stress-strain relationships,
- the mean value of the modulus of elasticity.

For determining the effect of imposed deformations at the ultimate limit state a reduced stiffness corresponding to cracked sections may be assumed. For the serviceability limit state a gradual evolution of cracking should be considered.

7.2.2.4.2 Analysis according to linear elasticity with limited redistribution

Linear analysis with limited redistribution may be applied to the analysis of structural members for the verification at the ULS.

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If redistribution of moments is applied in determining the reinforcement this may have an influence on deflection and crack width.

The influence of any redistribution of moments on other aspects of design shall be considered.

The moments at the ULS calculated using a linear elastic analysis may be redistributed, provided that the resulting distribution of moments remains in equilibrium with the applied loads.

Redistribution of bending moments without explicit check on the rotation capacity is allowed for continuous beams or slabs which are predominantly subjected to flexure and have a ratio of the lengths of adjacent spans in the range of 0.5 to 2. In this case the following relations should apply:

$$\delta \geq k_1 + k_2 x_u / d \qquad \text{for } f_{ck} \leq 50 \text{ MPa} \qquad (7.2\text{-}5)$$

$$\delta \geq k_3 + k_4 x_u / d \qquad \text{for } f_{ck} > 50 \text{ MPa} \qquad (7.2\text{-}6)$$

and

$\delta \geq k_5$ where Class B, Class C or Class D reinforcement is used, see subclause 5.2.5.4;

$\delta \geq k_6$ where Class A reinforcement is used, see subclause 5.2.5.4;

where:

δ is the ratio of the redistributed moment to the elastic bending moment

x_u is the depth of the neutral axis at the ULS after redistribution;

d is the effective depth of the section;

$k_1 = 0.44$;

$k_2 = 1.25(0.6+0.0014/\varepsilon_{cu2})$;

$k_3 = 0.54$;

$k_4 = 1.25(0.6+0.0014/\varepsilon_{cu2})$;

$k_5 = 0.7$;

$k_6 = 0.8$;

ε_{cu2} is ultimate strain according to subclause 7.2.3.1.1.

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Redistribution should not be carried out in circumstances where the rotation capacity cannot be defined with confidence (e.g. in corners of frames with opening moments).

For the design of columns it should be checked whether the moment before redistribution is governing for the design.

7.2.2.4.3 Theory of plasticity

General

Plastic analysis should be based either on the lower bound (static) or the upper bound (kinematic) theorem.

When applying methods based on the theory of plasticity it should be ensured that the deformation capacity of critical areas is sufficient for the envisaged mechanism to be developed.

The effects of previous applications of loading may generally be ignored and a monotonic increase of the intensity of the actions may be assumed.

Plastic analysis of beams, frames and slabs with the kinematic theorem

Plastic analysis without any check of the rotation capacity may be used for the ultimate limit state if all the following conditions are met:

- the area of tensile reinforcement is limited to such a value that at any section

  $x_u/d \leq 0.25$ for concrete strength classes $\leq$ C50/60;

  $x_u/d \leq 0.15$ for concrete strength classes $\geq$ C55/67;
- reinforcing steel is either Class B or C;
- the ratio of the moments at intermediate supports to the moments in the span is between 0.5 and 2.

Columns should be checked for the maximum plastic moments which can be transmitted by connecting members. For connections to flat slabs this moment should be included in the punching shear calculation.

When plastic analysis of slabs is carried out, account should be taken of any non-uniform reinforcement, corner tie down forces and torsion at free edges.

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For the application of the simplified procedure, it is supposed that the plastic hinge considered occurs as the first of those resulting in a kinematic system.

If the demand of rotation θ_s is calculated by integrating the curvatures between the plastic hinges, in general the application of a trilinear moment-curvature relation is appropriate. The moments caused by prestressing should be considered as a part of the effect of the load on the structure.

Rotation capacity

If continuous beams or continuous one way slabs do not meet the conditions for which no check of rotation capacity is required, as formulated previously, a simplified procedure can be used. This procedure is based on a control of the rotation capacity. The rotation capacity is determined over a length of approximately 1.2 times the depth of the section. It is assumed that these zones undergo a plastic deformation (formation of yield hinges) under the relevant combination of action. The verification of the plastic rotation in the ultimate limit state is considered to be fulfilled, if it is shown that under the relevant combination of actions the demand in rotation of the plastic hinge, calculated θ_s is smaller than or equal to the allowable plastic rotation $\theta_{pl,d}$ (Figure 7.2-4).

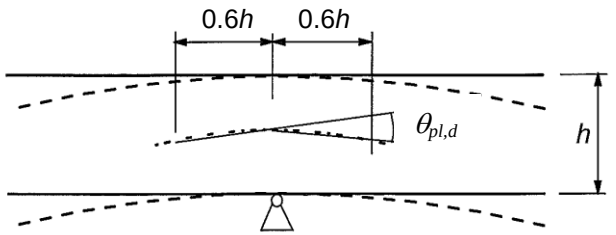


Figure 7.2-4: Rotation capacity $\theta_{pl,d}$ of reinforced concrete sections for continuous beams and continuous one way spanning slabs

In regions of yield hinges, x_u/d should not exceed the value 0.45 for concrete strength classes less or equal to C50/60, and 0.35 for classes higher than or equal to C55/67.

The rotation $\theta_{pl,d}$ should be determined on the basis of design values for action and materials and on the basis of mean values for prestressing at the relevant time.

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In Figure 7.2-5 the x_u-values are calculated considering design values of material properties, bilinear elasto-plastic stress-strain response of steel with a strain limit of $\varepsilon_s = \varepsilon_{ud}$ and the behaviour of concrete under compression simplified with a parabola-rectangle stress-strain diagram with nominal strain limits according to Figure 7.2-9.

According to the simplified procedure, the allowable plastic rotation may be determined by multiplying the basic value of allowable rotation, $\theta_{pl,d}$, by a correction factor k_λ that depends on the shear slenderness. The values for the allowable rotation for the steel classes B and C (the use of steel class A is not recommended for plastic analysis) and concrete strength classes below or equal to C55/67 and C100/115 are given in Figure 7.2-5.

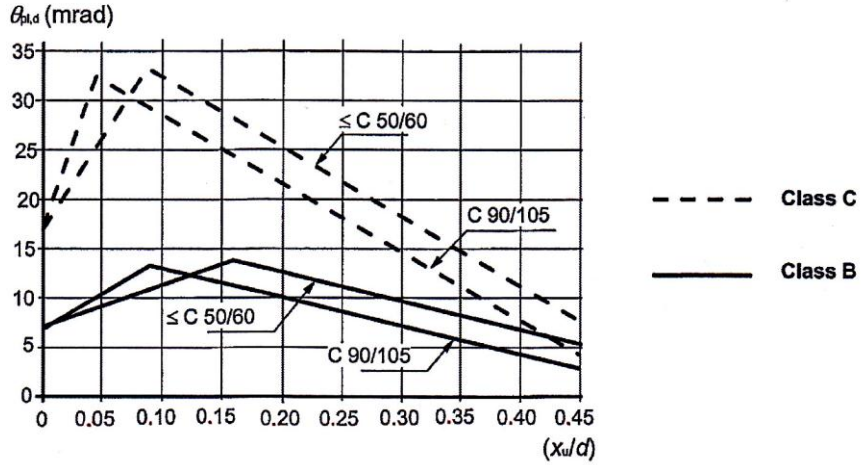


Figure 7.2-5: Basic values for allowable rotation $\theta_{pl,d}$ of reinforced concrete sections for Class B and C reinforcement. The values apply for a shear slenderness $\lambda = 3.0$

The values for the strength classes C55/67 to C100/115 may be interpolated accordingly.

The values in Figure 7.2-5 apply for a shear slenderness λ = 3.0. For different values of the shear slenderness $\theta_{pl,d}$ should be multiplied by:

$$k_\lambda = \sqrt{\lambda / 3} \qquad (7.2\text{-}7)$$

where:

λ is the ratio of the distance between the point of zero and maximum moment after redistribution and the effective depth d.

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Orienting the struts to the compression trajectories in the uncracked state, assuming linear elastic behaviour, aims at minimizing redistribution of forces after cracking, which could result in violation of service requirements (crack width and deformation), or even premature failure. If the redistribution of forces is minimized, for SLS and ULS the same strut and tie model is appropriate.

Analysis with strut and tie models

Analysis with a strut and tie model is a method according to the lower bound theorem of plasticity (static method). According to this method a state of equilibrium between external and internal forces has to be found, which fulfils the static boundary conditions and nowhere violates the yield conditions.

Strut and tie models may be used for design of the reinforcement in continuity regions (B-regions) in the ULS (cracked state of beams and slabs) and for the design and detailing of discontinuity regions (D-regions) in the ULS. In general D-regions extend up to a distance h from the discontinuity, where h is the largest cross-sectional dimension.

Verifications in the SLS may also be carried out using strut and tie models, e.g. verification of steel stresses and crack width control, if approximate compatibility for strut and tie models is ensured (in particular the position and direction of important struts should be oriented according to the compression trajectories in the linear elastic stage.

Strut and tie models consist of struts representing compressive stress fields, of ties representing the reinforcement, and of connecting nodes. Both for ULS and SLS calculations the struts should be oriented to the compressive trajectories in the uncracked stage based on linear elastic behaviour. The forces in the elements of a strut and tie model should be determined by maintaining the equilibrium with the applied loads in the ultimate limit state. The elements of strut and tie models should be dimensioned according to the rules given in subclause 7.3.6.

The ties in a strut and tie model should coincide in position and direction with the corresponding reinforcement.

7.2.2.4.4 Nonlinear analysis

Nonlinear methods of analysis may be used for both ULS and SLS, provided that equilibrium and compatibility are satisfied and adequate nonlinear behaviour for materials is assumed. The analysis may be first or second order.

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Nonlinear analysis should be carried out on the basis of the principles given in clause 7.11. For predicting the mean behaviour mean values of the material characteristics should be used as defined in clause 5.1.

7.2.3 Dimensioning values

7.2.3.1 Concrete

7.2.3.1.1 Strength and strain characteristics

The compressive strength of concrete is denoted by concrete strength classes which are related to the characteristic (5%) cylinder strength f_{ck} or the cube strength $f_{ck,cube}$ at an age of 28 days.

The characteristic strengths for f_{ck} and the corresponding mechanical characteristics necessary for design are given in Table 7.2-1.

The mechanical characteristics given in Table 7.2-1 represent average values on the basis of the concrete compressive strength for a wide range of compositions.

General rules for design by testing are given in clause 7.13.

Concrete can be tailored for particular applications. In that case the relation between the compressive strength of the concrete and relevant mechanical properties may deviate from the relations found in Table 7.2-1 In such case different relations can be obtained by testing.

In certain situations (e.g. prestressing) it may be appropriate to assess the compressive strength for concrete before or after 28 days. General relations between the compressive and tensile strengths of the concrete at other ages than 28 days are given in clause 5.1.

The tensile strength of concrete can be determined directly by a uniaxial tensile test or by a splitting tensile test. Where the tensile strength is determined as the splitting tensile strength $f_{ct,sp}$ an approximate value of the axial tensile strength f_{ct} is found from:

$$f_{ct} = 1.0\, f_{ct,sp} \qquad (7.2\text{-}8)$$

Axial tensile tests are very sensitive to the way the test is carried out. For further information on axial tensile testing of concrete, reference is made to J.G.M. van Mier, M.R. van Vliet “Uniaxial tension test for determination of fracture parameters of concrete: state of the art”, Engineering Fracture Mechanics, Vol. 69, Issue 2, Jan. 2002, pp.235-247.

Further reference is made to subclause 5.1.5.

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Table 7.2-1 Strength classes for concrete

Concrete grade	C12	C16	C20	C25	C30	C35	C40	C45	C50	C55	C60	C70	C80	C90	C100
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	100
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	115
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	108
f_{ctm} (MPa)	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1	4.2	4.4	4.6	4.8	5.0	5.2
$f_{ctk,0.05}$ (MPa)	1.1	1.3	1.5	1.8	2.0	2.2	2.5	2.7	2.9	3.0	3.1	3.2	3.4	3.5	3.7
$f_{ctk,0.95}$ (MPa)	2.0	2.5	2.9	3.3	3.8	4.2	4.6	4.9	5.3	5.5	5.7	6.0	6.3	6.6	6,8
E_{cm} (GPa)	27.1	28.8	30.3	32.0	33.6	35.0	36.3	37.5	38.6	39.7	40.7	42.6	44.4	46.0	47.5
ε_{c1} (‰)	1.8	1.9	2.1	2.2	2.3	2.3	2.4	2.4	2.5	2.5	2.6	2.7	2.8	2.9	3.0
ε_{cu1} (‰)	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.4	3.4	3.3	3.2	3.1	3.0	3.0
ε_{c2} (‰)	2.0									2.2	2.3	2.4	2. 5	2.6	2,7
ε_{cu2} (‰)	3.5									3.1	2.9	2.7	2.6	2.6	2,7
n	2.0									1.75	1.6	1.45	1.4	1.4	1,3
ε_{c3} (‰)	1.75									1.8	1.9	2.0	2.2	2.3	2.4
ε_{cu3} (‰)	3.5									3.1	2.9	2.7	2.6	2.6	2.4

The values in Table 7.2-1 are based on the following analytical relations (in all formulas the strengths f_{cm}, f_{ck}, f_{ctm} and f_{ctk} are in MPa and the strains ε in ‰).

$f_{cm} = f_{ck} + 8$ with f_{cm} and with f_{ck} in MPa

$f_{ctm} = 0.30\, f_{ck}^{\,2/3}$ for concrete classes ≤ C50

$f_{ctm} = 2.12 \ln(1 + f_{cm}/10)$ for concrete classes > C50

$f_{ctk;0.05} = 0.7\, f_{ctm}$

$f_{ctk;0.95} = 1.3\, f_{ctm}$

$E_{cm} = 21.5(f_{cm}/10)^{1/3}$ with f_{cm} in MPa

ε_{c1} (‰) see Table 5.1-8

ε_{cu1} (‰) = $\varepsilon_{c,lim}$ see Table 5.1-8

Table 7.2-1 shows a survey of design values for concrete strength classes up to C100. Although in subchapter 5.1 mechanical properties of concrete are given until a strength class C120, in this table the strength classses are limited to C100. The reason is that not in all areas of application sufficient data are available yet for all aspects of behaviour.

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ε_{c2} (‰) = 2.0 + 0.085 (f_{ck} - 50) $^{0.53}$ for > C50
ε_{cu2} (‰) = 2.6 + 35 [(90 – f_{ck}) / 100]^4 for > C50
n = 1.4 + 23.4 [(90 – f_{ck}) / 100] ^{4} for > C50
ε_{c3} (‰) = 1.75 + 0.55 [(f_{ck} - 50) / 40] for > C50
$\varepsilon_{cu\,3}$ (‰) = 2.6 + 35 [(90 – f_{ck}) / 100]^4 for > C50

Further reference is made to subclause 5.1.4 (compressive strength), subclause 5.1.5.1 (tensile strength), subclause 5.1.7.2 (modulus of elasticity), subclause 5.1.8.1 (stress-strain relations in compression for short term loading).

The constant 0.06 in Eq. (7.2-9) is appropriate for normal strength concrete. With increasing brittleness of the concrete the coefficient decreases. This means that for high strength concrete and lightweight concrete lower values for the constant in Eq. (7.2-9) apply, see subclause 5.1.5.1, comment to Eq. (5.1-8a, b).

The flexural tensile strength can be formulated as a function of the axial tensile strength. A general relation is:

$$f_{ctm,\,fl} = f_{ctm}\,\frac{1+0.06h_b^{\,0.7}}{0.06h_b^{\,0.7}} \qquad (7.2\text{-}9)$$

where:

f_{ctm} is the mean axial tensile strength [MPa];

$f_{ctm,fl}$ is the mean flexural strength [MPa];

h_b is the overall depth of beam [mm].

This relation applies as well to the characteristic values of the concrete strength.

7.2.3.1.2 Elastic deformation

The elastic deformations of concrete largely depend on its composition (especially the aggregates). The values given in Table 7.2-1 should be regarded as indicative for general applications. The values given in Table 7.2-1 are approximate values for the E-modulus E_{cm}, being the secant value between σ_c=0 and 0.4f_{cm} for concrete with quartzite aggregate, subjected to short term loading. More detailed information for concrete with other aggregates is given in clause 5.1. The elastic deformations should be specially assessed if the structure is likely to be sensitive to deviations from

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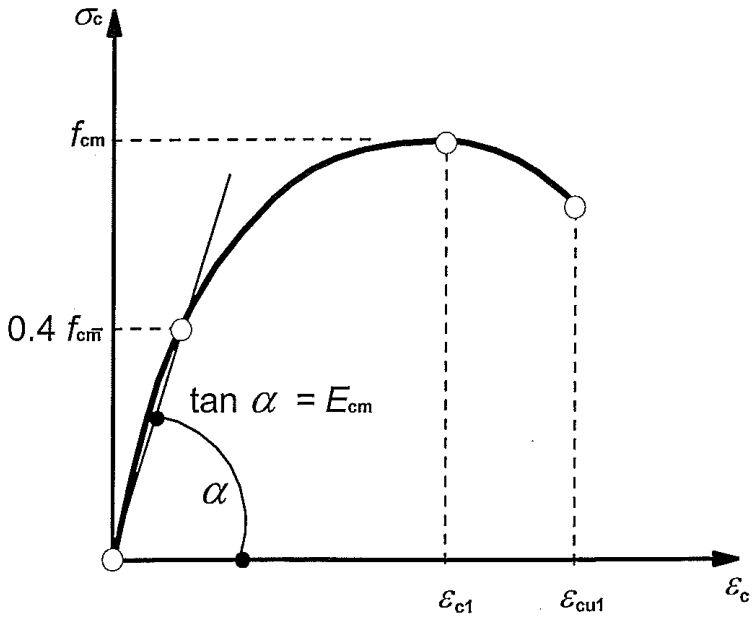


Figure 7.2-6: Schematic representation of the stress-strain relation for structural concrete

the indicative values. More detailed information on the E-modulus is given in clause 5.1.

The Poisson's ratio may be taken equal to 0.2 for uncracked concrete and 0 for cracked concrete.

Unless more accurate information is available, the linear coefficient of thermal expansion may be taken equal to $10 \cdot 10^{-6}$ K^{-1}

7.2.3.1.3 Stress-strain relation for non-linear structural analysis

The relation between σ_c and ε_c shown in Figure 7.2-6 (compressive stress and shortening strain shown as absolute values) for short term uniaxial loading is described by the expression:

$$\frac{\sigma_c}{f_{cm}} = \frac{k\eta - \eta^2}{1 + (k-2)\eta} \qquad (7.2\text{-}10)$$

where:

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

ε_{c1} is the strain at peak stress according to Table 7.2-1;

$k = 1.05 E_{cm} \cdot |\varepsilon_{c1}| / f_{cm}$ (f_{cm} according to Table 7.2-1).

Eq. (7.2-10) is valid for $0 < |\varepsilon_c| < |\varepsilon_{cu1}|$ where ε_{cu1} is the nominal ultimate strain.

Other idealized stress-strain relations may be applied, if they adequately represent the behaviour of the concrete considered.

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Generally the first variable loads on a structure are applied months after the determination of the 28-days strength. Since then the strength of the concrete has increased by continued hydration of the cement. This increase approximately balances the unfavourable effect of sustained loading, so that $\alpha_{cc} = \alpha_{ct} = 1.0$ is appropriate. For concrete strength determined at an age greater than 28 days, the effect of hydration may not be able any more to compensate the effect of sustained loading , so that $\alpha_{cc} = \alpha_{ct} = 0.85$ is more suitable, unless tests show otherwise.

For concrete strength classes > C50/60 the stress strain relation is characterized by a reduced length of the horizontal plateau (Figure 7.2-7 and Figure 7.2-8).

7.2.3.1.4 Design compressive and tensile strengths

The value of the design compressive strength is defined as:

$$f_{cd} = \alpha_{cc} f_{ck} / \gamma_c \qquad (7.2\text{-}11)$$

where:

γ_c is the partial safety factor for concrete, being 1.5 for transient and persistent situation and 1.2 for incidental situations;

α_{cc} is a coefficient taking account of long term effects on the compressive strength and of unfavourable effects from the way the load is applied. For normal design situations it may be assumed that the increase of the compressive strength after 28 days compensates the effect of sustained loading, so that $\alpha_{cc} = 1.0$ for new structures.

The value of the tensile strength f_{ctd} is defined as:

$$f_{ctd} = \alpha_{ct} f_{ctk} / \gamma_c \qquad (7.2\text{-}12)$$

For the same reason as mentioned for α_{cc} , for new structures $\alpha_{ct} = 1.0$

7.2.3.1.5 Stress strain relations for the design of cross-sections

For the design of cross-sections a choice can be made between two types of stress-strain relations. A parabola-rectangle relation (Figure 7.2-7) is defined according to:

$$\sigma_c = f_{cd}[1-(1-\frac{\varepsilon_c}{\varepsilon_{c2}})^n] \qquad \text{for } 0 \le \varepsilon_c \le \varepsilon_{c2} \qquad (7.2\text{-}13)$$

$$\sigma_c = f_{cd} \qquad \text{for } \varepsilon_{c2} \le \varepsilon_c \le \varepsilon_{cu2} \qquad (7.2\text{-}14)$$

where:

n is the exponent acc. to Table 7.2-1;

ε_{c2} is the strain reaching the maximum strength according to Table 7.2-1;

ε_{cu2} is the ultimate strain according to Table 7.2-1.

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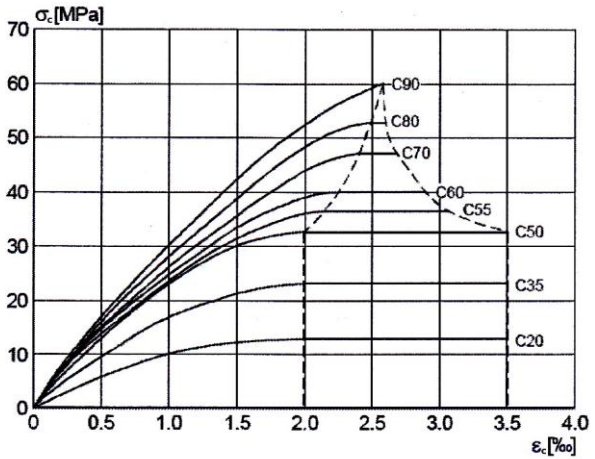


Figure 7.2-7: Design stress strain relations for various concrete strength classes (parabola-rectangle) for $\gamma_c = 1.5$

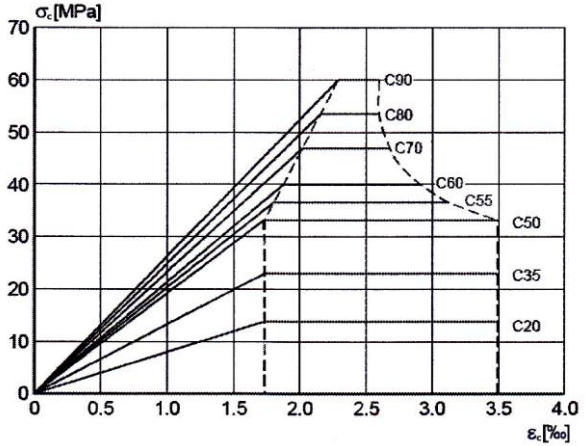


Figure 7.2-8: Design stress strain relation for various concrete strength classes (bilinear relation) for $\gamma_c = 1.5$

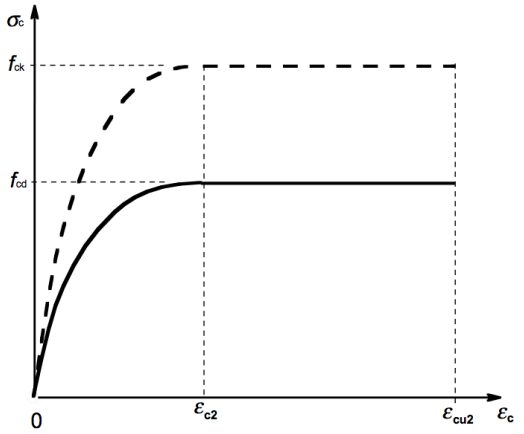


Figure 7.2-9: Parabola rectangle diagram for concrete in compression

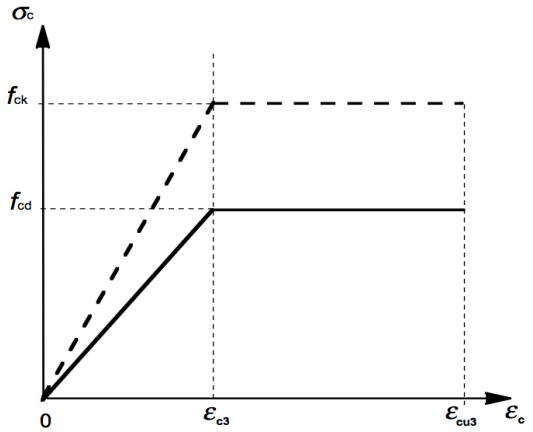


Figure 7.2-10: Bilinear stress-strain relation for concrete in compression

As an alternative a bilinear relation can be chosen, based on the values ε_{c3} and ε_{cu3} (Figure 7.2-10).

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The values for η and λ have been derived in such a way that they give the same results as the parabola rectangle stress distribution, see Figure 7.2-11.

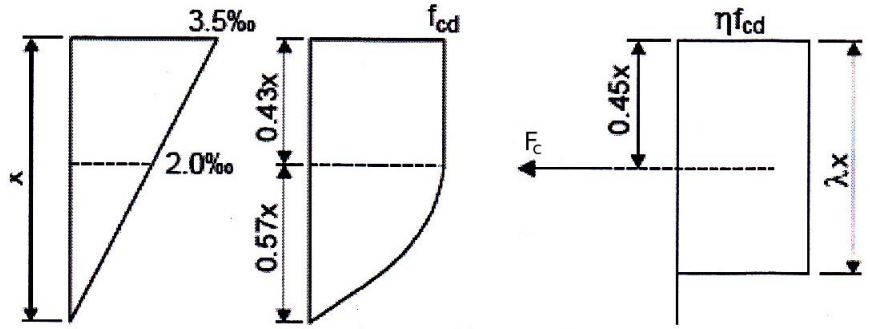


Figure 7.2-11: Derivation of rectangular stress block from parabolic rectangle stress distribution for concrete strength class ≤ C50/60

The equations for the rectangular stress block have been derived for the basic case that the neutral axis is inside the cross-section. For concrete strength classes ≤ C50/60 they are also correct if the neutral axis is outside the cross-section. However, for higher concrete strength classes the results underestimate the strength if the cross-section is completely in compression.

By appropriate confining of concrete the axial strength and the ductility are increased. If these properties are to be exploited in terms of calculations, it shall be checked whether the ultimate resistance is still sufficient after spalling of the cover concrete, and it shall be ensured that premature buckling of the longitudinal reinforcement is prevented by means of closely spaced stirrups.

Finally a rectangular stress distribution can be used, as shown in Figure 7.2-12.

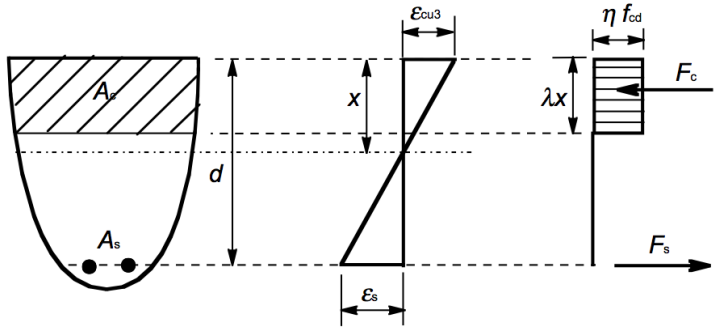


Figure 7.2-12: Rectangular stress distribution

The factor λ, defining the height of the compression zone and the factor η, defining the effective strength, follow from:

$\lambda = 0.8$ for $f_{ck} \leq 50$ MPa (7.2-15)

$\lambda = 0.8 - (f_{ck} - 50)/400$ for $50 < f_{ck} \leq 100$ MPa (7.2-16)

and

$\eta = 1.0$ for $f_{ck} \leq 50$ MPa (7.2-17)

$\eta = 1.0 - (f_{ck}\text{-}50)/200$ for $50 < f_{ck} \leq 100$ MPa (7.2-18)

If the width of the compression zone decreases in the direction of the extreme compression fibre, the value ηf_{cd} should be reduced by 10%.

7.2.3.1.6 Confined concrete

Confinement of concrete results in a modification of the effective stress-strain relationship: higher strength and higher critical strains are achieved. The other basic material characteristics may be considered as unaffected by design.

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The confining pressure σ_2 (with positive sign) can be calculated with the expressions:

$$\sigma_2 = \omega_c f_{cd}(1-\frac{s_c}{d_c}) \quad \text{(circular cross-section confined by spiral reinforcement)}$$

$$\sigma_2 = \omega_c f_{cd}(1-\frac{s_c}{d_c})^2 \quad \text{(circular cross-section confined by circular hoops)}$$

where $\omega_c = \dfrac{A_{sc} f_{yd}}{s_c d_c f_{cd}}$

$$\sigma_2 = \omega_c f_{cd}(1-\frac{s_c}{a_c})(1-\frac{s_c}{b_c})(1-\frac{\Sigma b_i^2/6}{a_c b_c}) \quad \text{(rectangular cross-sections)}$$

where $\omega_c = \min\{\omega_y = \dfrac{A_{sy} f_{yd}}{a_c s_c f}, \omega_z = \dfrac{A_{sz} f_{yd}}{b_c s_c f_{cd}}\}$

and b_i is the centre line spacing along the section parameter of longitudinal bars (indexed by i) engaged by a stirrup corner or a cross-tie.

In the absence of more precise data, the stress-strain relation shown in Eqs. (7.2-19)-(7.2-23) may be used, with increased characteristic strength and strains according to:

$$\frac{f_{ck,c}}{f_{ck}} = 1 + 3.5(\frac{\sigma_2}{f_{ck}})^{\frac{3}{4}} \tag{7.2-19}$$

$$\varepsilon_{c2,c} = \varepsilon_{c2}\left[1+5\left(\frac{f_{ck,c}}{f_{ck}}-1\right)\right] \tag{7.2-20}$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.2\sigma_2 / f_{ck} \tag{7.2-21}$$

where σ_2 (=σ_3) is the effective lateral compressive stress at the ULS due to confinement and ε_{c2} and ε_{cu2} follow from Table 7.2-1.

Confinement can be generated by adequately closed links or cross-ties, which reach the plastic condition due to lateral extension of the concrete.

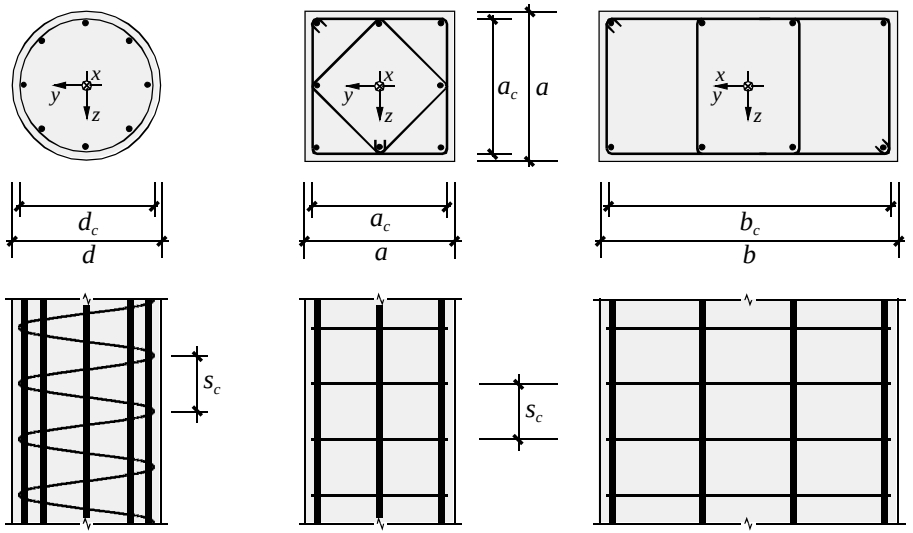


Figure 7.2-13: Compression members with confining reinforcement

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The dispersion of the concentrated forces causes biaxial or triaxial compression immediately under the load, whereas it produces transverse tension further away. Reinforcement should be provided for those tensile forces.

The increased stress according to Eq. (7.2-22) can only be combined with the increased stress due to confining action (7.2.3.1.6) if confirmed by appropriate test results.

For very large confining stresses or very small loading area failure occurs due to wedging action under the loaded area, at a stress which can be significantly higher than $3f_{cd.}$

If no further data are available the average bearing capacity can be calculated with the expression:

$$\frac{f_{cc}^{*}}{f_{cc}} = 12.5\sqrt{(40 / f_{cc}} \qquad (7.2\text{-}23)$$

(f_{cc} in MPa)

However, if limited penetration is considered, f_{cc}^{*} values not higher than $4 f_{cc}$ should be taken.

In Fig. 7.2-14 the following limits to h apply:

$h \geq a_2 - a_1$

$h \geq b_2 - b_1$

7.2.3.1.7 Partially loaded areas

For a uniform distribution of load on an area A_{c0} (Figure 7.2-14) the concentrated resistance force may be determined as follows:

$$F_{Rdu} = A_{c0} f_{cd} \sqrt{A_{c1} / A_{c0}} \leq 3.0 f_{cd} A_{c0} \qquad (7.2\text{-}22)$$

where:

A_{c0} is the loaded area;

A_{c1} is the maximum design distribution area with a similar shape to $A_{c0.}$

The design distribution area A_{c1} required for the resistance force F_{Rdu} should correspond to the following conditions:

- the height for the load distribution in the load direction should correspond to the conditions given in Figure 7.2-14;
- the centre of the design distribution area A_{c1} should be on the line of action passing through the centre of the load area A_{c0};
- if there is more than one compression force acting on the concrete cross-section, the designed distribution areas should not overlap.

The value F_{Rdu} should be reduced if the load is not uniformly distributed on the area A_{c0} or if high shear forces exist.

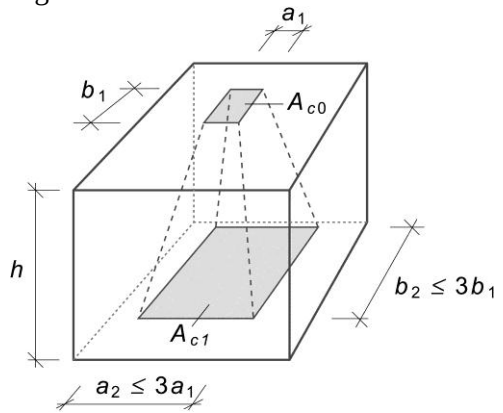


Figure 7.2-14: Load distribution for partially loaded areas

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7.2.3.2 Reinforcing steel

The design tensile strength of reinforcing steel f_{yd} follows from:

$$f_{yd} = f_{yk} / \gamma_s \qquad (7.2\text{-}24)$$

where:

f_{yk} is the characteristic yield stress of the steel;

γ_s = 1.15 for persistent and transient situations and 1.0 for accidental situations.

The behaviour of reinforcing steel in tension and compression is idealized in accordance with the stress-strain relations given in Figure 7.2-15.

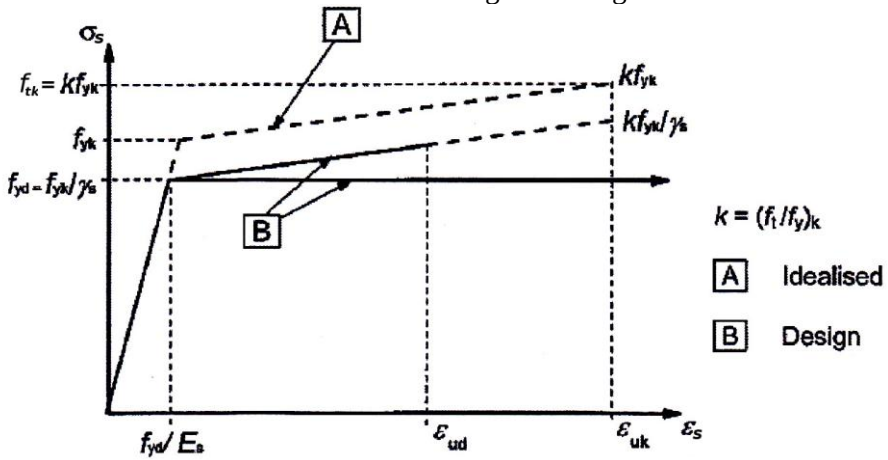


Figure 7.2-15: Idealized and design stress-strain relations for reinforcing steel in tension and compression

For situations in which the plastic structural deformations are of importance, it is suitable to assume a linear strain-hardening behaviour of the steel. In this case the maximum stress $k\, f_{yd} / \gamma_s$ is reached at a strain of $\varepsilon_{ud} = 0.9\varepsilon_{uk}$. The value k follows from $k = f_t / f_y$. For the values of the ultimate strength f_t and the yield strength f_y reference is made to clause 5.2.

In general the structural analysis and dimensioning may be based on the mean value of the modulus of elasticity E_s given in clause 5.2.

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7.2.3.3 Prestressing steel

The dimensioning values of the yield strength of the prestressing steel are determined on the basis of clause 5.3.

The behaviour of prestressing steel under tensile or compressive stresses is idealized in accordance with the stress-strain diagram given in Figure 7.2-16.

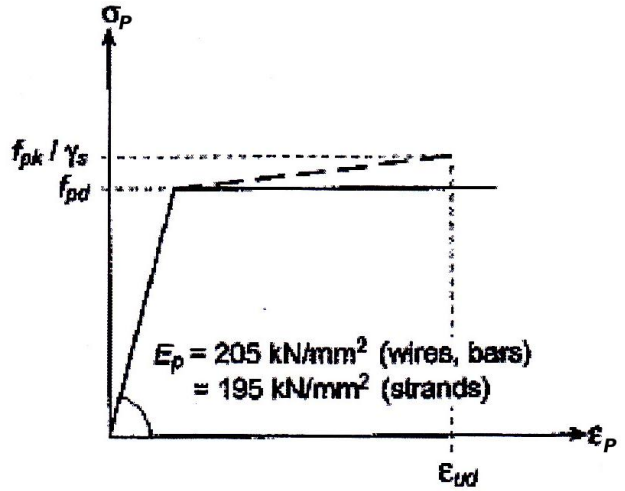


Figure 7.2-16: Idealized stress strain diagram for prestressing steel

The dimensioning is based on the nominal cross-sectional areas of the prestressing steel.

In general a perfectly plastic behaviour may be assumed.

For situations in which the plastic structural deformations are of importance, it is practical to assume a linear strain hardening behaviour of the prestressing steel. The ultimate strain shall be limited to $\varepsilon_{ud} = \varepsilon_{uk}$ as specified in the relevant product standard.

In general the structural analysis and dimensioning may be based on the mean value of the modulus of elasticity E_p according to clause 5.3.

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Shrinkage strains influence the state of deformation and induce stresses when they are restrained; they also cause stress redistributions in inhomogeneous and composite structures and sections and stress losses in prestressed structures.

The consequences of creep can be either beneficial or detrimental. On the one hand, creep exercises a beneficial effect by relieving undesirable stresses due to unintentional imposed strains like shrinkage, extreme initial temperatures, settlement of supports and yielding of restraints, etc. On the other hand, the long-term reliability of structures may be adversely affected, as creep:

- increases by an important factor their initial deformations;
- reduces the beneficial effects of stresses artificially imposed to improve the performance of the structure with regard to serviceability, either causing prestress losses in structures prestressed with tendons or strands, or significantly jeopardizing stress corrections enforced by jacking;
- activates the delayed restraints in case of changes in the structural system after the application of sustained loads, inducing in some cases significant redistributions of internal actions and stresses that may lead to unfavourable increases of their values in some regions of the structures;
- reintroduces, for the same reason, a significant part of the internal actions and stresses due to self weight that were provisionally eliminated in statically indeterminate structures by temporary reductions, during the construction stages, of their degree of restraint (e.g. bending moments in fixed end arches due to axial shortening, provisionally eliminated through the adoption of temporary hinges).

In non-homogeneous structures, creep induced stress redistributions transfer stresses from the parts of the structure creeping more to the parts creeping less, or from concrete to steel elements. Non-homogeneities may be

7.2.4 Analysis of structural effects of time-dependent behaviour of concrete

7.2.4.1 General

The inelastic strains due to creep and shrinkage of concrete may cause non-negligible variations of deformations and/or of internal actions and stresses in structures and structural elements.

The overall dimensions and the slenderness of structures and structural elements, in particular if combined with the adoption of thin sections, magnify their sensitivity to the time-dependent behaviour of concrete. In fact, while the initial deformations are large, the creep amplification factor and the shrinkage strains are augmented, as a consequence of the drying in thin elements.

The time sequence of casting, loading and application of restraints, as well as the presence of important non-homogeneities exert a significant influence on the time-dependent response.

Creep and shrinkage affect primarily the long-term serviceability and durability of structures. In particular, if the effects of time-dependent strains are mainly in the sense of an increment of the deformations, the limit-state of deformations should be checked. If the state of stress is mainly influenced by time-dependent effects, the limit-states of stresses and of cracking should be checked.

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due to differences in casting and loading ages, mixture proportions and components, size and shapes of structural elements and cross-section components, environmental conditions, etc., in the various concrete parts of the structure, and to the association of concrete and steel elements.

For example, in high-rise buildings, the combined effects of creep and shrinkage, of their non-uniform development and of sequential construction, besides generating several serviceability concerns regarding both structural and non-structural components, can cause also reductions of the safety margins with respect to ultimate limit state. This second concern can depend on the possible increase with time of action effects on some structural elements, and may become relevant especially when a limited ductility is available because of high axial loads, as in vertical elements, and of the use of high-strength concrete, or when buckling of slender steel elements must be feared. Influence of non-symmetrical time-dependent vertical shortenings on gravity induced sidesway and its effects on structural reliability must also be investigated.

When checking the stability of compressed members the long-term deformations of concrete may approximately be taken into account as indicated in subclause 7.3.7.1 for Level II of approximation. When more refined analyses are required, the indications given in the same subclause for Level IV of approximation shall be respected.

For more information reference shall be made to CEB Design Manual "Structural Effects of Time-dependent Behaviour of Concrete", CEB Bulletin 142, 1984; CEB Bulletin 215, 1993; Section 4.1.6 of "Structural Concrete Textbook" *fib* Bulletin 52, 2010; "Analysis of Creep and Shrinkage Effects in Concrete Structures" to be published as ACI document 209.3R.

Structures may be conveniently classified according to their levels of sensitivity to time-dependent effects. The lower levels are represented by small and simple structures. The higher levels refer to important, large and technically complex structures. Typical examples of highly sensitive structures are segmentally built large span cantilever and arch bridges joined

Influence on the safety margins with respect to the ultimate limit state of strength depends on the ductile behaviour of the structure or structural element and can become a concern in presence of significant time-dependent amplifications of action effects such as internal forces and moments in cases where these effects cannot be redistributed, especially when the ultimate limit state is governed by non-plastic failure of concrete.

In slender or thin structures or structural elements and whenever second order effects are of importance, the increase of deflections due to creep reduces the safety margins with respect to instability and may lead to creep buckling; the unfavourable influence of shrinkage should be considered as well.

The guidelines given in the following apply essentially to the verifications with respect to serviceability limit states.

7.2.4.2 Levels of refinement of the analysis

When choosing the level of refinement for the analysis the following aspects should be considered:

– sensitivity of the structure to the time-dependent behaviour of concrete;

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at a later stage, cable-stayed bridges and structures, steel-tied arches, concrete or steel and concrete structures for high-rise and supertall buildings, steel-concrete composite beams or framed structures, structures with high ratios of prestressing and reinforcing steel, large span slender concrete arches and shells, structures stressed by jacking, off-shore, marine and nuclear structures, etc.

In general, a high sensitivity of the structure to the time-dependent behaviour of concrete may be responsible of final uneconomies in its service life costs. A reduction of this sensitivity by proper design and construction provisions, which may require higher initial costs, besides contributing to the reduction of the margins of uncertainty in the assessment of long-term reliability, may result also in service life economies.

Excessive refinement is not warranted for low to moderate sensitivity structures, and in the preliminary and conceptual design stages of all types of structures Sophisticated and laborious analyses should be reserved to important, sensitive, and very sensitive structures in their final stages of design.

A refined creep and shrinkage prediction model ought always to be used for structures analyzed by sophisticated computational methods, while excessive refinement in the analysis should be avoided if the prediction of material properties is poor. The error caused by replacing an accurate prediction of creep and shrinkage values on the basis of the most realistic available models with a simple but crude estimation is often larger than the gain from replacing simplified analysis approaches with sophisticated approaches.

Important uncertainties affect both, and to large extent, the prediction of the material response and the evaluation of the consequent structural response.

Appropriate confidence limits should be better considered also in these cases on the basis of adequate estimations.

– importance of the limit state under consideration;
– design stage;

– reliability of the information on material properties (prediction on the basis of the prediction models like e.g. the model given in subclause 5.1.9.4, or prediction accompanied by tests at early ages, or test extrapolation; mean cross-section behaviour or local rheological properties within the cross-section, etc.).

A balanced attention should be given in any case to both the material properties problem and to the structural analysis problem.

7.2.4.3 Probabilistic and deterministic approach

The problem of evaluation of time-dependent effects in concrete structures in the serviceability domain is statistical in nature, as most of the contributing factors are inherently random with significant coefficients of variation.

A deterministic approach based on mean prediction of time-dependent effects may be adequate for low to moderate sensitivity structures, and whenever a refined analysis is not required.

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As evidenced in subclause 5.1.9.4, the deformation prediction suffers of important uncertainties, due to inherent scatter of creep and shrinkage strains, errors of the model and randomness of material properties and environment, and may result in a considerable prediction error.

In ACI 209.23-08 "Guide for Modeling and Calculating Shrinkage and Creep in Hardened Concrete", ACI 2008, besides the 1999 edition of the model of subclause 5.1.9.4, three other alternative mean-cross-section prediction models are presented, termed ACI 209R-92, GL2000 and B3. The last model predicts also basic creep (creep at constant temperature and moisture content), considered a material constitutive property independent of the specimen size and shape.

In cases of highly sensitive structures for which cracking is a dominant consideration for serviceability and safety (e.g. marine and nuclear structures), a realistic analysis of the time evolution of internal stresses and strains within cross-sections and throughout the structure by appropriate discretization techniques should be based on the description of local rheological properties (constitutive laws for a material point of the concrete), taking into account their intrinsic non-linearities, coupling with moisture and temperature distributions and non-linear effects of cracking. Reference shall be made to specialized literature [see e.g. Z. P. Bažant et al. "Prediction of creep and shrinkage and their effects in concrete structures: Critical appraisal", in: Tanabe et al. (eds), Proc. Eighth Int. Conf. Creep, Shrinkage and Durability Mechanics of Concrete and Concrete Structures, Taylor and Francis, 2009, pp. 1275-89, and referenced literature. See also subclause 7.2.4.11 *(b)*].

A probabilistic approach is highly recommendable for sensitive structures, and becomes mandatory for very sensitive ones. The design of these structures should be based on predictions of extreme values of time-dependent effects that are exceeded with a certain specified small probability, such as 5%. The probabilistic approach should take into account the various factors of uncertainty.

Concerning the prediction of creep and shrinkage properties, reference should be made to the statistical indicators of the prediction model being considered. Updating the model parameters by testing is recommended in any case.

As for the evaluation of the consequent structural effects, the use of refined analyses contributes to the reduction of uncertainties.

7.2.4.4 Prediction models for concrete and significance of the analysis

When adopting a prediction model for creep and shrinkage of concrete of the type given in subclause 5.1.9.4, attention must be paid to its range of applicability as specified in subclause 5.1.9.4.2.

While this kind of models that predict the mean cross-section behaviour of a concrete member is generally suitable for the analysis of time-dependent effects on the overall response of structures (with the exception of highly sensitive structures) in terms of internal actions, restraint reactions and deformations, it should be applied with some cautions in the analysis of creep and shrinkage effects on local stress distributions within cross-sections, where larger errors may be introduced.

Rather, the results of such cross-sections analyses should be considered to have essentially a nominal value, and not as realistic descriptions of the actual stress values in the sections. However, for ordinary structures, in which fine cracking of concrete is not much of concern, or ample prestress is provided, more refined analyses are normally not needed.

In the case of cross-sections composed of elements with different thickness or environmental exposure conditions (e.g. in box girders), the non-uniformity of drying can be captured, in a simple but rather approximate way, applying the mean cross-section prediction model for creep and shrinkage to each component separately.

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For the range of validity of the linearity assumption, see subclause 5.1.9.4.3. Hygrotermal effects and cracking introduce major deviations from the principle of superposition. Non-linearities due to high stresses may be taken into account on the basis of subclause 5.1.9.4.3 *(d).*

Shrinkage strains may be considered as imposed stress independent strains.

In constitutive relations (5.1-62) or (7.2-25) strain or, respectively, stress may be considered as a functional of the previous stress (or strain) history alone.

If, on the contrary, the histories of the strain or, conversely, of the stress are prescribed in Eq. (5.1-62) or in Eq. (7.2-25), respectively, these equations represent Volterra hereditary integral equations for the determination of the corresponding stress or, respectively, strain histories.

Generalization to multiaxial stress of constitutive relation (5.1-62) requires the additional assumption of constancy of creep Poisson's ratio of concrete νc. For further information reference is made to Z. P., Bažant "Theory of Creep and Shrinkage in Concrete Structures: a Précis of Recent Developments", Mechanics Today, Vol. 2, Pergamon Press, 1975, pp. 1-93; J. Salençon "Viscoélasticité pour le Calcul des Structures", Presses des Ponts et Chaussées, 2009.

Different values of t_0 must be considered and repeated solutions of Eq. (7.2-26) must be performed to obtain $R(t,t')$ from $J(t,t')$. The procedure for numerical solution indicated in subclause 7.2.4.11 *(a)* shall be adopted.

7.2.4.5 Time-dependent analysis based on aging linear viscoelasticity

Within the limits specified in subclauses 5.1.9.4. and 7.2.4.4, the time-dependent analysis of concrete structures, for what concerns the evaluation of the effects of creep, may be based on the theory of aging linear viscoelasticity.

Aging linear viscoelasticity implies the validity of the principle of superposition of the responses in terms of strains or stresses to all imposed stress or strain increments applied during the time.

7.2.4.6 Constitutive laws in aging linear viscoelasticity

The application of aging linear viscoelasticity gives the stress-strain constitutive relation in the form of the hereditary integral relation (5.1-62) representing the strain response to a sustained variable imposed stress history with initial value $\sigma_c(t_0)$. In this case, the time-dependent behaviour of concrete is fully characterized by the compliance function $J(t,t')$.

Alternatively, with the same assumptions and range of validity, the stress response to a sustained variable imposed strain history with initial value $\varepsilon_\sigma(t_0)$ may be written as:

$$\sigma(t) = \varepsilon_{c\sigma}(t_0)R(t,t_0) + \int_{t_0}^{t} R(t,t')d\varepsilon_{c\sigma}(t') \qquad (7.2\text{-}25)$$

where:

$R(t,t')$ is the relaxation function, representing the stress response at time t to a sustained constant unit imposed stress-dependent strain applied at time t';

$R(t,t')$ can be obtained from the compliance function $J(t,t')$ specified by the creep prediction model being considered as the stress response in Eq. (5.1-62) for a constant unit imposed stress dependent strain $\varepsilon_{c\sigma} = 1$. Therefore, the compliance and the relaxation functions are reciprocally related by the integral equation:

$$1 = R(t_0,t_0)\,J(t,t_0) + \int_{t_0}^{t} J(t,t')dR(t',t_0) \qquad (7.2\text{-}26)$$

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The ranges of applicability of these simplifications are given in the corresponding subclauses.

The application of this assumption is based on the following considerations:

- the differences in the rheological properties of the concrete along the structure in terms of long-term values of creep and shrinkage strains, are usually rather contained if compared to the large magnitude of these values obtained from prediction models;
- the influence of the non-homogeneities due to the presence of reinforcement is generally small in most prestressed structures, due to the small geometrical percentage of reinforcement and to the lack of cracking;
- although this influence may be larger in ordinary reinforced concrete structures, as a consequence also of the interaction between creep and cracking, it may equally be disregarded in most cases, in particular if the analysis does not concern specifically local effects and the structure does not pertain to the high ranks of sensitivity;
- the stress-independent yielding of the restraints may be considered as a set of imposed deformations at the points of applications of the restraints of the structure;

7.2.4.7 Simplified approaches for time-dependent analysis

When a highly refined analysis is not required, some convenient simplifications may be introduced:

- at the level of the structural model, by the introduction of the assumption of an effective rheological homogeneity for the concrete structure (subclause 7.2.4.8) or for the concrete part of a structure including steel structural elements (subclause 7.2.4.9) when performing the analysis of the overall behaviour of the structure;
- at the constitutive level, through the adoption of the approximate algebraic formulation of the AAEM method for the constitutive relation (5.1-62) (subclause 7.2.4.10).

7.2.4.8 Effective homogeneous concrete structures with rigid or stress-independent yielding of restraints

A simplification of the structural model consists in neglecting the non-homogeneities in the rheological properties of the material and referring to an effective homogeneous concrete structure of averaged creep and shrinkage properties with rigid restraints.

Accordingly, a set of simple formulations for the determination of the overall time-dependent response of the structure is obtained from the fundamental theorems of the theory of aging linear viscoelasticity. The system of the stresses $S(t)$ and of the deformations $D(t)$ of the structure, under sustained imposed loads or deformations or consequent to a modification of the restraint conditions, may then be evaluated on the basis of the stresses $S^{el}(t)$ and deformations $D^{el}(t)$ for an elastic structure of constant reference elastic modulus E_c.

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– the case of restraints characterized by an elastic behaviour, or, equivalently, of the presence of steel structural elements considered as elastic elements, is discussed in subclause 7.2.4.9.

The formulations obtained from the assumption of effective rheological homogeneity are particularly apt for the serviceability assessment of low to moderate sensitivity structures, and, especially, in the preliminary and conceptual design stages for a wider class of structures. In fact, their sound theoretical fundaments guarantee their reliability from the conceptual point of view, while the reference to averaged rheological properties allows capturing the basic orders of magnitude. Therefore, if extreme and very special cases characterized by significant non-homogeneities in the concrete properties are excluded, the general trends of the time-dependent phenomena under consideration are unquestionably seized.

Assuming the principle of superposition to be valid, the solutions presented here in sections *(a)* to *(d)* for each separate problem in terms of stress and strain histories may be superimposed.

(a) Imposed loads

Under sustained variable imposed loads the elastic stresses are not modified by creep. The deformations at time t may be evaluated summing up the increments of the elastic deformations factored by the non-dimensional creep factor $E_c\, J(t,t')$ (1st theorem of aging linear viscoelasticity):

$$S(t) = S^{el}(t) \tag{7.2-27}$$

$$D(t) = E\int_0^t J(t,t')dD^{el}(t') \tag{7.2-28}$$

where:

$S(t)$ is system of the stresses (internal stresses, internal forces, restraint reactions) at time t;

$S^{el}(t)$ is elastic solution for the system of the stresses in the associated elastic problem at time t;

$D(t)$ is system of the deformations (internal strains, internal deformations, external displacements) at time t;

$D^{el}(t)$ is elastic solution for the system of the deformations in the associated elastic problem at time t;

E_c is reference elastic modulus for the associated elastic problem.

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Under sustained constant loads creep does not alter the initial stresses in the structure. The initial deformations are followed by their creep induced gradual increase, the deformation history being related by an affinity to the compliance function (*creep problem* at the structural level).

For a normal weight ordinary structural concrete, the magnification factor $E_{ci}(t_0)J(t,t_0)$ at the end of a service life of 100 years can easily reach values in the range of 3 to 5 for typical ages t_0 at loading, depending on the creep prediction model, on the values of the various influencing parameters and on the reference value of the elastic modulus $E_c(t_0)$ considered in the evaluation of the initial deformations (CEB Bulletin 215 and *fib* Bulletin 52).

If reference is made to the prediction model of subclause 5.1.9.4.3, the magnification factor is $E_{ci}(t_0)J(t,t_0) = \left[1+\varphi(t,t_0)E_{ci}(t,t_0)/E_{ci}\right]$.

When applying Eq. (7.2-33) for the evaluation of the effects of cyclic time histories of the imposed deformations, e.g. when considering the effects of restrained annual thermal variations, reference to the comments of subclause 7.2.4.11 *(a)* is appropriate.

Under sustained constant imposed deformations creep does not alter the initial deformations. The initial stresses are followed by their creep induced gradual decrease, the stress history being related by an affinity to the relaxation function (*relaxation function* at the structural level).

For a normal weight ordinary structural concrete, the creep induced mitigation factor $R(t,t_0)/E_{ci}(t_0)J(t,t_0)$ lies in the range 0.1 to 0.3 at the end of a service life of 100 years for typical ages t_0 at loading, depending on the creep prediction model, on the values of the various influencing parameters and on the reference value of the elastic modulus $E_c(t_0)$ considered in the evaluation of the initial stresses (CEB Bulletin 215 and *fib* Bulletin 52). Therefore, as a

The response to a system of sustained constant loads imposed at $t = t_0$ is thus given by:

$$S(t) = S^{el,t_0} \tag{7.2-29}$$

$$D(t) = E_{ci}(t_0)J(t,t_0)D^{el,t_0} \tag{7.2-30}$$

where:

S^{el,t_0}, D^{el,t_0} are the elastic solutions for the system of the stresses and deformations calculated with the initial value $E_{ci}(t_0)$ of the elastic modulus.

(b) Imposed deformations

Under sustained variable imposed deformations the elastic deformations are not modified by creep. The stresses at time t may be evaluated summing up the increments of the elastic stresses factored by the non-dimensional relaxation factor $R(t,t')/E_c$ (2nd theorem of aging linear viscoelasticity):

$$D(t) = D^{el}(t) \tag{7.2-31}$$

$$S(t) = 1/E_c \int_0^t R(t,t')dS^{el}(t') \tag{7.2-32}$$

The response to a system of sustained constant imposed deformations imposed at $t = t_0$ is thus given by:

$$D(t) = D^{el,t_0} \tag{7.2-33}$$

$$S(t) = \frac{R(t,t_0)}{E_c(t_0)} S^{el,t_0} \tag{7.2-34}$$

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result of relaxation, creep exerts a beneficial action in significantly relieving the stresses due unintentional sudden imposed deformations. On the other hand, it drastically reduces the beneficial effects of stress corrections artificially induced e.g. by jacking.

As a general indication, the formulations of Eqs. (7.2-35) to (7.2-37) show the importance of stress redistributions due to creep in case of changes in the structural system. In fact, the inspection of the diagrams of the redistribution function $\xi(t,t_0,t_1)$ shows that for average creep properties and for typical ages t_0 at loading and t_1 at change in structural system, characterizing usual construction sequences, the long-term values of ξ are contained in a range between 0.5 and 0.9, if reference is made to the prediction model of subclause 5.1.9.4.3. Higher values, in the range of 0.7÷0.9, are obtained for GL2000 and B3 models (CEB Bulletin 215 and *fib* Bulletin 52).

As a result, the long-term distribution of stresses, internal actions and restraint reactions tends to approach those of the structure in its final structural system. Therefore, the creep induced stress redistribution must be accounted for in the serviceability analyses of the structure and may require additional reinforcement and/or the introduction of additional prestressing after the change in the structural system. When assessing the reliability with respect to the ultimate limit state, this redistribution must be properly considered in case of structures in which the internal actions cannot be redistributed due to limited available ductility, the collapse being governed by the brittle failure of concrete.

One possible strategy for reducing the creep induced stress redistribution consists of reducing the difference in the stress distributions corresponding to the two reference elastic configurations of the original and final structural systems, respectively. For instance, in prestressed structures under bending, in cases where the additional restraints prevent the delayed flexural deformations, this may be done by balancing the permanent loads in the original structural system by prestressing, thus reducing the corresponding flexural deformations. Therefore, the additional restraints have a limited effect in altering the original elastic stress distribution.

(c) Modification of restraint conditions after loading

In a structure subjected to sustained constant loads imposed at time t_0, whose initial structural system 1 is modified into a final structural system 2 by the introduction of additional restraints at time $t_1 \geq t_0^+$, the stress distribution evolves for $t > t_1$ according to Eq. (7.2-35) (3rd theorem of aging linear viscoelasticity):

$$S^2(t) = S^{el,1} + \Delta S^1(t) = S^{el,1} + \Delta S^{el,1}\xi(t,t_0,t_1) \qquad (7.2\text{-}35)$$

where:

$S_2(t)$ is system of the stresses at $t > t_1$ in the structure in the final structural system 2;

$S^{el,1}$ is elastic solution for the system of the stresses in the structure in the initial structural system 1;

$\Delta S^{el,1}$ is correction to be applied to the elastic solution $S^{el,1}$ in order to comply with the elastic solution $S^{el,2}$ corresponding to an application of the loads in the final structural system 2, i.e.:

$S^{el,2} = S^{el,1} + \Delta S^{el,1}$;

$\xi(t,t_0,t_1)$ is redistribution function.

The redistribution function ξ is a non-dimensional factor whose values lie in the interval (0,1) (with $\xi = 0$ for $t = t_1$), and is related to the compliance function through the integral equation:

$$J(t,t_0) - J(t_1,t_0) = \int_{t_1}^{t} J(t,t')d\xi(t',t_0,t_1) \qquad (7.2\text{-}36)$$

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The delayed introduction of additional restraints is sometime conceived as an artifice to improve the behaviour of the structure under permanent loads, when the corresponding stress distribution in the original structural system is more favourable, if compared to the one that would be obtained, for the same loads, in the final system. In such cases, the introduction of the additional restraints, intended to improve the response to live loads and the final robustness of the structure by an increase of its statical redundancy, substantially reduces the original benefits, because of the significant creep induced stress redistributions altering the original response to permanent loads (e.g. fixed concrete arches provisionally built as three-hinged arches with a centre line corresponding to the funicular curve for dead loads).

Different values of t_0 and t_1 must be considered and repeated solutions of Eq. (7.2-36) must be performed to obtain $\xi(t,t_0,t_1))$ from $J(t,t')$. The procedure for numerical solution indicated in subclause 7.2.4.11 *(a)* shall be adopted.

For further information reference is made to M. A. Chiorino "A Rational Approach to the Analysis of Creep Structural Effects", ACI SP-227, 2005, pp.107-141.

Design aids. For a given creep prediction model, and for the corresponding compliance function $J(t,t')$, design aids can be provided for the evaluation of the related functions $R(t,t')$ and $\xi(t,t_0,t_1)$ in terms of set of graphs of these three functions and of the aging coefficient $\chi(t,t')$ introduced in subclause 7.2.4.10 (refer to CEB Bulletin 215 for the creep prediction model of CEB Model Code 1990; a few examples for the models indicated in subclause 7.2.4.4 are given in *fib* Bulletin 52), or in terms of computational programs to be downloaded (see e.g. www.polito.it/creepanalysis) or inserted in computational softwares. For further information reference is made to M. Sassone, M. A. Chiorino "Design Aids for the Evaluation of Creep Induced Structural Effects", ACI SP-227, 2005, pp. 239-259.

(d) Multiple changes in the structural system

In cases where the transition from the initial to the final structural system is obtained by means of several different restraint modifications applied at times $t_i \geq t_0^+$ ($i = 1,\ldots,j$), the redistribution effects consequent to every single change in the structural system may be superimposed in time. Therefore, the system of the stresses evolves for $t > t_1$ according to Eq. (7.2-38) (4th theorem of aging linear viscoelasticity):

$$S^{j+1}(t) = S^{el,1} + \sum_{i=1}^{j} \Delta S^{el,i} \xi(t', t_0, t_i) \qquad (7.2\text{-}37)$$

where:

$\Delta S^{el,i}$ is correction to be applied, in the associated elastic problem, to the elastic solution $S^{el,i}$ in order to respect the geometrical conditions imposed by the additional restraints of structural system i+1, imagined as introduced before the loads.

7.2.4.9 Effective homogeneous concrete structures with additional steel structural elements

If the main cause of heterogeneity is represented by the presence of steel structural elements that may be considered equivalent to redundant elastic restraints, while the concrete part of the structure may still be approximately regarded as an effective homogeneous structure of averaged rheological

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Reference shall be made to the specialized literature indicated in *fib* Bulletin 52, p. 53, and to ACI 2090.3R Guide.

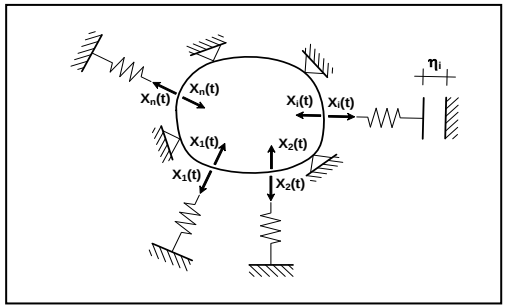


Figure 7.2-17: Schematic representation of a homogeneous structure with n redundant elastic restraints

Although the theoretical condition of invariance can hardly be obtained in practical cases, due to construction sequences and differences in the creep properties along the structure, an appropriate stressing of the steel restraints can substantially reduce the variation in time of the stress state. This artifice, which may be responsible of higher initial costs as the steel restraints must be designed for higher forces, contributes to the reduction of uncertainties in the long-term reliability assessments and may result in service life economies.

properties, a set of simple formulations can be provided for the determination of the overall time-dependent response of the structure for the cases of constant imposed loads or of constant deformations imposed at the points of applications of the restraints, or of a change in structural system (Fig. 7.2-17).

These formulations, which are expressed in matrix form, represent the extension to the case of elastic restraints of the aging linear viscoelastic formulations presented in subclause 7.2.3.8.

Typical structures that can be schematically modelled as effective homogeneous concrete structures with redundant elastic restraints are e.g. tied arches and frames, and cable-stayed bridges and structures. Although the final assessment of these types of structures - being inherently complex and usually characterized by intricate construction procedures and phases - must normally be performed by proper sequential approaches adopting the numerical techniques indicated in subclause 7.2.3.11 and accounting for possible causes of non-linearity (e.g. due cable sag in stayed structures), the reference to this schematic model and the inherent formulations allows setting some general guidelines listed in the following, which are particularly useful in the preliminary and conceptual design stages.

(a) Constant imposed loads

Differently from the case of rigid restraints, for which the invariance of the state of stress is stated by Eq. (7.2-27), the initial elastic state of stress in the restraints and in the structure is significantly altered by creep. The higher the deformability of the restraints, the higher is the difference between the initial and long-term values. In the long term, the system of stresses tends to approach the solution corresponding to the case of rigid restraints.

A reduction of the time dependence of the state of stress under permanent loads when highly deformable restraints are adopted (e.g. high strength steel stays of small section as in cable-stayed bridges), can be obtained through a convenient stressing of these restraints. In fact, in the theoretical case of an effective homogeneous structure with elastic restraints introduced all at the same time, the invariance of the stress state is obtained forcing the restraints up to the values of the rigid restraints reactions.

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In cable-stayed bridges, stressing of the stays is normally performed in more than one step to allow for adjustments, as required by usual cantilever sequential construction techniques. One other reason for stressing the stays is the need of eliminating the non linear mechanical behaviour of the stays due to cable sag. For more information reference is made to C. Casalegno et al. "Time dependent effects in cable-stayed bridges built by segmental construction", Proceedings 3rd *fib* International Congress, Washington 2010.

Eq. (7.2-40), with the expression (7.2-41) for the aging coefficient χ, corresponds exactly to Eq. (5.1-62) for all one-step imposed action histories resulting from linear combinations of a creep and a relaxation problem, i.e. for all the strain histories of the type:

$$\varepsilon_{c\sigma}(t) = c_1 + c_2 J(t,t_0) = c_1 + c_2\left[\frac{1}{E_c(t_0)} + \frac{\varphi_{28}(t,t_0)}{E_{ci}}\right] = a + b\varphi_{28}(t.t_0) \tag{7.2-38}$$

where the time-dependent part is related by an affinity to the compliance function $J(t,t_0)$ or, equivalently, to the creep coefficient $\varphi_{28}(t,t_0)$. This includes a broad range of strain (and corresponding stress) histories. With sufficient accuracy, its use may be extended to cover a large number of actual action histories in structures showing an initial finite or zero value at $t = t_0$ and a time-dependent part varying at a gradually decreasing -rate over wide time intervals. In current use of AAEM method Eq. (7.2-39) is given a quasi-elastic incremental formulation relating the variations of the total strain $\Delta\varepsilon_c(t)$ and of the stress $\Delta\sigma(t)$ occurring over the interval (t_0^+,t) after the initial

(b) Constant imposed deformations

The stress losses are less pronounced with respect to the case of rigid restraints represented by Eq. (7.2-34). This is a consequence of the elastic energy stored in the system of the restraints. If the deformations are imposed at the points of applications of the restraints, the invariance of the stress state may be obtained as specified in (a).

(c) Modification of restraint conditions after loading

The theoretical solutions show that the system of elastic restraints contributes to a lower degree to the variation of the original system of the stresses in the structure, attracting lower values of restraint reactions, with respect to the case of delayed additional rigid restraints discussed in subclause 7.2.4.8 (c).

7.2.4.10 Approximate algebraic formulation for the constitutive relation: age-adjusted effective modulus (AAEM) method

For a compliance function expressed in the form of Eq. (5.1-61) the hereditary integral constitutive relation of Eq. (5.1-62) may be written in the following equivalent algebraic form:

$$\varepsilon_c(t) = \sigma_c(t_0)J(t,t_0) + [\sigma_c(t) - \sigma_c(t_0)]\left[\frac{1}{E_c(t_0)} + \chi(t,t_0)\frac{\varphi_{28}(t,t_0)}{E_{ci}}\right] + \varepsilon_{cn}(t)$$

$$= \frac{\sigma_c(t_0)}{E_{c,ef}(t,t_0)} + \frac{\sigma_c(t) - \sigma_c(t_0)}{E_{c,adj}(t,t_0)} + \varepsilon_{cn}(t) \tag{7.2-39}$$

having introduced the aging coefficient:

$$\chi(t,t_0) = \frac{1}{1 - R(t,t_0)/E_c(t_0)} - \frac{1}{E_c(t_0)J(t,t_0) - 1}$$

$$= \frac{E_c(t_0)}{E_c(t_0) - R(t,t_0)} - \frac{E_{ci}}{E_c(t_0)\varphi_{28}(t,t_0)} \tag{7.2-40}$$

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stressed state at time t_0^+. In both alternative procedures, the responses to multistep load histories can be obtained by superimposing the solutions for several one-step histories. For further information reference is made to Z.P. Bažant "Numerical determination of long-range stress history from strain history in concrete", Material and Structures, Vol. 5, 1972, pp. 135-141; M. Jirásek, Z.P., Bažant, "Inelastic Analysis of Structures", Wiley and Sons, 2002.

Under these conditions, the AAEM method may be applied firstly to the analysis of the overall time-dependent response of concrete structures that may be considered as homogeneous on the basis of the same assumption of effective rheological homogeneity discussed in subclause 7.2.4.8. The same ranges of applicability of the solution shall be considered.

The accuracy of the results remains normally satisfactory if the application is extended to cover the cases of heterogeneous structures consisting of concrete portions with different creep properties, and/or containing steel elements. However, in case of very complex structures and construction sequences, preference shall be given to the numerical approaches of the General method illustrated in subclause 7.2.4.11.

The AAEM algebraic formulation of the constitutive relation in association with the assumption of plane sections shall be normally adopted for the estimation of stress redistributions due to creep and shrinkage in non-homogeneous and composite cross-sections of one dimensional elements, like prestressed concrete sections with prestressing and reinforcing steel in one or multiple layers, concrete-concrete and steel-concrete composite sections, etc. In fact, while on one hand the AAEM solutions are generally very accurate with respect to the use of the integral formulation (5.1-62) of the constitutive law, on the other hand reference to the comments of subclause 7.2.4.4 on the nominal character of these estimations is appropriate.

For any given compliance function $J(t,t')$, the aging coefficient $\chi(t,t')$ can be determined from Eq. (7.2-40) for different values t_0 of t' introducing the values of the relaxation function $R(t,t')$ calculated on the basis of the numerical procedure indicated in subclause 7.2.4.11 *(a)*. For design aids refer to subclause 7.2.4.8.

the effective modulus:

$$E_{c,ef}(t,t_0) = \frac{1}{J(t,t_0)} = \frac{E_c(t_0)}{1+[E_c(t_0)/E_{ci}]\varphi_{28}(t,t_0)} \qquad (7.2\text{-}41)$$

and the age-adjusted effective modulus:

$$E_{c,adj}(t,t_0) = \frac{E_c(t_0)}{1+\chi(t,t_0)[E_c(t_0)/E_{ci}]\varphi_{28}(t,t_0)} \qquad (7.2\text{-}42)$$

$\chi(t,t_0)$ varies relatively little with the age t_0 for sufficiently long elapsed times. For typical values of t_0 and other influencing parameters, its long-term values are contained in range between 0.6 and 0.9 for the prediction model of subclause 5.1.9.4 and in a narrower range for other models like GL2000 and B3.

The adoption of a fixed long-term value comprised in this narrower range, independently of the age at loading and of the creep properties of the structural element being considered, leads often to satisfactory accuracies in the evaluation of the long-term structural responses, particularly in the conceptual and preliminary design stages and in the assessment of structures of low sensitivity to time-dependent effects.

In such situations, it is often adequate to use the value χ =0.8.

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Notation. In the referenced literature, and in ACI 209R-92 and B3 prediction models, the creep coefficient φ (noted also as ϕ) represents the ratio between the creep strain and the initial elastic strain $1/E_c(t_0)$ at the age t_0 at loading. The notation φ_{28} adopted here must be intended to correspond, on the contrary, to the definition of the creep coefficient in Eq. (5.1-60).

Reference is made to constitutive relation (5.1-62) as the majority of creep prediction model specify the compliance function $J(t,t')$. No real advantage would be obtained from the computational point of view if reference were made to the equivalent constitutive relation (7.2-25), even in the frame of the equilibrium method. In fact, the relaxation function $R(t,t')$ needs to be numerically calculated through Eq. (7.2-26), from the given compliance function

The Volterra integral equations of structural creep problems can be solved analytically only for some simple forms of the compliance function $J(t,t')$. For the compliance function of the creep prediction model given at subclause 5.1.9.4.3, and for the other modern creep prediction models referenced in subclause 7.2.4.4, an incremental numerical solution of the type indicated here is required.

7.2.4.11 General method

The most general and refined approach for the evaluation of creep and shrinkage structural effects in the frame of aging linear viscoelasticity consists in the incorporation of the constitutive relation (5.1-62) for concrete into the computational algorithms of continuum mechanics or of structures composed of beams.

The use of the aging linear viscoelastic model for the concrete portions of the structure and of the elastic model for steel leads to a system of Volterra integral compatibility or equilibrium equations, when the force or, respectively, the deformation method is adopted for the structural analysis.

A completely general, accurate and most effective approach is to obtain first the incremental form of Eq. (5.1-62) for a small time step by one of procedures indicated in the following. The incremental form represents a linear elastic stress-strain relation with initial strains, in which the elastic moduli and initial strains vary through the body, and from step to step. Thus the problem of aging linear viscoelasticity gets converted, already at the constitutive level, to a sequence of elasticity problems. This eliminates any need to ever formulate the Volterra integral equations governing structural variables.

Any elastic finite element program, used repeatedly in a loop, can thus be generalized for creep, allowing the formulation of the solution in terms of a time-history.

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In consideration of the particular shape of the creep curves and of the typical imposed strain histories in most structural problems, both characterized by increments at decreasing rate, if fluctuating strains such as cyclic thermal strains are excluded, it is possible to gradually increase the time steps Δt_k in order to reach the long-term response with an acceptable number of steps, considering that the first intervals should be of the order of fractions of the day (due to the high initial slope of the creep curves).

The same incremental numerical procedure shall be adopted for the solution of Eqs. (7.2-26) and (7.2-36) to obtain the relaxation function $R(t,t')$ and the redistribution function $\xi(t,t_0,t_1)$ from the given compliance function $J(t,t')$. The aging coefficient $\chi(t,t')$ can then be determined from Eq. (7.2-40). Further information and reference to computer programs in *fib* Bulletin 52.

This storage requirement used to be a computational burden, but for modern computers it matters only for systems with a huge number of unknowns.

The rate-type form is also advantageous to deal with variable humidity and temperature, as it allows separating the effects of variable pore humidity or temperature on creep viscosity from those on the aging rate, and with material damage (fracture and distributed cracking) as they are rate-dependent processes. Only with the rate-type form it is possible to meet the thermodynamic restrictions and introduce physical concepts related to these processes. Reference should be made to the specialized literature (Bažant 2009; ACI document 209.3R).

(a) Incremental numerical solution based on the hereditary integral

An incremental form may be obtained by replacing the integral over the past stress or strain history with a sum.

The procedure is based on the approximation of the superposition integral of Eq. (5.1-62) with finite sums using the 2nd order trapezoidal integration rule. The time t is subdivided by discrete times t_0, t_1, t_2,...t_i...t_k into sub intervals $\Delta t_i = t_i - t_{i-1}$ (i =1,2,...,k) and at each step the average value of the compliance function $\frac{1}{2}[J(t_k,t_i) + J(t_k,t_{i-1})]$ is adopted in the calculation. In order to allow strain histories with an initial finite step, times t_0 and t_1 are assumed to be coincident, so that $\Delta t_1 = t_1 - t_0 = 0$, and consequently $\Delta\varepsilon_{c\sigma}(t_1) = \varepsilon_\sigma(t_0)$.

At each step the calculations of the previous steps must be stored, so that the entire history of stress and strain must be stored in computer memory.

(b) Incremental numerical solution based on rate-type creep laws

The computation can be made more efficient approximating the integral-type constitutive law with a rate-type relation based on Kelvin or Maxwell chains of spring-dashpot rheological models for aging materials. In that case, the history needs not to be stored because it is implied by the current values of a few hidden variables. The resulting differential equations are of second order. Rate-type laws are particularly helpful for the solution of structural problems by means of the finite elements method, because they are immediately compatible with this computational approach.

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Figure 7.3-1 shows the possible range of strain distributions for concrete, reinforcing steel and prestressing steel. In the figure, the following limits are shown:

A = reinforcing strain limit;

B = concrete compression limit; and

C = concrete pure compression strain limit.

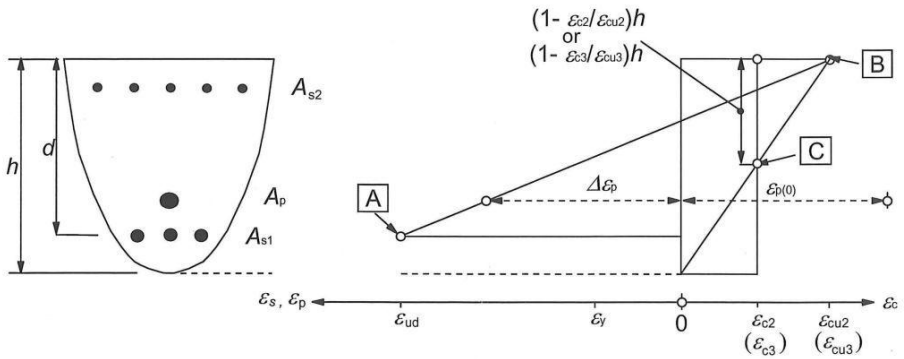


Figure 7.3-1: Possible strain distributions in the ultimate limit state

7.3 Verification of structural safety (ULS) for predominantly static loading

7.3.1 General

This subclause gives methods of verifying that, for a structure as a whole and for its component parts, the probability of an ultimate limit state exceeding the resistance of critical regions is acceptably small.

The determination of the partial safety coefficients and action effects shall be undertaken in accordance with the principles set out in chapter 4.

7.3.2 Bending with and without axial force

7.3.2.1 Beams, columns and slabs

This subclause applies to undisturbed areas of beams, slabs and similar types of members for which sections remain approximately plane before and after loading. The discontinuity regions of beams and other members, where plane sections do not remain plane, may be designed and detailed according to subclause 7.3.6.

When determining the ultimate bending resistance of reinforced or prestressed concrete cross-sections, the following assumptions are made:

- plane sections remain plane;
- the strain in bonded reinforcement or bonded prestressing tendons, whether in tension or in compression, is the same as that in the surrounding concrete;
- the tensile strength of the concrete is ignored;
- the stresses in the concrete are derived from stress-strain relations for the design of cross-sections as given in subclause 7.2.3.1.5;
- the stresses in the reinforcing and prestressing steel are derived from the design curves in subclauses 7.2.3.2 and 7.2.3.3; and
- the initial strain in the prestressing tendons is taken into account when assessing the stresses in the tendons.

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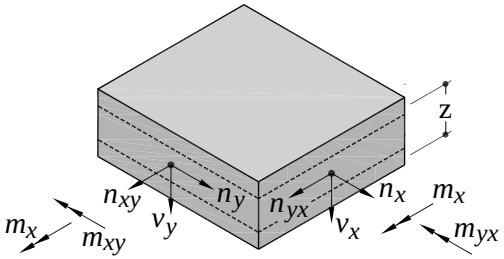


Figure 7.3-2: Three-layer plate model and stress resultants

The subscript notations "*inf*" and "*sup*" refer to the inferior and superior faces of the element. The inferior face is the tensile face for an element in positive bending.

The effective shear depth z may be taken as $0.9d$, where d denotes the distance between the extreme compression fibre and the mean planes of the reinforcement layers at the opposite face. The effective shear depth need not be taken as less than 0.72h.

For cross-sections with symmetrical reinforcement loaded by a compression force, the minimum eccentricity shall be taken as $e_0 = h/30$ but not less than 20 mm, where h is the depth of the section.

7.3.2.2 Shells

Shell elements may be modelled as comprising three layers (Figure 7.3-2 and Figure 7.3-3). The outer layers provide resistance to the in-plane effects of both the bending and the in-plane axial loading, while the core layer provides a shear transfer between the outer layers.

The action effects of the applied loads are expressed as eight components, three moments per unit width, 3 axial forces per unit width and two shear forces per unit width in directions parallel to the orthogonal reinforcement. The stress resultants m_x, m_y, m_{xy}, n_x, n_y, n_{xy}, v_x, v_y produce the following forces per unit width on the element:

$$n_{x\,inf,sup} = \frac{n_x}{2} \pm \frac{m_x}{z} + \frac{v_x^2}{2v_o}\cot\theta \qquad (7.3\text{-}1)$$

$$n_{y\,inf,sup} = \frac{n_y}{2} \pm \frac{m_y}{z} + \frac{v_y^2}{2v_o}\cot\theta \qquad (7.3\text{-}2)$$

$$n_{xy\,inf,sup} = \frac{n_{xy}}{2} \pm \frac{m_{xy}}{z} + \frac{v_x\ v_y}{2v_o}\cot\theta \qquad (7.3\text{-}3)$$

where:

θ is the inclination of the compression stresses in the core layer;

z is the average lever arm between the forces in the x and y directions in the top and bottom layers and the effective shear depth, respectively;

v_o is the principal transverse shear force per unit length and follows from

$$v_o = \sqrt{v_x^2 + v_y^2} \qquad (7.3\text{-}4)$$

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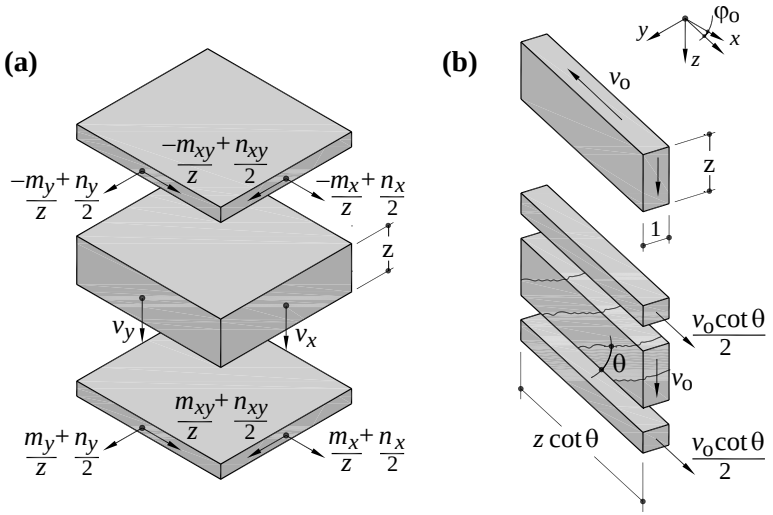


Figure 7.3-3: (a) Layer forces in sandwich model and (b) transfer of transverse shear force in uncracked and cracked core

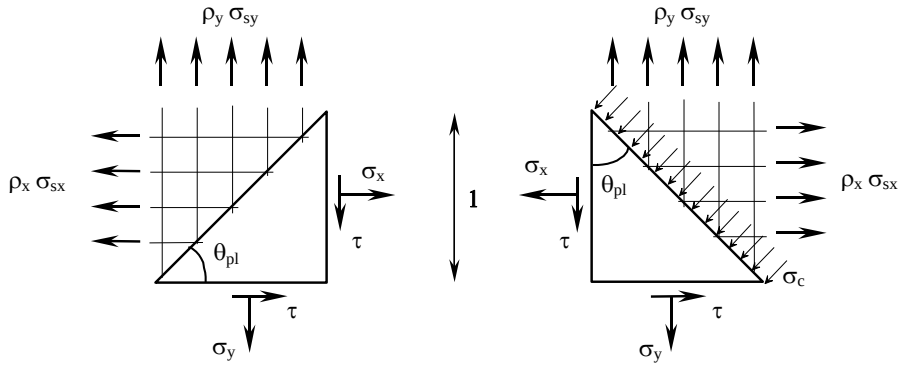


Figure 7.3-4: Stresses acting on and within a reinforced concrete element

For members with shear reinforcement the angle θ shall be selected in accordance with subclause 7.3.3.4. For members without shear reinforcement a value of cot θ = 2 may be used as is (implicitly) suggested in subclause 7.3.3.3.

Design of outer membrane layers

If at least one principal stress is in tension, the outer layers may be designed as membrane elements according to plasticity theory such that:

$$\sigma_{sx} = \frac{1}{\rho_x}\left(\sigma_x + \tau\cot\theta_{pl}\right) \le f_{yd} \tag{7.3-5}$$

$$\sigma_{sy} = \frac{1}{\rho_y}\left(\sigma_y + \tau\cot\theta_{pl}\right) \le f_{yd} \tag{7.3-6}$$

$$\sigma_c = \frac{\tau}{\sin\theta_{pl}\cos\theta_{pl}} \le \frac{\nu\, f_{ck}}{\gamma_c} \tag{7.3-7}$$

If no reinforcement is yielded and at least one principal stress is in tension:

$$\nu = \frac{1.18}{1.14 + 0.00166\sigma_{si}} \le 1.0 \tag{7.3-8}$$

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To ensure that the ductility demand is met, the term $|\theta_{pl} - \theta_{el}|$ in Eq. (7.3.9) should not be greater than 15°, unless refined calculations are undertaken to justify a higher value.

where σ_{si} is the maximum tensile stress (in MPa) in any layer of reinforcing steel ($i = x, y$).

If one or more layers of reinforcement yield:

$$\nu = \left(1 - 0.032\left|\theta_{pl} - \theta_{el}\right|\right) \cdot \frac{1.18}{1.14 + 0.00166\, f_{yd}} \qquad (7.3\text{-}9)$$

where:

θ_{pl} is the compression field angle with respect to x-axis at the ULS

θ_{el} is the first cracking angle with respect to the x-axis.

If both principal stresses are compressive:

$$|\sigma_2| \le \nu\, f_{cd} \qquad (7.3\text{-}10)$$

where σ_2 is the minor principal (compressive) stress and ν may be taken as 1.0 or determined in accordance with subclause 5.1.6.

Design of inner core layer

The shear core shall be designed in accordance with subclause 7.3.3.

7.3.3 Shear

7.3.3.1 General

Design shear force and shear resistance

The following equations are provided for the shear resistance of the webs of beams and the core layers of slabs and do not include the effects of flanges. Figure 7.3-3 and Figure 7.3-5 show the regions of members being designed both for slabs and beams, respectively.

In beams, a minimum quantity of shear reinforcement in accordance with subclause 7.13.5.2 shall be provided.

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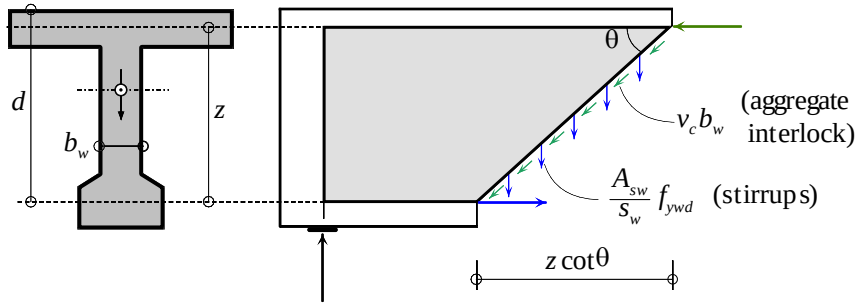


Figure 7.3-5: Forces in the web of a beam

The depth *d* denotes the effective depth in flexure which is defined as the distance from the extreme compressive fibre of the concrete to the resultant tensile force in the tensile reinforcing steel and tendons.

The dimension *z* may also be taken as the distance between the center lines of the top and the bottom chord, where the depth of the compression chord may be calculated for the location of maximum bending and a stress block according to Figure 7.2-12. For non-prestressed members *z* needs not be taken as less than 0.9*d*.

The shear resistance of a web or slab is determined according to:

$$V_{Rd} = V_{Rd,c} + V_{Rd,s} \geq V_{Ed} \qquad (7.3\text{-}11)$$

where:

V_{Rd} is the design shear resistance;

$V_{Rd,c}$ is the design shear resistance attributed to the concrete;

$V_{Rd,s}$ is the design shear resistance provided by shear reinforcement;

V_{Ed} is the design shear force.

The design can be based on a stress field analysis or a strut-and-tie model as outlined in subclause 7.3.6. Such models are especially suitable for the design of discontinuity regions (D-regions) at supports or transverse applied forces.

Alternatively, a cross-sectional design procedure may be applied. The corresponding rules are given in the following subclauses.

Cross-sectional design

For a cross-sectional design, the design shear force shall in general be determined at control sections at a location *d* from the face of supports (see Figure 7.3-6) and from discontinuities of geometry or transverse applied load shall be considered.

For the effective shear depth *z* a value of 0.9*d* can be assumed.

Other control sections may be required, for example in case of varying web widths along a span, for non-uniform or significant concentrated loads, or at sections near points of curtailment of reinforcement.

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For members containing mild steel reinforcement as well as prestressed tendons, the effective shear depth z can be taken as:

$$z = \frac{z_s^2 A_s + z_p^2 A_p}{z_s A_s + z_p A_p} \tag{7.3-12}$$

where z_s and z_p denote the distances between the centerline of the compressive chord and the reinforcement and tendon axes, respectively.

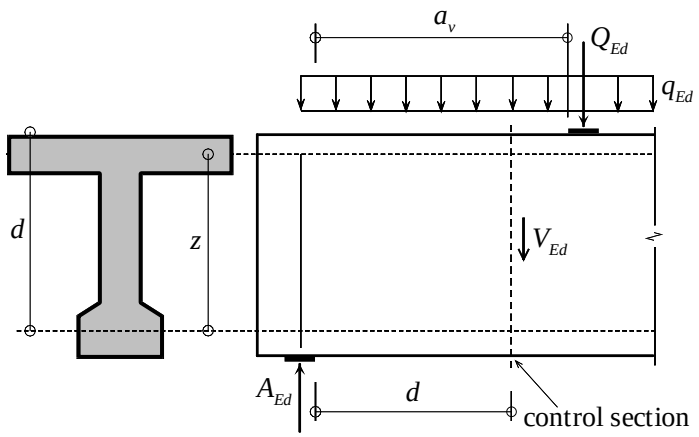


Figure 7.3-6: Definition of control section for sectional design

The effect of redistribution of internal forces in slabs with concentrated loads can result in higher shear capacities when compared to one-way slabs or beams subjected to uniformly distributed loadìng. This effect may be accounted for by assuming a uniform distribution of the shear force along a control width b_w, as shown in Figure 7.3-7.

Sections closer to supports than the distance d may be designed for the same shear force as at the control section provided that the member is directly supported.

Unless more refined modelling techniques are used to consider loads taken directly to a support through strut or arch action (see subclause 7.3.6), the following rules apply:

– the contribution of point loads applied within a distance of $d < a_v \leq 2d$ from the face of the support to the design shear force V_{Ed} may be reduced by the factor:

$$\beta = a_v/(2d) \tag{7.3-13}$$

– in the case of point loads applied as close as $a_v < d$ from the face of the support, the design shear force V_{Ed} shall be calculated with $\beta = 0.5$ as if the load was applied at $a_v = d$.

Where a concentrated load is applied to a slab near a support line, its capacity shall be checked for punching at the control perimeter around the loaded area, as described in subclause 7.3.5, and for shear at a control section taken parallel to the line of the support, as defined in Figure 7.3-7.

The control section is taken at the lesser of a distance equal to d and $a_v/2$ from the face of the support. The load distribution angle shall be taken as $\alpha = 45^o$ for the case of clamped edges and $\alpha = 60^o$ for simply supported edges.

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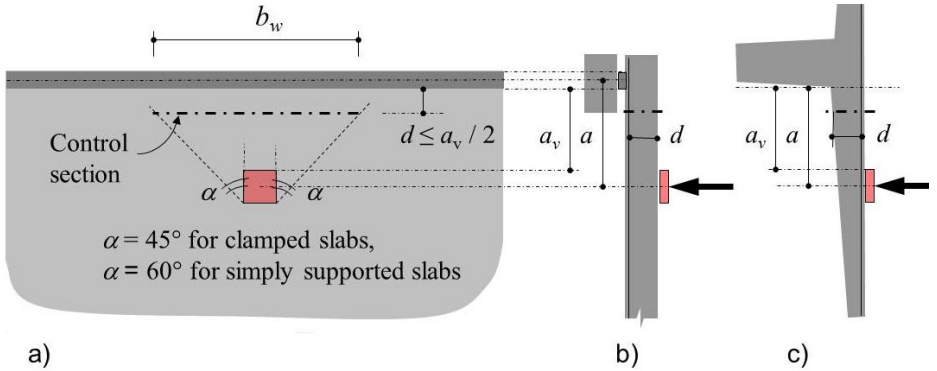


Figure 7.3-7: Location and length of the control section, bw, for the determination of the shear resistance of slabs with point loads located near a support-line; (b) simple edge support; (c) clamped edge support

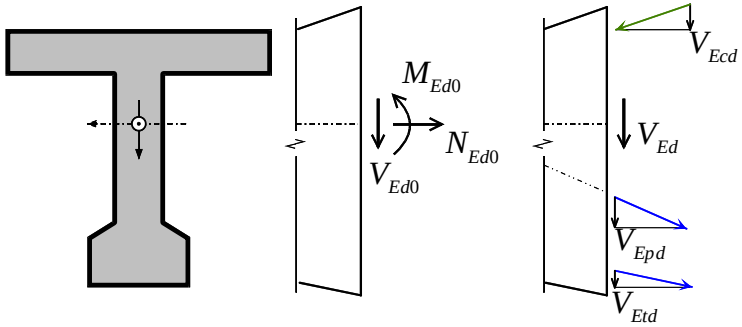


Figure 7.3-8: Contributions of inclined chord forces to design shear force (M_{Ed0}, V_{Ed0} and N_{Ed0} denote bending moment, shear and normal forces resulting from sectional analysis)

Membrane or arching action due to internal or external restraints further increases the design shear resistance and, as a consequence, the shear reinforcement may be decreased; however, the compression stresses in the concrete are enhanced and should be checked carefully. It is recommended to study such a situation with help of a strut-and-tie model (see subclause 7.3.6.).

For determining V_{Ed} the shear force from the sectional analysis V_{Ed0} may be reduced by any favourable contributions resulting from any inclined tension chords (V_{Etd}), compression chords (V_{Ecd}) and prestressing tendons (V_{Epd}), see Figure 7.3-8. In determining V_{Epd}, an eventual reduction in prestress due to the development length shall be considered.

Any unfavourable contributions from inclined chord and prestressing tendon forces must be added to V_{Ed0}.

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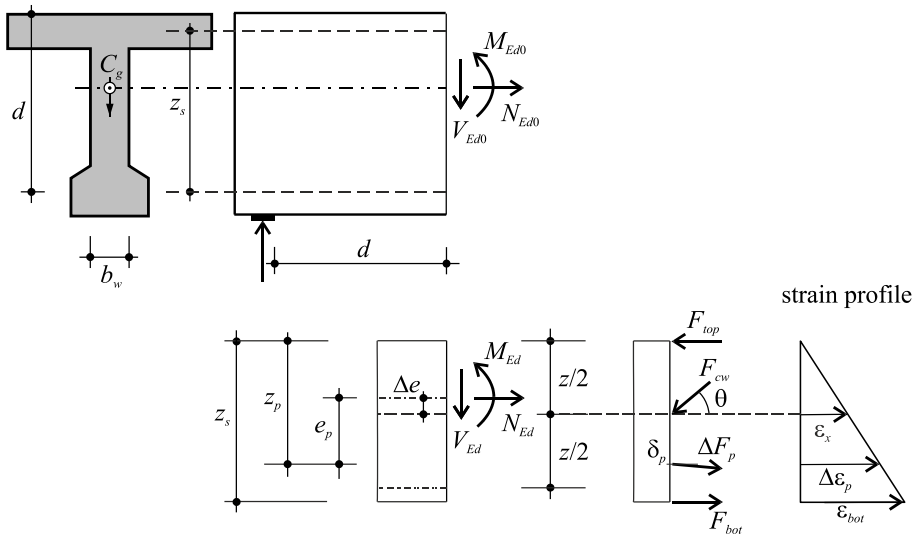


Figure 7.3-9: Definitions

For prestressed members, Eq. (7.3-16) is replaced by:

$$\varepsilon_x = \frac{\left(\frac{M_{Ed}}{z} + V_{Ed} + N_{Ed} \frac{\left(z_p - e_p\right)}{z} \right)}{2\left(\frac{z_s}{z} E_s A_s + \frac{z_p}{z} E_p A_p \right)} \qquad (7.3\text{-}14)$$

If the value of ε_x is negative it shall be taken as zero.

For prestressed members the sectional forces are taken as:

$$\begin{aligned} M_{Ed} &= M_{Ed0} + M_{Pd} \\ N_{Ed} &= N_{Ed0} - F_p \cos\delta_p \\ V_{Ed} &= V_{Ed0} - F_p \sin\delta_p \end{aligned} \qquad (7.3\text{-}15)$$

In the design for shear in beams and in slabs, the effects of axial tension due to creep, shrinkage and thermal effects in restrained members should be considered wherever appropriate.

Design and analysis of members in shear may require the state of strain to be taken into account. Within the framework of undertaking a cross-sectional analysis, the longitudinal strain (Figure 7.3-9) is calculated at the mid-depth of the effective shear depth or core layer being considered as follows:

$$\varepsilon_x = \frac{1}{2\,E_s A_s}\left(\frac{M_{Ed}}{z} + V_{Ed} + N_{Ed}\left(\frac{1}{2} \mp \frac{\Delta e}{z} \right) \right) \qquad (7.3\text{-}16)$$

In the use of Eq. (7.3-16), the following conditions apply:

- M_{Ed} and V_{Ed} shall be taken as positive quantities and N_{Ed} as positive for tension and negative for compression.
- It is permissible to use a value of ε_x that is greater than half the yield strain of the longitudinal bars ($\varepsilon_{sy}/2$) but a more detailed cross-sectional analysis shall be undertaken. The strain ε_x shall not exceed 0.003.
- If the value of ε_x is negative it shall be taken as zero.
- For sections closer than d to the face of the support, the value of ε_x taken at d from the face of the support may be used.
- For sections within a distance $z/2$ of a significant bar curtailment, the calculated value ε_x shall be increased by a factor of 1.5.
- A_s comprises the main longitudinal reinforcing bars in the tensile chord; any distributed longitudinal reinforcement is neglected.
- In calculating A_s (and A_p) the area of the bars that are terminated less than their development length from the section under consideration shall be reduced in proportion to their lack of full development.
- If the axial tension is large enough to crack the flexural compression face of the section, the calculated value of ε_x shall be multiplied by a factor of 2.0.

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where M_{Pd} denotes the design bending moment due to prestressing which includes a possible moment $M_{P,ind}$ resulting from static indeterminacy, i.e. $M_{Pd} = \pm F_p \cos\delta_p(e_p) + M_{P,ind}$.

In case of a support that penetrates into the beam or slab, z is replaced with the effective depth, d_v in accordance with subclause 7.3.5.2.

The Level I equation is derived from the more general Level II Approximation with the assumption that the mid-depth strain at the control section can be taken as $\varepsilon_x = 0.00125$, which corresponds to half the yield strain for a reinforcing bar with $f_{yk} = 500$ MPa ($\varepsilon_x \approx f_{yk}/(2E_s)$).

7.3.3.2 Members without shear reinforcement

General

The design shear resistance of a web or a slab without shear reinforcement is given by:

$$V_{Rd,c} = k_v \frac{\sqrt{f_{ck}}}{\gamma_c} z b_w \qquad (f_{ck} \text{ in MPa}) \qquad (7.3\text{-}17)$$

where the value of $\sqrt{f_{ck}}$ shall not be taken as greater than 8 MPa.

The longitudinal reinforcement in the flexural tensile chord at each section of interest must be able to resist an additional force component due to the shear of:

$$\Delta F_{td} = V_{Ed} \qquad (7.3\text{-}18)$$

However, the total demand on longitudinal reinforcement needs not to exceed the demand due to maximum moment alone in the respective maximum moment region.

Level I Approximation

For members with no significant axial load, with $f_{yk} \leq 600$ MPa, $f_{ck} \leq 70$ MPa and with a maximum aggregate size of not less than 10 mm:

$$k_v = \frac{180}{1000 + 1.25z} \qquad (z \text{ in mm}) \qquad (7.3\text{-}19)$$

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In higher strength concrete and light-weight aggregate concretes, the fracture surface may go through the aggregate particles, rather than around, reducing the crack roughness.

There is evidence that the shear resistance of members without shear reinforcement is influenced by the maximum size of the aggregate d_g. If concrete with a maximum size of the aggregate different from d_g = 16 mm is used the value k_{dg} may be calculated with

$$k_{dg} = \frac{32}{16 + d_g} \geq 0.75 \qquad (7.3\text{-}20)$$

For concrete strengths in excess of 70 MPa and for light-weight concrete, d_g in Eq. (7.3-20) shall be taken as zero, in order to account for the loss of aggregate interlock in the cracks due to fracture of aggregate particles.

The web reinforcement ratio given by Eq. (7.3-22) corresponds to the minimum reinforcement ratio as defined in subclause 7.13.5. Members containing a lower reinforcement ratio are to be treated according to clause 7.3.3.2.

Size effects are limited in members with web reinforcement greater than that required by Eq. (7.3-22).

The strength reduction factor k_c consists of two parts: the state of strain in the webs of beams or the core layers of slabs is taken into account by k_ε; the effect of more brittle failure behaviour of concrete of strengths greater than 30 MPa is considered in η_{fc}.

Level II Approximation

For the Level II Approximation, the design shear resistance is determined with:

$$k_v = \frac{0.4}{1 + 1500\varepsilon_x} \cdot \frac{1300}{1000 + k_{dg} z} \qquad (z \text{ in mm}) \qquad (7.3\text{-}21)$$

Provided that the size of the maximum aggregate particles, d_g, is not less than 16 mm, k_{dg} in Eq. (7.3-21) can be taken as k_{dg} = 1.0.

7.3.3.3 Members with shear reinforcement

General

This subclause applies to members that meet the demand for minimum shear reinforcement according to:

$$\rho_w \geq 0.08 \frac{\sqrt{f_{ck}}}{f_{yk}} \qquad (f_{ck} \text{ and } f_{yk} \text{ in MPa}) \qquad (7.3\text{-}22)$$

The design shear resistance is then determined from:

$$V_{Rd} = V_{Rd,c} + V_{Rd,s} \qquad (7.3\text{-}23)$$

but shall not be taken as greater than:

$$V_{Rd,\max} = k_c \frac{f_{ck}}{\gamma_c} b_w z \sin\theta \cos\theta \qquad (7.3\text{-}26)$$

where θ denotes the inclination of the compressive stress field.

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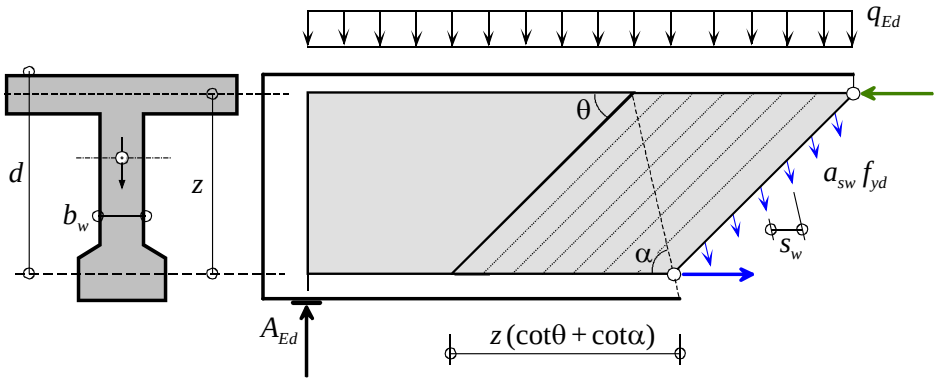


Figure 7.3-10: Geometry and definitions

In the case of stirrups that are inclined relative to the beam axis the Eqs. (7.3-26) and (7.3-29) are replaced by:

$$V_{Rd,\max} = k_c \frac{f_{ck}}{\gamma_c} b_w z \frac{\cot\theta + \cot\alpha}{1+\cot^2\theta} \qquad (7.3\text{-}24)$$

and

$$V_{Rd,s} = \frac{A_{sw}}{s_w} z f_{ywd} \left(\cot\theta + \cot\alpha\right) \sin\alpha \qquad (7.3\text{-}25)$$

where α is the inclination of the stirrups as shown in Figure 7.3-10.

The limitation on $\sqrt{f_{ck}}$ is provided due to the larger observed variability in shear strength of members with higher strength concrete, particularly for members without stirrups such as slabs.

The strength reduction factor is defined as:

$$k_c = k_\varepsilon \eta_{fc} \qquad (7.3\text{-}27)$$

with k_ε as given in the following and:

$$\eta_{fc} = \left(\frac{30}{f_{ck}}\right)^{1/3} \le 1.0 \qquad (f_{ck} \text{ in MPa}) \qquad (7.3\text{-}28)$$

The design shear resistance provided by stirrups is:

$$V_{Rd,s} = \frac{A_{sw}}{s_w} z f_{ywd} \cot\theta \qquad (7.3\text{-}29)$$

where f_{ywd} denotes the design yield strength of the shear reinforcement.

The design shear resistance attributed to the concrete can be taken as:

$$V_{Rd,c} = k_v \frac{\sqrt{f_{ck}}}{\gamma_c} b_w z \qquad (7.3\text{-}30)$$

where the value of $\sqrt{f_{ck}}$ shall not be taken as greater than 8 MPa.

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Values of k_D depend on the material of the duct and whether it is grouted or not. Suggested values for design are:

– grouted steel duct: $k_D = 0.5$;

– grouted plastic duct: $k_D = 0.8$;

– ungrouted duct: $k_D = 1.2$.

Factor k_D may be reduced in presence of reinforcement transverse to the plane of the web.

In the case of stirrups that are inclined relative to the beam axis the Eq. (7.3-34) is be replaced by:

$$\Delta F_{td} = \frac{V_{Ed}}{2}\left(\cot\theta - \cot\alpha\right) \tag{7.3-32}$$

In the level III approach, where a concrete contribution $V_{Rd,c} \neq 0$ is considered, in the Eqs. (7.3-32) and (7.3-34) the design shear force V_{Ed} is replaced by: V_{Ed}^{*} where:

$$V_{Ed}^{*} = V_{Ed} + V_{Rd,c} \tag{7.3-33}$$

The value of θ_{min} is determined by the Level of Approximation.

The Level I Approximation represents a variable angle truss model approach.

In Eq. (7.3-30), the effective web width b_w shall be taken as the minimum concrete web width within the effective shear depth z.

In the case of prestressing tendons with duct diameters $\varnothing_D \geq b_w/8$, the ultimate resistance of the compression struts shall be calculated on the basis of the nominal value of the web width:

$$b_{w,nom} = b_w - k_D \sum \varnothing_D \tag{7.3-31}$$

where $\Sigma\varnothing_D$ shall be determined for the most unfavourable prestressing tendon configuration.

The longitudinal reinforcement in the flexural tensile chord at every section of interest shall be designed to resist the additional force due to shear of:

$$\Delta F_{td} = \frac{V_{Ed}}{2}\cot\theta \tag{7.3-34}$$

However, the total demand on longitudinal reinforcement need not exceed the demand at the maximum moment location due to moment alone.

The limits of the compressive stress field inclination θ, relative to the longitudinal axis of the member (Figure 7.3-10), are:

$$\theta_{min} \leq \theta \leq 45° \tag{7.3-35}$$

where θ is the inclination of the compressive stress field and may be chosen freely between these limits.

Level I Approximation

In the level I approach, the design shear resistance is given by:

$$V_{Rd} = V_{Rd,s} \leq V_{Rd,max} \tag{7.3-36}$$

but needs not to be taken less than the resistance of the same member without shear reinforcement.

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The Level II Approximation is based on a generalized stress field approach.

The stress field approach allows the strut inclination θ to be selected within certain limits and is confirmed by experimental observations.

For a preliminary design or analysis, ε_x maybe taken as 0.001.

The variable value k_ε considers the influence of the state of strain in the web. This influence is important for prestressed members or members in compression, but less significant for reinforced elements and/or sections with higher θ and ε_x values.

The minimum inclination of the compressive stress field is:

$\theta_{min} = 25°$ for members with significant axial compression or prestress;
$\theta_{min} = 30°$ for reinforced concrete members;
$\theta_{min} = 40°$ for members with significant axial tension.

The width of the beam or web shall be checked for the selected inclinations of the compression stresses where k_ε is taken as:

$$k_\varepsilon = 0.55 \qquad (7.3\text{-}37)$$

Eqs. (7.3-36) and (7.3-37) apply to cross-sections where the longitudinal strain ε_x remains below a value of 0.001.

Level II Approximation

In the Level II approach, the design shear resistance is given by:

$$V_{Rd} = V_{Rd,s} \le V_{Rd,\max} \qquad (7.3\text{-}38)$$

but needs not to be taken less than the resistance of the same member without shear reinforcement.

Within the limits of Eq. (7.3-35) the inclination of the compressive stress field can be freely selected for design, or analytically determined for assessment.

The minimum inclination of the compressive stress field shall be taken as:

$$\theta_{\min} = 20° + 10000\varepsilon_x \qquad (7.3\text{-}39)$$

where ε_x represents the longitudinal strain at the mid-depth of the effective shear depth as shown in Figure 7.3-9.

The design shear resistance attributed to the concrete shall be neglected, i.e. $k_v = 0$. The width of the beam or web shall be checked for the respective inclination of the compression stresses where k_ε is taken as:

$$k_\varepsilon = \frac{1}{1.2 + 55\varepsilon_1} \le 0.65 \qquad (7.3\text{-}40)$$

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Level III Approximation represents a general form of sectional shear equations and is based on the simplified modified compression field theory.

A comparison of the relative predictions for modelling levels I to III is shown in Figure 7.3-11.

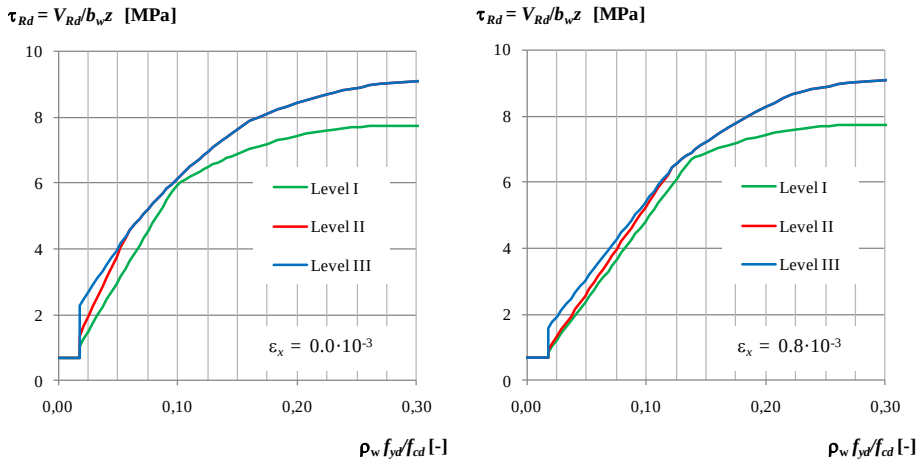


Figure 7.3-11: Comparison of level I, II and III results for members with f_{ck} = 50 MPa (Note: for the curves shown in the figure the value f_{cd} is defined as $\eta_{fc} f_{ck}/\gamma_c$ and calculated for $\theta = \theta_{min}$)

The use of tools based on advanced methods of analysis often requires extensive experience to ensure that safe and consistent results are obtained.

where:

$$\varepsilon_1 = \varepsilon_x + (\varepsilon_x + 0.002)\cot^2\theta \qquad (7.3\text{-}41)$$

The longitudinal strain ε_x at the mid-depth of the effective shear depth shall be calculated on the basis of Eq. (7.3-14) or (7.3-16).

Level III Approximation

In the Level III approach, the design shear resistance in the range of $V_{Rd} < V_{Rd,max}(\theta_{min})$ is given by:

$$V_{Rd} = V_{Rd,s} + V_{Rd,c} \qquad (7.3\text{-}42)$$

where $V_{Rd,max}(\theta_{min})$ is calculated from Eq. (7.3-26) for $\theta = \theta_{min}$.

In the range of $V_{Rd} \geq V_{Rd,max}(\theta_{min})$ the resistance is determined as in the Level II Approximation.

The inclination θ_{min} is taken as given by Eq. (7.3-39).

For determining the design shear resistance $V_{Rd,c}$ attributed to the concrete the following expression shall be used:

$$k_v = \frac{0.4}{1+1500\varepsilon_x}\left(1 - \frac{V_{Ed}}{V_{Rd,\max}(\theta_{\min})}\right) \geq 0 \qquad (7.3\text{-}43)$$

The strain ε_x at the mid-depth of the effective shear depth is calculated with Eq. (7.3-14) or (7.3-16).

Level IV Approximation

The resistance of members in shear or shear combined with torsion may be determined by satisfying the applicable conditions of equilibrium and compatibility of strains and by using appropriate stress-strain models for the steel and for diagonally cracked concrete.

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For hollow core slabs and similar structural members the design shear resistance may be calculated on the basis of this subclause or alternatively of subclause 7.3.3.3; the higher of the results may be adopted as the capacity.

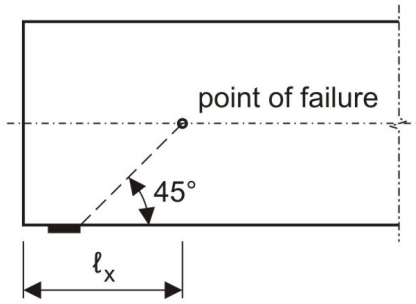


Figure 7.3-12: Basis for derivation Eq. (7.3-44)

7.3.3.4 Hollow core slabs

In single span prestressed hollow core slabs without shear reinforcement, shear failure occurs when the principal tension stress demand in the web exceeds the tensile strength of the concrete.

Level I Approximation

The design shear resistance can be approximated by:

$$V_{Rd,ct} = 0.8 \frac{I_c \cdot b_w}{S_c} \sqrt{f_{ctd}^2 + \alpha_l \cdot \sigma_{cp} \cdot f_{ctd}} \qquad (7.3\text{-}44)$$

where:

I_c is second moment of area;

S_c is first moment of area above and about the centroidal axis;

b_w is width of the cross-section at the centroidal axis;

σ_{cp} is concrete compressive stress at the centroidal axis due to prestressing, in the area where the prestressing force is fully introduced;

$\alpha_l = l_x/(1.2 \cdot l_{bd,0})$;

l_x follows from Figure 7.3-12;

$l_{bd,0}$ follows from Eq. (7.13-5).

Level II Approximation

In a Level II Approximation the design shear resistance is determined by:

$$V_{Rd,ct} = \frac{I_c \cdot b_w(y)}{S_c(y)} [\sqrt{f_{ctd}^2 + \alpha_l \cdot \sigma_{cp}(y) \cdot f_{ctd}} - \tau_{cp}(y)] \qquad (7.3\text{-}45)$$

where:

I_c is second moment of area;

$S_c(y)$ is first moment of area above height y and about the centroidal axis;

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$b_w(y)$ is width of the cross-section at the height y;

y is the height of the critical point at the line of failure;

$\sigma_{cp}(y)$ is concrete compressive stress at height y and distance l_x;

$\tau_{cp}(y)$ is the shear stress in the concrete due to transmission of prestress at height y and distance l_x.

The concrete compressive stress at height y and distance l_x is determined from:

$$\sigma_{cp}(y) = \left[\frac{1}{A_c} + \frac{y_c - y}{I} \right] \cdot F_p(l_x) \qquad (7.3\text{-}46)$$

and the shear stress in the concrete due to transmission of prestress:

$$\tau_{cp}(y) = \frac{1}{b_w(y)} \cdot \left[\frac{A_c(y)}{A_c} - \frac{S_c(y) \cdot (y_c - y_{pt})}{I} \right] \cdot \frac{dF_p(l_x)}{dx} \qquad (7.3\text{-}47)$$

where:

y_c is height of concrete centroidal axis;

A_c is area of concrete cross-section;

$A_c(y)$ is concrete area above height y;

y_{pt} is height of centroidal axis of prestressing steel;

$F_p(l_x)$ is the prestressing force at distance l_x.

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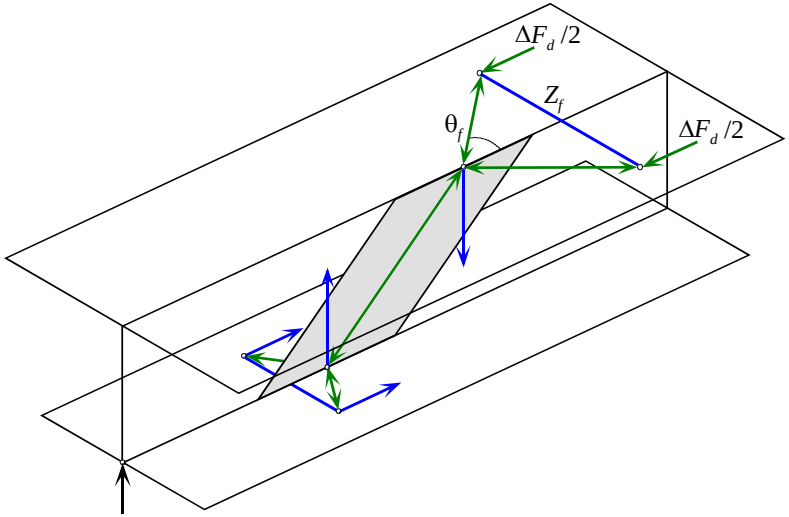


Figure 7.3-13: Strut-and-tie model for force introduction into the flanges

7.3.3.5 Shear between web and flanges of T-sections

The introduction of tensile or compressive forces into the flanges (Figure 7.3-13) creates shear forces at the transition of web and flanges, inducing corresponding transverse tensile and compressive forces in the flanges.

The spread of the compressive forces in the flanges shall be examined with the aid of stress fields. Recommended values for the angle of spread are:

$25° \le \theta_f \le 45°$ for compressive flanges; and

$35° \le \theta_f \le 50°$ for tensile flanges.

Unless a more detailed analysis is undertaken, reinforcement for the introduction of forces into the flanges shall be superimposed on that required for transverse bending.

In addition, the minimum transverse reinforcement shall not be less that that required by subclause 7.13.5.

The longitudinal flange reinforcement shall be anchored in accordance with the assumed stress field requirements.

7.3.3.6 Shear at the interface between concrete cast at different times

In addition to the requirements formulated in the subclauses 7.3.3.1 to 7.3.3.5 the shear stress at the interface between concrete cast at different times should also satisfy the following condition:

$$\tau_{Edi} \le \tau_{Rdi} \qquad (7.3\text{-}48)$$

where τ_{Edi} is the design value of the shear stress in the interface, given by

$$\tau_{Edi} = \beta \cdot V_{Ed} / (z b_i) \qquad (7.3\text{-}49)$$

where:

β is the ratio of the longitudinal force in the new concrete and the total longitudinal force either in the compression or tension zone, both calculated for the section considered;

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If a "rigid" bond-slip behaviour is expected and very good adhesive bonding guaranteed on the site, the adhesive bond effect shall be taken into account without superimposing effects of interface reinforcement. The most important precondition for the assumption of good adhesive bond is a well prepared and very clean concrete surface at the time of casting.

Adhesive bond resistance shall only be applied for design if no tensile loading perpendicular to the interface is expected.

The adhesion coefficients c_a actually depend on a variety of influencing parameters (see subclause 6.3.3); nevertheless the c_a factors given in Table 7.3-1 represent reasonable values on the safe side for the given roughness categories.

Special attention shall be put on edge zones, see detailing rules in clause 6.3.

For the definition of roughness of the classes distinguished in Table 7.3-1 reference is made to subclause 6.3.2.

z is the inner lever arm of the composed section;

b_i is the width of the interface and V_{Ed} is the shear force on the composed section.

Interface without reinforcement (rigid bond-slip behaviour)

The design limit value τ_{Rdi} for the interface shear in Eq. (7.3-31) follows from:

$$\tau_{Rdi} = c_a \cdot f_{ctd} + \mu \cdot \sigma_n \le 0.5 \cdot \nu \cdot f_{cd} \qquad (7.3\text{-}50)$$

where:

c_a is the coefficient for the adhesive bond;

μ is the friction coefficient acc. Table 7.3-2;

σ_n is the (lowest expected) compressive stress resulting from an eventual normal force acting on the interface.

The adhesion factor c_a depends on the roughness of the interface (see Table 7.3-1; R_t is derived from the sand patch method).

Table 7.3-1: Coefficients for the adhesive bond resistance

Surface characteristics of interface	c_a
Very rough (including shear keys) $R_t \ge 3.0$ mm	0.5
Rough (strongly roughened surface) $R_t \ge 1.5$ mm	0.40
Smooth (concrete surface without treatment after vibration or slightly roughened when cast against formwork)	0.20
Very smooth (steel, plastic, timber formwork)	0.025

Under fatigue or dynamic loads the values for c_a as found in Table 7.3-1 have to be reduced to 50%.

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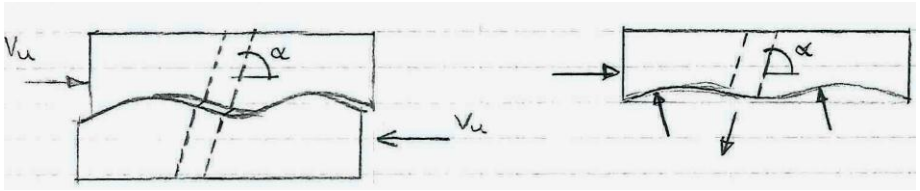


Figure 7.3-14: Transmission of shear forces across an interface intersected by reinforcing bars

Eq. (7.3-51) relates to interfaces intersected by dowels or reinforcement and characterized by a rather non-rigid bond-slip behaviour.

Connectors may be omitted in interface regions where the design shear stress does not exceed the resistance given in Eq. (7.3-50).

It is worthy of note that for a bar in tension, as explained in clause 6.3, the tensile strength of the bar is reduced when the bar is also subjected to dowel action, as shown in Figure 7.3-15.

The detailing rules given in clause 6.3 concerning embedment depth of connectors and minimum amount of steel cross-section shall be obeyed.

Interface intersected by dowels or reinforcement

If strong adhesive bond cannot be guaranteed on the site or the design shear resistance provided by adhesive bond acc. Eq. (7.3-50) is lower than the design shear stress, interface connectors are required and the design limit value τ_{Rdi} follows from:

$$\tau_{Rdi} = c_r \cdot f_{ck}^{1/3} + \mu \cdot \sigma_n + \kappa_1 \cdot \rho \cdot f_{yd} \cdot (\mu \cdot \sin\alpha + \cos\alpha) + \kappa_2 \cdot \rho \cdot \sqrt{f_{yd} \cdot f_{cd}} \leq \beta_c \cdot \nu \cdot f_{cd} \qquad (7.3\text{-}51)$$

where strength values are in N/mm^2 and,

c_r is the coefficient for aggregate interlock effects at rough interfaces;

κ_1 is the interaction coefficient for tensile force activated in the reinforcement or the dowels;

κ_2 is the interaction coefficient for flexural resistance;

μ is the friction coefficient;

ρ is the reinforcement ratio of the reinforcing steel crossing the interface;

σ_n is the (lowest expected) compressive stress resulting from an eventual normal force acting on the interface;

α is the inclination of the reinforcement crossing the interface (see Fig. 7.3-14);

β_c is the coefficient for the strength of the compression strut;

$$\nu = 0.55\left(\frac{30}{f_{ck}}\right)^{1/3} < 0.55\,.$$

The coefficients for different surface roughness in interfaces reinforced with dowels or rebars are given in Table 7.3-2.

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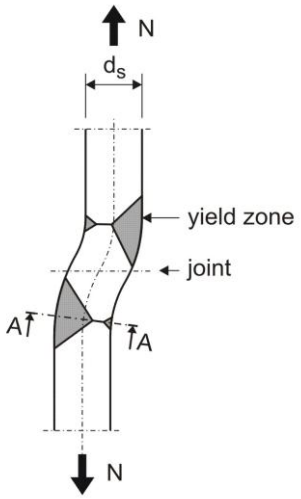


Figure 7.3-15: Dowel action under simultaneous tension and shear

For the background of the values in Table 7.3-2, reference is made to Randl, Münger, Wicke "Strengthening bridges by a structural top layer" (Der Bauingenieur 2005, pp. 207-214, in German).

The roughness of a concrete surface can be measured in various ways (see subclause 6.3.2). An appropriate way is the *sand patch method* as depicted in Figure 7.3-16 (*Kaufmann*, 1971). A volume of sand V is spread on the rough surface in a circular area with diameter d. The roughness parameter R_t follows from:

$$R_t[mm] = \frac{40 \cdot V}{\pi d^2} \tag{7.3-52}$$

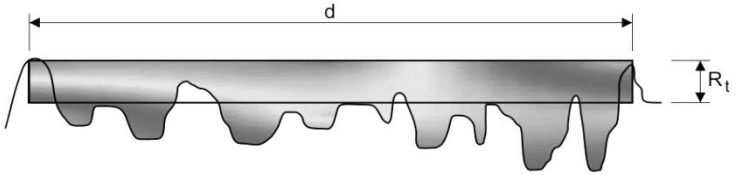


Figure 7.3-16: Principle of sand patch method for the qualification of the roughness of an interface

Table 7.3-2: Coefficients for different surface roughness

Surface Roughness	c_r	κ_1	κ_2	β_c	μ	
					$f_{ck} \geq 20$	$f_{ck} \geq 35$
Very rough* $R_t \geq 3.0$ mm	0.2	0.5	0.9	0.5	0.8	1.0
Rough $R_t \geq 1.5$ mm	0.1	0.5	0.9	0.5	0.7	
Smooth	0	0.5	1.1	0.4	0.6	
Very smooth	0	0	1.5	0.3	0.5	

* valid also for shear keys

Under fatigue or dynamic loads the values for τ_{Rdi} according to Eq (7.3-51) have to be reduced to 40%.

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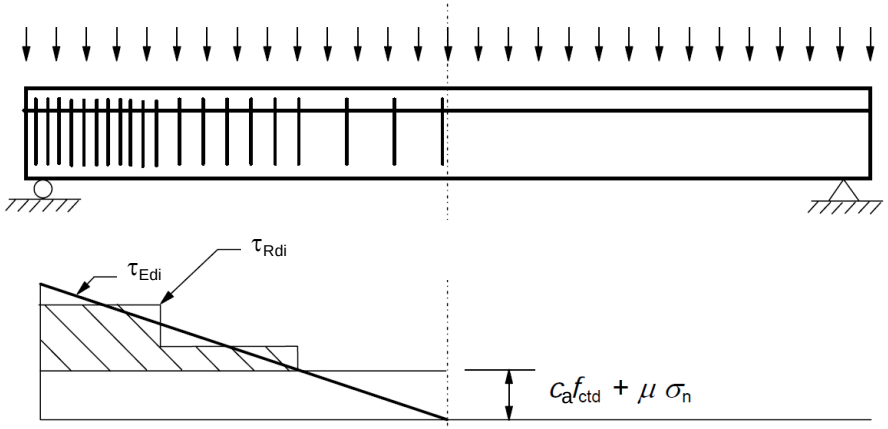


Figure 7.3-17: Shear diagram representing the required interface reinforcement

A stepped distribution of the transverse reinforcement may be used, as indicated in Figure 7.3-17.

7.3.4 Torsion

Where static equilibrium depends on the torsional resistance of elements of the structure, a full torsional design for both the ultimate and serviceability limit states shall be undertaken.

Where in structures torsion arises from consideration of compatibility only, and the structure is not dependent on torsional resistance for its stability, it will normally not be necessary to consider torsion at the ultimate limit state. In such cases minimum torsional reinforcement shall be provided in the form of stirrups and longitudinal bars should be provided to prevent excessive cracking, as per the requirements of subclause 7.13.5.2.

The determination of the torsional resistance of box-girders and beams of solid cross-section is based on an ideal hollow cross-section as shown in Figure 7.3-18.

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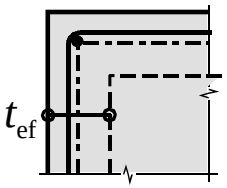


Figure 7.3-19: Minimum effective panel thickness

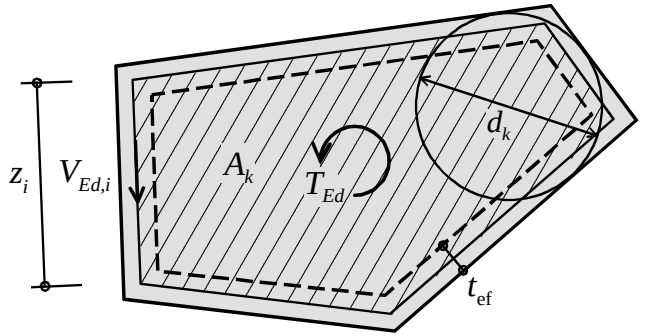


Figure 7.3-18: Definition of the ideal hollow cross-section

If the internal forces and moments, the cross-sectional dimensions and the reinforcement do not change abruptly in the longitudinal direction, it may be assumed that the shear flow due to torsion is constant over the circumference of the effective cross-sectional area. The torsional moment may then be resolved into equivalent panel forces such that:

$$V_{Ed,Ti} = \frac{T_{Ed}}{2A_k} z_i \tag{7.3-53}$$

The provisions of subclause 7.3.3 apply analogously for the dimensioning of the reinforcement and checking of the panel dimensions. The effective panel thickness of solid cross-sections (Figure 7.3-19) can thereby be taken into account as:

$$t_{ef} \le \frac{d_k}{8} \tag{7.3-54}$$

where d_k denotes the diameter of the circle that might be inscribed at the most narrow part of the cross-section.

A minimum effective panel thickness $t_{ef,\min}$ of twice the distance between the concrete surface and the centre of the longitudinal reinforcement may be considered.

In the case of box-girders, the effective panel thickness corresponds to the wall thickness, if the wall is reinforced on all sides.

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In the case of combined action of torsion, bending and shear force in a solid section, the core within the idealized hollow cross-section may be used for the transmission of the shear forces.

Punching failures may develop with limited deformations (brittle behaviour). Therefore, the effects of imposed deformations (temperature, creep and shrinkage, settlements, etc.) should be taken into account in design. The influence of imposed deformations can however be neglected if sufficient deformation capacity is provided. Strategies for increasing the deformation capacity are:

– choice of a sufficiently large supported area and depth of slab in combination with low bending reinforcement ratios (rules are given in subclause 7.3.5.3);

The longitudinal reinforcement due to torsion shall either be distributed evenly over the length of the panels or concentrated at the corners.

In the case of combined action of torsion, bending and shear force, the internal forces and moments are replaced by a statically equivalent set of normal and shear forces. The reinforcement is then determined according to the provisions of subclauses 7.3.2 and 7.3.3.

For box-girder sections the maximum resistance of a panel is given by $V_{Rd,c}$ for members without and $V_{Rd,max}$ for members with shear reinforcement, respectively.

For other sections (such as rectangular cross-sections) the maximum resistance has to be checked by

$$\left(\frac{T_{Ed}}{T_{Rd,\max}}\right)^2+\left(\frac{V_{Ed}}{V_{Rd,\max}}\right)^2\le 1 \qquad (7.3\text{-}55)$$

where $T_{Rd,max}$ is calculated as:

$$T_{Rd,\max}=k_c\frac{f_{ck}}{\gamma_c}t_{ef}\,2A_k\sin\theta\cos\theta \qquad (7.3\text{-}56)$$

and the definitions are given in subclauses 7.3.2 and 7.3.3.

7.3.5 Punching

7.3.5.1 General

Punching can result from a concentrated load applied on a relatively small area of the structure. In flat slabs, punching shear failures normally develop around supported areas (columns, capitals, walls). In other cases (as for instance foundation slabs, transfer slabs, deck slabs of bridges) punching failures can also develop around loaded areas. The rules presented hereafter for flat slabs apply by analogy to loaded areas.

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– use of punching shear reinforcement (rules are given in subclause 7.3.5.3).

With flat slabs, safety against punching is particularly significant as the failure of one column can propagate to adjacent columns leading to a complete collapse of a structure. To avoid such progressive collapses, one of the following strategies should be adopted:

– increase of the deformation capacity of the potential failure zones (see above) to allow internal forces to redistribute; and/or

– arrange integrity reinforcement for slabs with limited deformation capacity (rules are given in subclause 7.3.5.6).

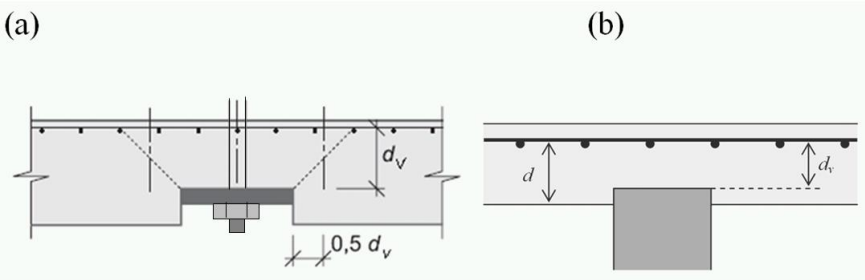


Figure 7.3-20: Effective depth of the slab considering support penetration (dv) and effective depth for bending calculations (d)

7.3.5.2 Design shear force, shear-resisting effective depth and control perimeter

(1) Design shear force

The design shear force with respect to punching (V_{Ed}) is calculated as the sum of design shear forces acting on a basic control perimeter (b_1).

(2) Shear-resisting effective depth

The shear-resisting effective depth of the slab (d_v) is the distance from the centroid of the reinforcement layers to the supported area (Figure 7.3-20).

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For flat slabs and footings, the design shear force is equal to the value of the support reaction reduced by the actions applied inside the basic control perimeter (such as gravity loads, soil pressure at footings and deviation forces of prestressing cables).

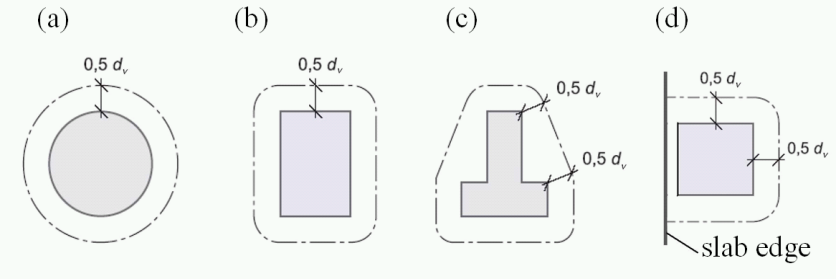


Figure 7.3-21: Basic control perimeters around supported areas

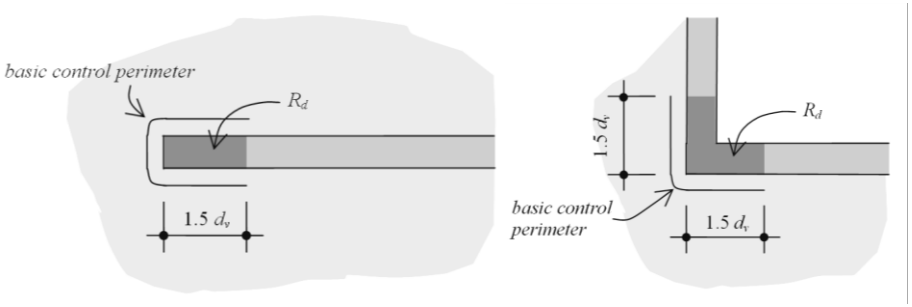


Figure 7.3-22: Basic control perimeter around walls

(3) Basic control perimeter (b_1)

The basic control perimeter b_1 may normally be taken at a distance $0.5d_v$ from the supported area (Figure 7.3-21 and Figure 7.3-22) and should be determined in order to minimize its length (Figure 7.3-21c). The length of the control perimeter is limited by slab edges (Figure 7.3-21d).

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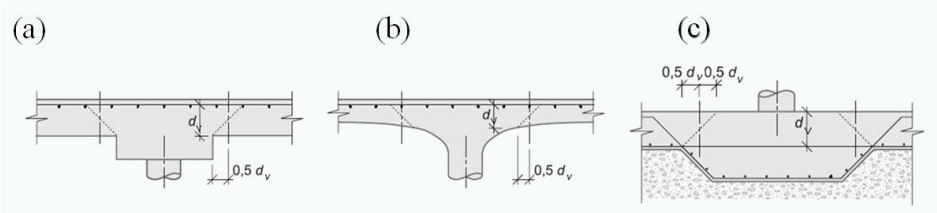


Figure 7.3-23: Choice of the potentially governing control perimeter

The shear-resisting control perimeter b_0 can be obtained on the basis of a detailed shear field analysis as:

$$b_0 = \frac{V_{Ed}}{v_{perp,d,max}} \qquad (7.3\text{-}57)$$

where $v_{perp,d,\max}$ is the maximum shear force per unit length perpendicular to the basic control perimeter (Figure 7.3-24).

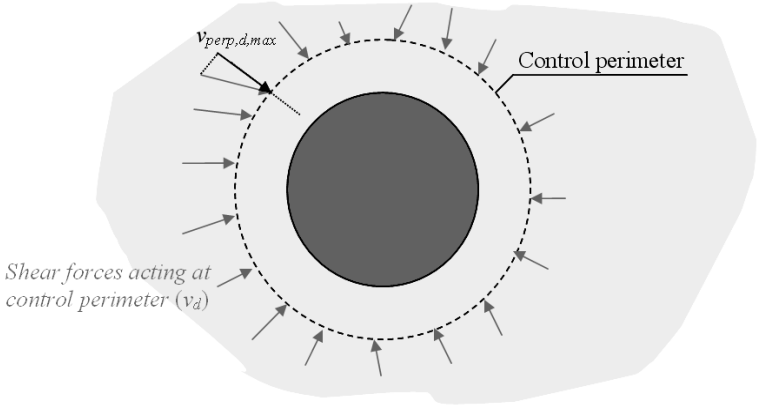


Figure 7.3-24: Shear force per unit length (vd) and maximum value perpendicular to the basic control perimeter

In the case of slabs with variable depth control sections at a greater distance from the supported area may be governing (refer to Figure 7.3-23).

(4) Shear-resisting control perimeter (b_0)

For calculating the punching shear resistance, a shear-resisting control perimeter (b_0) is used. The shear-resisting control perimeter accounts for the non-uniform distribution of shear forces along the basic control perimeter.

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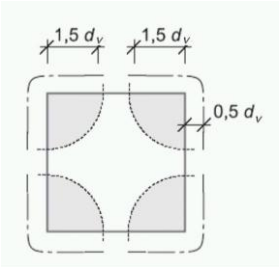
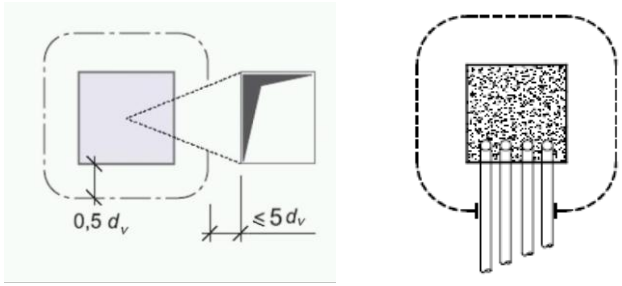


Figure 7.3-25: Reduction of basic control perimeter for large supported areas ($b_{1,red}$)



(a) (b)

Figure 7.3-26: Reduction of basic control perimeter ($b_{1,red}$) in presence of: (a) openings; and (b) pipes or inserts

Cast-in pipes, pipe bundles or slab inserts, where the distance from the supported area is less than $5d_v$ shall be arranged perpendicular to the control perimeter. In these cases, the control perimeter should be reduced in accordance to Figure 7.3-26.

The coefficient of eccentricity can be determined as a function of the moment transferred from the column to the slab as:

$$k_e = \frac{1}{1+e_u/b_u} \tag{7.3-59}$$

where e_u is the eccentricity of the resultant of shear forces with respect to the centroid of the basic control perimeter, see Figure 7.3-27b, and b_u is the diameter of a circle with the same surface as the region inside the basic

A non-uniform distribution of the shear forces may result due to:

1. *Concentrations of the shear forces at the corners of large supported areas.* This effect can approximately be taken into account by reducing the basic control perimeter ($b_{1,red}$) assuming that the length of its straight segments does not exceed $3d_v$ for each edge (Figure 7.3-25).
2. *Geometrical and static discontinuities of the slab.* In presence of openings and inserts, the basic control perimeter ($b_{1,red}$) is to be reduced according to the rules of Figure 7.3-26.
3. *Concentrations of the shear forces due to moment transfer between the slab and the supported area*. This effect can approximately be taken into account by multiplying the length of the reduced basic control perimeter ($b_{1,red}$) by the coefficient of eccentricity (k_e):

$$b_0 = k_e \cdot b_{1,red} \tag{7.3-58}$$

4. *Presence of significant loads near the supported area*. In cases where significant concentrated loads ($\geq 0.2V_{Ed}$) are applied near the supported area (closer than $3d_v$ from the edge of the supported area) the general procedure for calculating b_0 should be used, refer to Eq. (7.3-57).

In cases where the lateral stability does not depend on frame action of slabs and columns and where the adjacent spans do not differ in length by more than 25%, the following approximated values may be adopted for the coefficient k_e:

- 0.90 for inner columns;
- 0.70 for edge columns;
- 0.65 for corner columns;
- 0.75 for corners of walls (horizontal shear resisting members where the rules of Figure 7.3-22 apply).

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control perimeter. For design purposes, the position of the centroid of the basic control perimeter can be calculated by approximating its shape with straight lines, see Figure 7.3-27b.

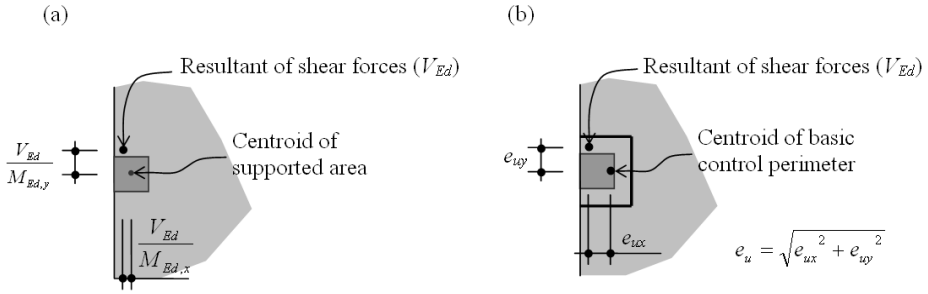


Figure 7.3-27: Resultant of shear forces: (a) position with respect to the centroid of the supported area; and (b) approximated basic control perimeter for calculation of the position of its centroid and eccentricity between the resultant of shear forces and the centroid of the basic control perimeter

The calculation of the punching shear strength is based on the critical shear crack theory.

7.3.5.3 Punching shear strength

The punching shear resistance is calculated as:

$$V_{Rd} = V_{Rd,c} + V_{Rd,s} \geq V_{Ed} \tag{7.3-60}$$

The design shear resistance attributed to the concrete may be taken as:

$$V_{Rd,c} = k_{\psi} \frac{\sqrt{f_{ck}}}{\gamma_c} b_0 \, d_v \tag{7.3-61}$$

with f_{ck} in [MPa].

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The parameter ψ refers to the rotation of the slab around the supported area (Figure 7.3-28).

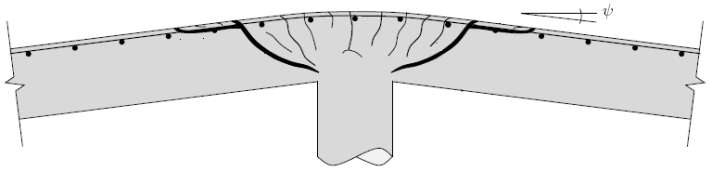


Figure 7.3-28: Rotation (ψ) of a slab

There is evidence that the punching shear resistance is influenced by the maximum size of the aggregate (d_g). If concrete with a maximum aggregate size smaller than d_g = 16 mm is used, the value of k_{dg} in Eq. (7.3-63) is assessed as:

$$k_{dg} = \frac{32}{16 + d_g} \geq 0.75 \qquad (7.3\text{-}62)$$

where d_g is in mm. For aggregate sizes larger than 16 mm, Eq. (7.3-63) may also be used. For high strength and lightweight concrete the aggregate particles may break, resulting in a reduced aggregate interlock contribution. In that case the value d_g should be assumed to be 0.

For inclined shear reinforcement or bent-up bars (Figure 7.3-29), Eq. (7.3-66) is replaced by:

$$V_{Rd,s} = \sum A_{sw} k_e \sigma_{swd} \sin\alpha \qquad (7.3\text{-}64)$$

and Eq. (7.3-67) is replaced by:

$$\sigma_{swd} = \frac{E_s \psi}{6} (\sin\alpha + \cos\alpha) \cdot \left(\sin\alpha + \frac{f_{bd}}{f_{ywd}} \frac{d}{\phi_w} \right) \leq f_{ywd} \qquad (7.3\text{-}65)$$

The parameter k_ψ depends on the deformations (rotations) of the slab and follows from:

$$k_\psi = \frac{1}{1.5 + 0.9 k_{dg} \psi d} \leq 0.6 \qquad (7.3\text{-}63)$$

where d is the mean value [in mm] of the (flexural) effective depth for the x- and y-directions.

Provided that the size of the maximum aggregate particles, d_g, is not less than 16 mm, k_{dg} in Eq. (7.3-63) can be taken as k_{dg} = 1.0.

The design shear resistance provided by the stirrups may be calculated as

$$V_{Rd,s} = \sum A_{sw} k_e \sigma_{swd} \qquad (7.3\text{-}66)$$

where ΣA_{sw} is the sum of the cross-sectional area of all shear reinforcement suitably anchored, or developed, and intersected by the potential failure surface (conical surface with angle 45°) within the zone bounded by $0.35d_v$ and d_v from the edge of the supported area (Figure 7.3-29).

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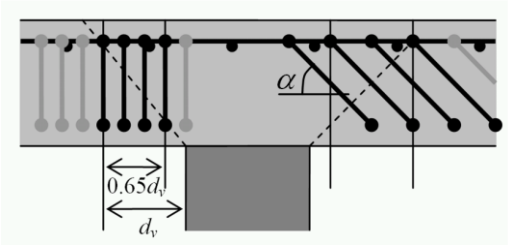


Figure 7.3-29: Shear reinforcement activated at failure

If more restrictive detailing rules are adopted ($s_0 \le 0.5d_v$ and $s_1 \le 0.6d_v$, with s_0 and s_1 according to Figure 7.13-10 and if the placing of the transverse reinforcement is checked at the construction site (distance between transverse reinforcements, top and bottom cover), the value k_{sys} can be increased as follows:

- k_{sys} = 2.4 for stirrups with sufficient development length at the compression face of the slab and bent (no anchorages or development length) at the tension face;
- k_{sys} = 2.8 for studs (diameter of heads larger or equal than three times the bar diameter).

Other values may be used for the coefficient k_{sys} provided that they are experimentally verified.

Slabs calculated under this assumption comply with deformation capacity requirements stated in subclause 7.3.4.

The term σ_{swd} refers to the stress that is activated in the shear reinforcement and can be calculated as:

$$\sigma_{swd} = \frac{E_s \psi}{6}\left(1 + \frac{f_{bd}}{f_{ywd}} \cdot \frac{d}{\phi_w}\right) \le f_{ywd} \qquad (7.3\text{-}67)$$

where ϕ_w denotes the diameter of the shear reinforcement and f_{ywd} its yield strength. The bond strength (f_{bd}) can be calculated according to subclause 6.1.3.2. Alternatively, a value f_{bd} = 3 MPa for corrugated bars may be used for design.

In order to ensure sufficient deformation capacity, in slabs with punching shear reinforcement a minimum amount of punching shear reinforcement is required such that:

$$\sum A_{sw} k_e f_{ywd} \ge 0.5 V_{Ed} \qquad (7.3\text{-}68)$$

The maximum punching shear resistance is limited by crushing of the concrete struts in the supported area:

$$V_{Rd,\max} = k_{sys} k_\psi \frac{\sqrt{f_{ck}}}{\gamma_c} b_0 d_v \le \frac{\sqrt{f_{ck}}}{\gamma_c} b_0 d_v \qquad (7.3\text{-}69)$$

The coefficient k_{sys} accounts for the performance of punching shear reinforcing systems to control shear cracking and to suitably confine compression struts at the soffit of the slab. In absence of other data, and provided that reinforcement is detailed as per the provisions of subclause 7.13.5.3, a value k_{sys} = 2.0 can be adopted.

7.3.5.4 Calculation of rotations around the supported area

Level I of Approximation

For a regular flat slab designed according to an elastic analysis without significant redistribution of internal forces, a safe estimate of the rotation at failure is:

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The value of r_s can be approximated as 0.22 L_x or 0.22 L_y for the x- and y-directions, respectively, for regular flat slabs where the ratio of the spans (L_x/L_y) is between 0.5 and 2.0. In Level I of Approximation, the maximum value of r_s has to be considered in Eq. (7.3-70).

The average bending moment acting in the support strip (m_{Ed}) can be approximated for each reinforcement direction and support type as:

– For inner columns (top reinforcement in each direction):

$$m_{Ed} = V_{Ed} \cdot \left(\frac{1}{8} + \frac{|e_{u,i}|}{2 \cdot b_s} \right) \tag{7.3-71}$$

– For edge columns:

when calculations are made considering the tension reinforcement parallel to the edge:

$$m_{Ed} = V_{Ed} \cdot \left(\frac{1}{8} + \frac{|e_{u,i}|}{2 \cdot b_s} \right) \geq \frac{V_{Ed}}{4} \tag{7.3-72}$$

when calculations are made considering the tension reinforcement perpendicular to the edge:

$$m_{Ed} = V_{Ed} \cdot \left(\frac{1}{8} + \frac{|e_{u,i}|}{b_s} \right) \tag{7.3-73}$$

– For corner columns (tension reinforcement in each direction):

$$m_{Ed} = V_{Ed} \cdot \left(\frac{1}{8} + \frac{|e_{u,i}|}{b_s} \right) \geq \frac{V_{Ed}}{2} \tag{7.3-74}$$

In these equations, term e_{ui} refers to the eccentricity of the resultant of shear forces with respect to the centroid of the basic control perimeter in the direction investigated (i = x and y for x and y directions respectively, see Figure 7.3-27).

$$\psi = 1.5 \cdot \frac{r_s}{d} \frac{f_{yd}}{E_s} \tag{7.3-70}$$

where r_s denotes the position where the radial bending moment is zero with respect to the support axis

Level II of Approximation

In cases where significant bending moment redistribution is considered in the design, the slab rotation can be calculated as:

$$\psi = 1.5 \cdot \frac{r_s}{d} \frac{f_{yd}}{E_s} \cdot \left(\frac{m_{Ed}}{m_{Rd}} \right)^{1.5} \tag{7.3-75}$$

where:

m_{Ed} is the average moment per unit length for calculation of the flexural reinforcement in the support strip (for the considered direction);

m_{Rd} is the design average flexural strength per unit length in the support strip (for the considered direction).

The rotation has to be calculated along the two main directions of the reinforcement.

The width of the support strip for calculating m_{Ed} is:

$$b_s = 1.5 \cdot \sqrt{r_{s,x} \cdot r_{s,y}} \leq L_{\min} \tag{7.3-76}$$

where close to slab edges, the width of the strip is limited to b_{sr} according to Figure 7.3-30. The same value for r_s as that for Level I of Approximation can be adopted.

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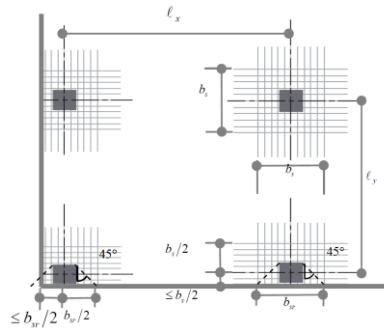


Figure 7.3-30: Support strip dimensions

Slabs calculated under this assumption do not comply with deformation capacity requirements stated in subclause 7.3.5.1. Therefore, they need to be provided with integrity reinforcement.

The design average flexural strength per unit length in the support strip is to be calculated accounting for both ordinary and prestressing steel at yielding.

This Level of Approximation is recommended for irregular slabs or for flat slabs where the ratio of the span lengths (L_x/L_y) is not between 0.5 and 2.0.

Parameter m_{Ed} has to be calculated consistently with the method used for determining the flexural reinforcement and is to be determined at the edge of the supported area maximizing m_{Ed}, see Figure 7.3-31.

Eq. (7.3-75) also applies to slabs with a flexural reinforcement that is increased over the supported areas in order to increase their punching shear strength.

For prestressed slabs, Eq. (7.3-75) can be replaced by:

$$\psi = 1.5 \cdot \frac{r_s}{d} \frac{f_{yd}}{E_s} \cdot \left(\frac{m_{Ed} - m_{Pd}}{m_{Rd} - m_{Pd}} \right)^{1.5} \qquad (7.3\text{-}77)$$

where m_{Pd} denotes the average decompression moment over the width of the support strip (b_s) due to prestressing. Constrained forces and moments and losses due to shrinkage, creep and relaxation shall be taken into account.

Level III of Approximation

The coefficient 1.5 in Eqs. (7.3-75) and (7.3-77) can be replaced by 1.2 if:

- r_s is calculated using a linear elastic (uncracked) model;
- m_{Ed} is calculated from a linear elastic (uncracked) model as the average value of the moment for design of the flexural reinforcement over the width of the support strip (b_s).

The width of the support strip can be calculated as in Level II of Approximation taking $r_{s,x}$ and $r_{s,y}$ as the maximum value in the direction investigated. For edge or corner columns, the following minimum value of r_s has to be considered:

$$r_s \geq 0.67 b_{sr} \qquad (7.3\text{-}78)$$

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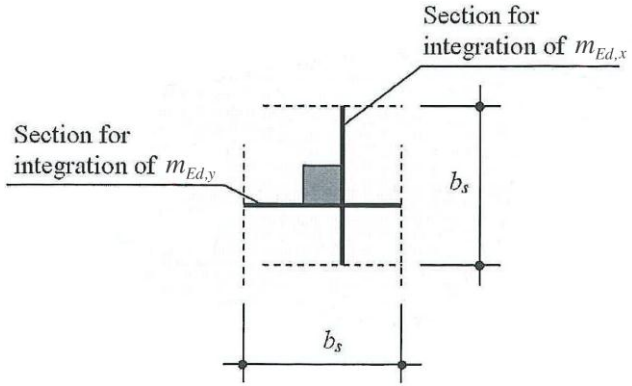


Figure 7.3-31: Example of sections for integration of support strip moments

Analytical or numerical techniques (for example, finite elements, finite differences, etc.) may be used for Level IV Approximation.

Level IV of Approximation

The rotation ψ can be calculated on the basis of a nonlinear analysis of the structure and accounting for cracking, tension-stiffening effects, yielding of the reinforcement and any other non-linear effects relevant for providing an accurate assessment of the structure.

7.3.5.5 Punching shear resistance outside the zones with shear reinforcement or shearheads

The extent of the slab with shear-reinforcement can be determined by checking the resistance of the slab outside this region. Subclause 7.3.5.3 applies by accounting for a control perimeter with a maximum effective distance between two shear reinforcing elements of $3d_v$ (Figure 7.3-32).

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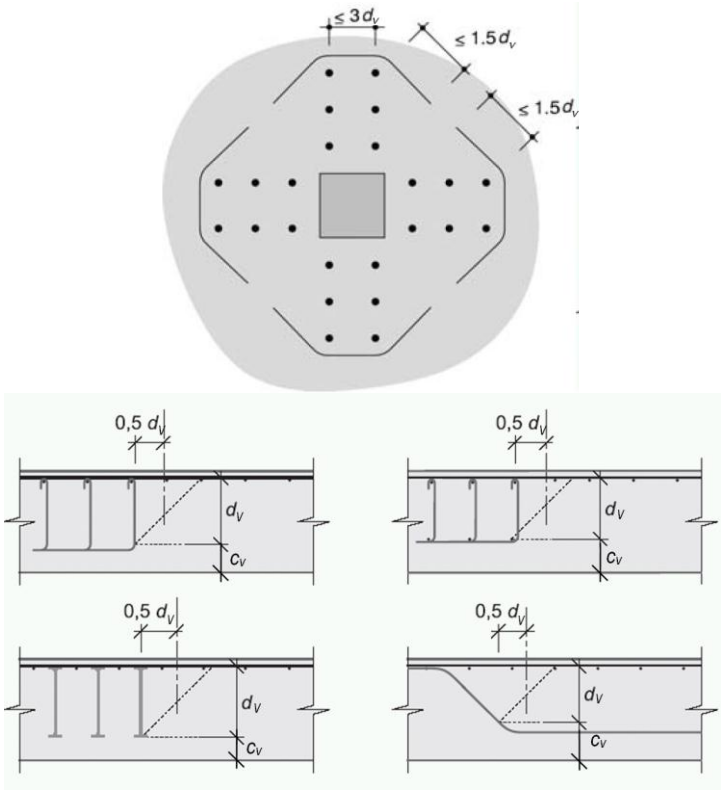


Figure 7.3-32: Reduced control perimeter and shear-resisting effective depth

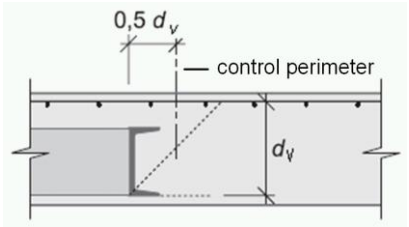


Figure 7.3-33: Shear-resisting effective depth and control perimeter accounting for shearhead penetration

The punching shear resistance of a slab outside of the shearhead is calculated on the basis of subclause 7.3.5.3 considering the shearhead as a rigidly supported area. The shear-resisting effective depth shall account for the position of the shearhead in the slab as shown in Figure 7.3-33.

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The design shear for calculation of the integrity reinforcement can be calculated on the basis of an accidental situation where progressive collapse has to be avoided.

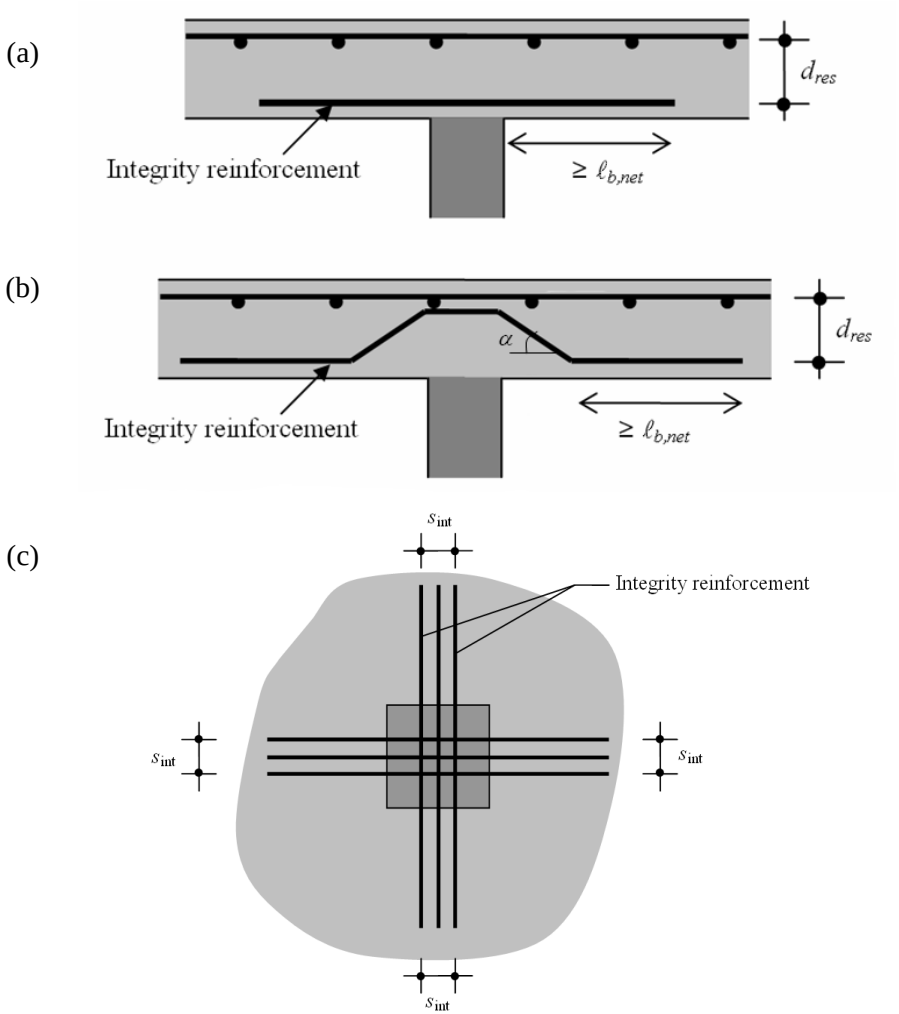


Figure 7.3-34: Integrity reinforcement: (a) straight bars; (b) bent-up

7.3.5.6 Integrity reinforcement

Slabs without shear reinforcement, or with insufficient deformation capacity, shall be provided with integrity reinforcement (Figure 7.3-34) to avoid progressive collapse.

The resistance provided after punching by the integrity reinforcement can be calculated as:

$$V_{Rd,int} = \sum A_s f_{yd} \left(f_t / f_y\right)_k \sin\alpha_{ult} \leq \frac{0.5\sqrt{f_{ck}}}{\gamma_c} d_{res} b_{int} \qquad (7.3\text{-}79)$$

where:

A_s refers to the sum of the cross-sections of all reinforcement suitably developed beyond the supported area on the compression side of the slab or to well-anchored bent-up bars;

f_{yd} is the design yield strength of the integrity bars;

ratio $(f_t/f_y)_k$ and parameter ε_{uk} are defined in subclause 5.2.5.4 and depend on the ductility class of the reinforcement;

α_{ult} is the angle of the integrity bar with respect to the slab plane at failure (after development of plastic deformations in the post-punching regime):

α_{ult}	*Type of integrity reinforcement*
0°	Straight bars, class of ductility: A
20°	Straight bars, class of ductility: B
25°	Straight bars, class of ductility: C or D
$\alpha \leq 40°$	Inclined or bent-up bars, class of ductility: B, C or D

where α is the angle of the integrity bars with respect to the slab plane (before punching occurs), Figure 7.3-34;

d_{res} is the distance between the centroid of the flexural reinforcement ratio and the centroid of the integrity reinforcement, see Figure 7.3-34(a) and (b);

b_{int} is the control perimeter activated by the integrity reinforcement after punching. It can be calculated as:

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The integrity reinforcement should at least be composed of four bars placed over the supported area and correctly developed on the compression side of the slab. Post-tensioning tendons can also be considered as integrity reinforcement.

In order to allow full activation of the integrity bars, the diameter of the integrity bars (ϕ_{int}) has to be chosen such that: $\phi_{int} \leq 0.12\, d_{res}$.

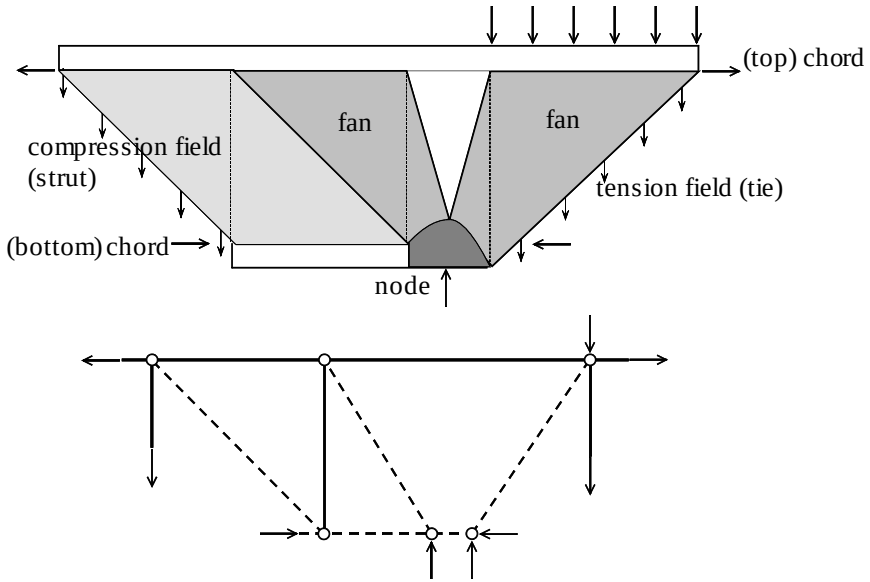


Figure 7.3-35: Basic elements for stress field analysis as well as for strut-and-tie modelling and statically equivalent truss models

$$b_{int} = \sum (s_{int} + \frac{\pi}{2} d_{res}) \qquad (7.3\text{-}80)$$

where the summation refers to the groups of bars activated at the edge of the supported area and s_{int} is equal to the width of the group of bars (refer to Figure 7.3-34).

7.3.6 Design with stress fields and strut-and-tie models

7.3.6.1 General

Structures can be subdivided into *B*-regions, where the assumption of a plane section may be used (*B* for Bernoulli) and *D*-regions, where a non-linear strain distribution exists (*D* for discontinuity); *D*-regions typically are located at supports or at places of concentrated loads.

Stress fields and strut-and-tie models represent the force flow within a structural member or structure and consist of nodes, compressive fields, struts, fans, tensile fields, ties and chords (Figure 7.3-35). These elements may be characterized as follows:

- node: highly bi- or triaxially stressed zone within a stress field;
- compressive field, strut: parallel stresses of uniaxial compression with uniform stress intensity;
- fan: fan-shaped stress field of uniaxial compression with variable stress intensity;
- tensile field, tie: parallel stresses of uniaxial tension with uniform stress intensity; and
- chord (tension, compression): tensile or compressive elements with a stress intensity that varies along their axis.

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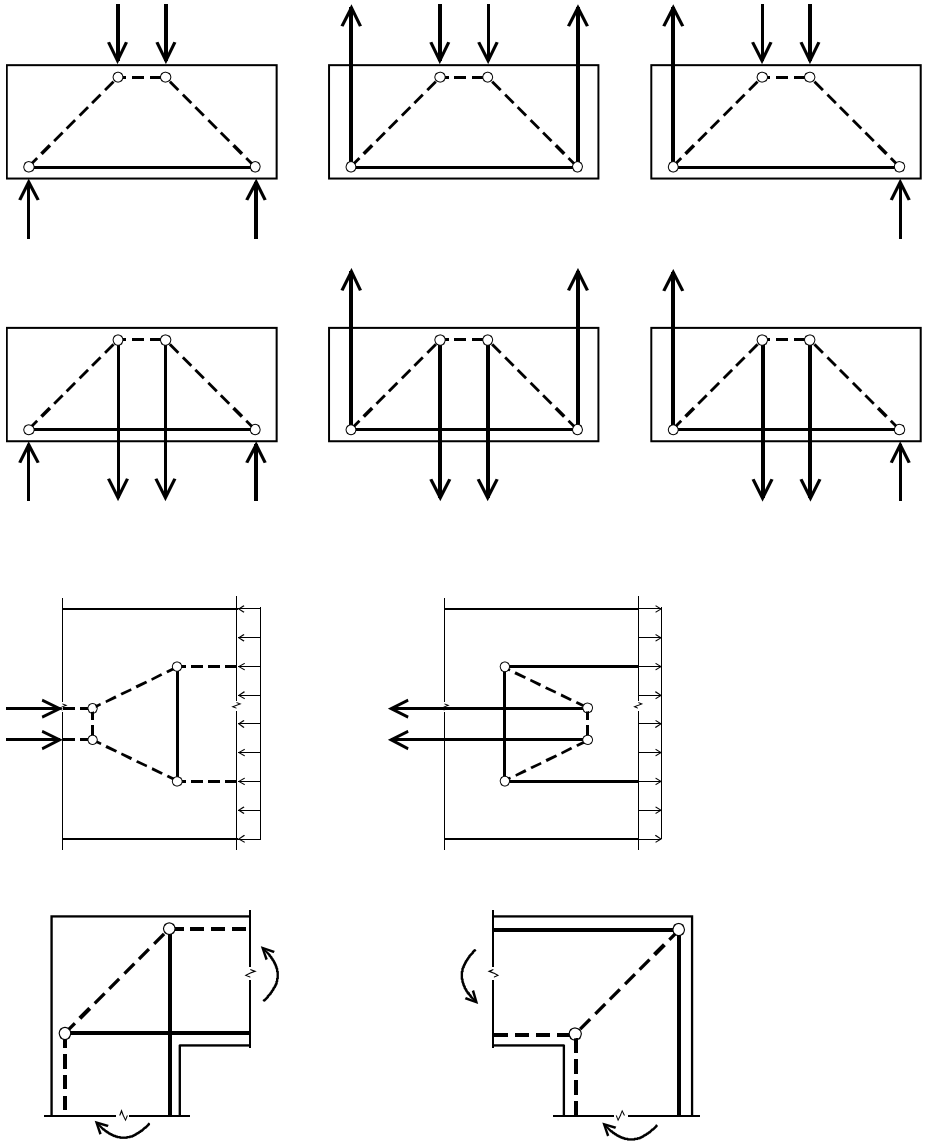


Figure 7.3-36: Exemplary basic elements for strut-and-tie modelling

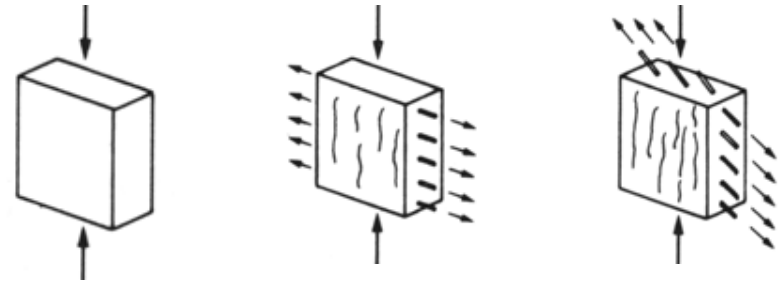
The forces within a stress field or strut-and-tie model can be calculated with help of equilibrium conditions.

When developing a model it is advisable to roughly take compatibility of deformations into consideration. In a first approximation, directions and magnitudes of the forces of the model may be orientated at the corresponding linear elastic stress state.

When applying stress fields or strut-and-tie models (Figure 7.3-36) the following steps may be considered:

- the geometry of the *D*-region may be assumed and have a minimum length equal to the maximum width of spread;
- a free-body with a (first) strut-and-tie model may be sketched. In order to minimize the effects of redistribution of forces (with consequences for crack width in the SLS) the struts should as much as possible be oriented to the compressive stress trajectories in the uncracked state;
- forces of the model have to be calculated such that they represent an equilibrium system of internal forces and external loads;
- the cross-section of the struts (compressive fields) and ties (tensile fields) shall be determined or checked;
- geometry of nodes shall be checked and detailing of reinforcement developed;
- the model has to be refined if necessary; and
- nodes, struts and ties of the final model must comply with the detailing of the reinforcement.

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Eq. (7.3-81) Eq. (7.3-82) Eq. (7.3-83)

Figure 7.3-37: Various states of stress: (a) undisturbed uniaxial compression; (b) tension normal to the direction of compression; (c) tension oblique to the direction of compression

Refer to subclause 7.3.3.4 for the background of the parameter η_{fc}.

7.3.6.2 Struts

For the dimensioning of the struts or stress fields the reduced concrete compressive strength $k_c f_{ck}/\gamma_c$ shall be used.

For undisturbed uniaxial compression states (Figure 7.3-37a) and for regions with transverse compression the reduction factor is:

$$k_c = 1.0\eta_{fc} \tag{7.3-81}$$

For struts or stress fields with cracks parallel to the direction of compression and tension reinforcement perpendicular to this (see Figure 7.3-37b) the reduction factor is:

$$k_c = 0.75\eta_{fc} \tag{7.3-82}$$

For struts or stress fields with reinforcement running obliquely (with angles smaller than 65°) to the direction of compression, for example webs of beams (see Figure 7.3-37c), the reduction factor is:

$$k_c = 0.55\eta_{fc} \tag{7.3-83}$$

$$\eta_{fc} = \left(\frac{30}{f_{ck}}\right)^{1/3} \le 1.0 \qquad (f_{ck} \text{ in MPa}) \tag{7.3-84}$$

The design compressive stress values given above may be increased by 10% where a biaxial state of compression is assured or all angles between struts and ties are ≥ 45° and where the reinforcement is arranged in multiple layers.

7.3.6.3 Ties

The design strengths of tensile fields or ties are defined as:

$$f_{yd} = \frac{f_{yk}}{\gamma_s} \qquad \text{for normal strength steel} \tag{7.3-85}$$

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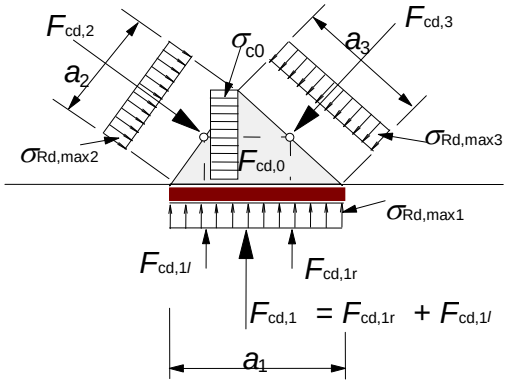


Figure 7.3-38: Compression nodes without ties

$$f_{pd} = \frac{f_{p0.1k}}{\gamma_s} \qquad \text{for prestressing steel} \tag{7.3-86}$$

In the case of post-tensioned members the initial prestressing state (defined by σ_{p0}) may be considered with the relevant anchorage and deviation forces. When applying this procedure the additional stress in the prestressing steel is limited to:

$$\Delta\sigma_p = f_{pd} - \sigma_{p0} \tag{7.3-87}$$

If appropriate, the reinforcement required to resist the forces may be distributed over the relevant length (Figure 7.3-35).

7.3.6.4 Nodes

The maximum stress to be applied at the edge of a node is limited to:

$$\sigma_{Rd,max} = k_c f_{ck} / \gamma_c \tag{7.3-88}$$

For compression nodes where no ties are anchored at the node, Figure 7.3-38, the reduction factor is the same as for Eq. (7.3-81).

A value of 1.1 η_{fc} may be assumed in regions where significant biaxial compression exists. This increase also applies if the stresses at supports or point loads are uniform and the node is confined by stirrups and if the node is reliably confined by means of bearing arrangement or friction. In cases with triaxial compression subclauses 7.2.3.1.6 and 7.2.3.1.7 apply.

For compression-tension nodes with anchored ties provided in one or two directions, Figure 7.3-39 the reduction factor is the same as for Eq. (7.3-82), where η_{fc} is defined in subclause 7.3.6.2.

The anchorage of the reinforcing bars in compression-tension nodes starts at the beginning of the node, e.g. in case of a support anchorage starting at its inner face (Figure 7.3-39). The anchorage lengths should extend over the entire node length.

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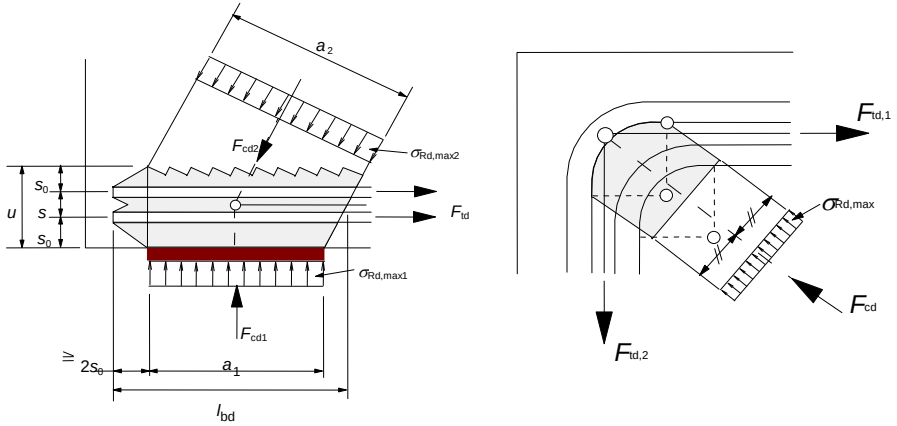


Figure 7.3-39: Compression-tension node with reinforcement provided in one and two directions

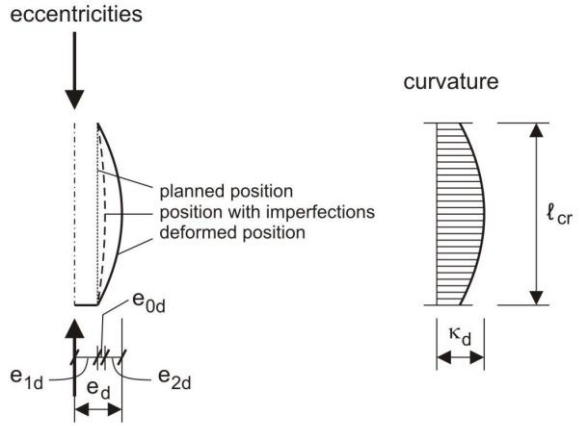


Figure 7.3-40: Compression resultant with eccentricities and curvature variation

The anchorage of the reinforcing bars behind the node is strongly recommended in cases where the member dimensions are large enough. This type of anchorage is beneficial as it creates a state of pure compression in the node.

7.3.7 Compression members

7.3.7.1 Stability of compressed members in general

Where the behaviour of a member is significantly influenced by 2nd order effects (Figure 7.3-40), the verification is carried out for the deformed system and the dimensioning values of the actions. The geometrical imperfections according to subclause 7.2.2.2 have to be taken into account. Furthermore the influence of cracking, nonlinear deformation of the structural materials and time-dependent deformations should be considered.

The dimensioning value of the bending moment is:

$$M_d = -N_d e_d \tag{7.3-89}$$

The maximum eccentricity e_d, that is the maximum distance between the compression resultant and the deformed axis of the compression member, may be determined as follows:

$$e_d = e_{0d} + e_{1d} + e_{2d} \tag{7.3-90}$$

where e_{0d} is eccentricity due to imperfections, being the greater value of:

$$e_{0d} = \alpha_i \, l_0/2 \quad \text{and} \quad e_{0d} = d/30 \tag{7.3-91}$$

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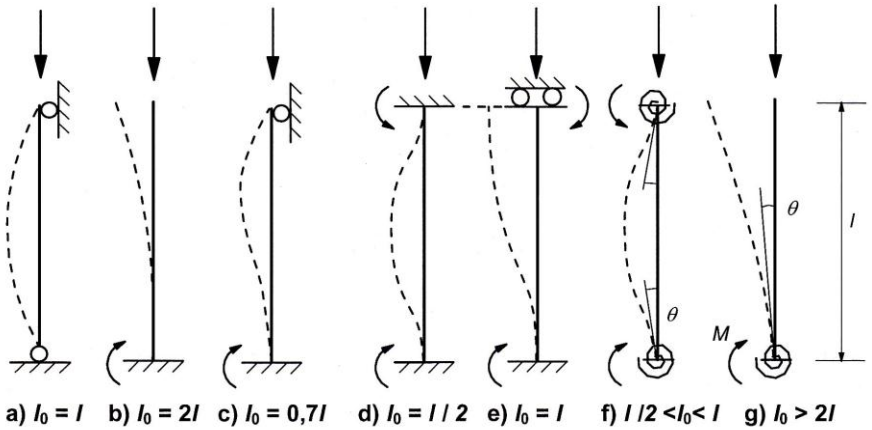


Figure 7.3-41: Examples of different buckling modes and corresponding effective lengths for isolated members

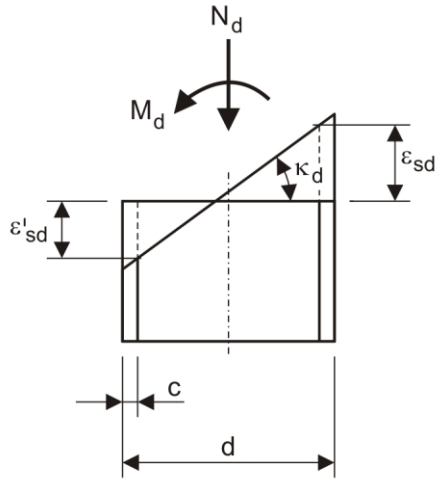


Figure 7.3-42: Strain plane corresponding to maximum curvature

where l_0, is the effective length of the compressed member, see for example Figure 7.3-41. The value of α_i can be estimated as:

$$\frac{1}{200} \geq \alpha_i = \frac{0.01}{\sqrt{l_0}} \geq \frac{1}{300} \qquad (l_0 \text{ in m}) \tag{7.3-92}$$

The first order eccentricity e_{1d} can be estimated according to:

$$e_{1d} = \frac{M_{1d}}{-N_d} \tag{7.3-93}$$

The eccentricity due to the deformation of the compression member e_{2d} may be calculated by:

$$e_{2d} = \kappa_d \frac{l_0^{\,2}}{c_0} \tag{7.3-94}$$

where κ_d is the maximum design curvature (Figure 7.3-42):

$$\kappa_d = \frac{\varepsilon_{sd} - \varepsilon'_{sd}}{d - 2c} \tag{7.3-95}$$

and c_0 is the integration factor accounting for the curvature distribution along the member.

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Eq. (7.3-98) is based on interpolation using the interaction diagram, which is normally used as a design aid to determine the reinforcement in cross-sections subjected to a normal force and a bending moment. Figure 7.3-43 shows a simplified representation of such a diagram. In point B the reinforcement yields at both sides of the column, so that the curvature is $\kappa = \varepsilon_{yd}/(0.45d)$. At point A, the curvature $\kappa = 0$. So, the curvature in point C can be obtained by interpolation from Eq. (7.3-98).

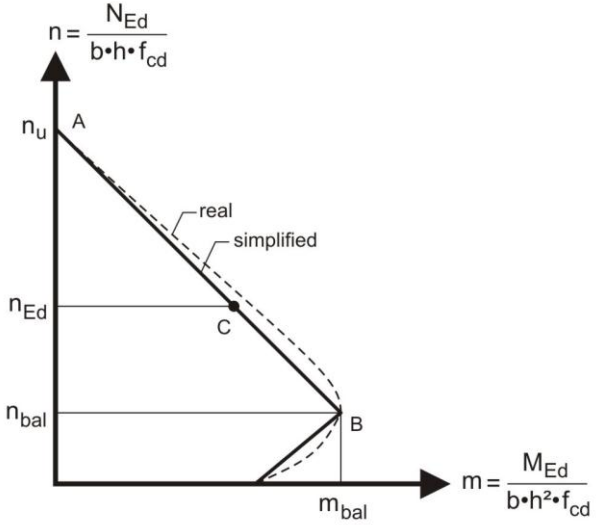


Figure 7.3-43: Simplified representation of interaction diagram

Level I approximation

The value of the integration factor can be assumed as

$$c_0 = \pi^2 \tag{7.3-96}$$

The maximum design curvature may be obtained from Eq. (7.3-95) with:

$$\varepsilon_{sd} = \frac{f_{yd}}{E_s} \quad \text{and} \quad \varepsilon_{sd}' = -\frac{f_{yd}}{E_s} \tag{7.3-97}$$

Level II approximation

A more accurate value of the maximum design curvature can be obtained from the equation:

$$\kappa_d = \left(\frac{n_u - n_d}{n_u - n_{bal}}\right) \cdot \frac{\varepsilon_{yd}}{0.45d} \tag{7.3-98}$$

where:

$n_u = 1 + \omega$;

$n = N_{Ed}/(A_c f_{cd})$;

n_{bal} = value of n at maximum moment resistance ≈ 0.4 (point B in Figure 7.3-43);

$\omega = A_s f_{yd}/(A_c f_{cd})$.

The long term deformations due to creep and shrinkage of the concrete may approximately be taken into account as pre-curvature of the cross-section. The maximum curvature according to Eq. (7.3-98) shall be increased by adding the irreversible portion:

$$\kappa_{d,\infty} = \frac{|\varepsilon_{c\infty}|}{d} \tag{7.3-99}$$

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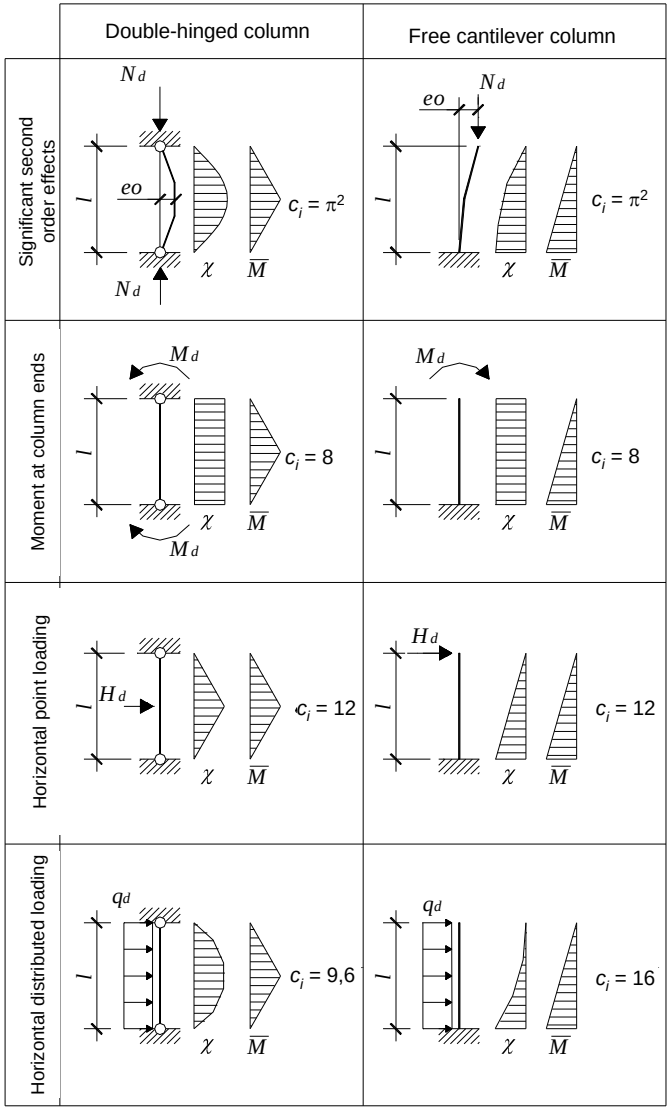


Figure 7.3-44: Values of integration factors c_i as a function of the load type and the boundary conditions

Level III approximation

A more refined value of the integration factor c_0 can be calculated on the basis of the values of the various integration factors c_i for each action according to:

$$c_0 = \pi^2 \cdot \frac{N}{N_{cr}} + \frac{\sum_{i=1}^{n} M_i}{\sum_{i=1}^{n} \frac{M_i}{c_i}} \cdot \left(1 - \frac{N}{N_{cr}}\right) \qquad (7.3\text{-}100)$$

where the factors c_i are given in Figure 7.3-44.

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Level IV of approximation

A refined calculation of second order effects can be determined using an analysis that accounts for nonlinear behaviour of concrete in compression, cracking, creep and shrinkage, reinforcement yielding and other non-linear effects important to the change in behaviour over time and loading state.

7.3.7.2 Biaxial eccentricities and out-of-plane buckling

For members with rectangular cross-sections, separate verifications in the two principal planes *y* and *z* are permissible, if the point of application of N_{Ed} is located close to one principal axis, for example within the hatched zones in Figure 7.3-45. The ratios of the corresponding eccentricities e_{y1}/b and e_{z1}/h have to satisfy one of the following:

$$(e_{z1}/h)/(e_{y1}/b) \le 1/4; \quad \text{or} \tag{7.3-101}$$

$$(e_{y1}/h)/(e_{z1}/b) \le 1/4 \tag{7.3-102}$$

where the eccentricities e_{y1} and e_{z1} are those in the directions of the section dimensions *b* and *h*, respectively, and include an imperfection allowance e_{0d}, as defined in Eq. (7.3-90).

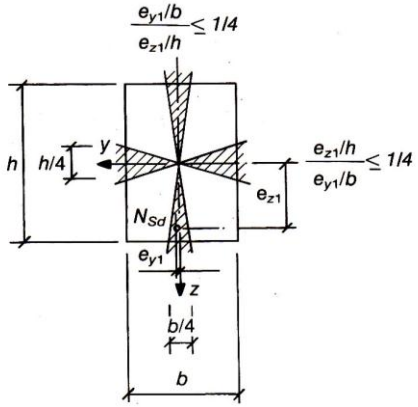


Figure 7.3-45: Condition for separate verification in the two principal planes

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If the criteria expressed by Eqs. (7.3-103) and (7.3-104) are not satisfied, the cross-section should be designed for biaxial bending, including the second order effects in each direction. In the absence of a refined cross-sectional analysis for biaxial bending, the following simplified criterion may be used:

$$\left(\frac{M_{Edx}}{M_{Rdx}}\right)^a + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^a \leq 1.0 \qquad (7.3\text{-}103)$$

where:

$M_{Edx/y}$ is the design moment around the respective axis, including nominal 2nd order moments;

$M_{Rdx/y}$ is the moment resistance of the cross-section in the respective direction;

a is an exponent which is 2 for circular and elliptical cross-sections and for rectangular cross-sections follows from:

N_{Ed}/N_{Rd}	0.1	0.7	1.0
a	1.0	1.5	2.0

N_{Ed} is the design value of axial force; and

N_{Rd} is the design axial resistance of section = $A_c f_{cd} + A_s f_{yd}$.

7.3.8 Lateral instability of beams

A check of the lateral instability of beams is relevant in long beams where lateral bracing is lacking.

Situations where a check is necessary are, for example, slender precast beams during transport and erection and beams with insufficient lateral bracing in the finished structure. In such cases, geometric imperfections should be taken into account.

A lateral deformation of $l/300$ should be assumed as a geometric imperfection in the verification of beams in unbraced conditions, where l is the total length of the beam. In finished structures, bracing from connected members may be taken into account.

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For background reference is made to Foster, S., Marti, P., and Mojsilović, N. "Design of Reinforced Concrete Solids Using Stress Analysis" (ACI Structural Journal, V100, N6, Nov-Dec. 2003, pp. 758-764).

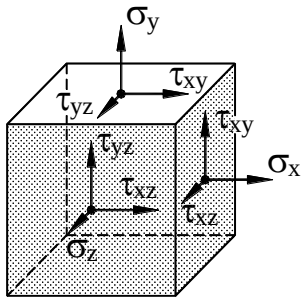


Figure 7.3-46: 3D stresses at a point defined in the orthogonal xyz axis system

In the application of this design method, the *xyz* axes are taken to correspond with reinforcing directions. The normal stresses applied at a point in a reinforced concrete solid element are carried by reinforcing steel and/or the concrete whilst shear stresses are carried by the concrete alone. Given

Second order effects with regard to lateral instability may be ignored if the following condition is fulfilled:

$$\frac{l_{0f}}{b} \le \frac{50}{(h/b)^{1/3}} \tag{7.3-104}$$

where:

l_{0f} is the unbraced length of the compression flange;

h is the total depth of the cross-section of the beam in the central part;

b is the width of the compression flange.

7.3.9 3D Solids

7.3.9.1 Stress limit requirements

Considering a stress tensor for a reinforced concrete 3D element (Figure 7.3-46), the applied stresses on an element can be replaced by equivalent stresses in the concrete (subscript *c*) and in the reinforcement (subscript *s*), according to:

$$\left(\frac{M_{Edx}}{M_{Rdx}}\right)^a + \left(\frac{M_{Edy}}{M_{Rdy}}\right)^a \le 1.0 \tag{7.3-105}$$

where *x*, *y* and *z* are the axis directions of the orthogonal reinforcing steel.

The equivalent reinforcement stresses are limited by:

$$\left|\sigma_{s.j}\right| \le \rho_{s.j} f_{yd.j} \tag{7.3-106}$$

where $\rho_{s.j}$ ($j = x, y, z$) are the reinforcement ratios in the *x*, *y* and *z* directions, respectively.

The concrete stresses (ordered as $\sigma_{c3} \le \sigma_{c2} \le \sigma_{c1}$ as shown in Figure 7.3-47) are required to satisfy:

$$-\sigma_{c3} \le \nu f_{cd} \tag{7.3-107}$$

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that the applied stress tensor has been determined, for example by 3D finite element solid modelling, the Mohr's circles of applied stress may be plotted, as shown in Figure 7.3-47. Within the circles the stress points (σ_i, S_i) are also plotted where $i = x, y, z$. As the reinforcing steel cannot carry shear stress it follows that the points denoting the concrete stresses (σ_{ci}, S_{ci}) must fall within the hatched region of the concrete stress circles where $\sigma_{ci} = \sigma_i - \rho_{s.j}\sigma_{s.j}$ and $S_{ci} = S_i$.

In the *xyz* space σ_x, σ_y and σ_z are, by definition, normal to the *yz*, *xz* and *xy* planes, respectively. The magnitudes of the shear stresses on these planes are given by

$$S_x = \sqrt{\tau_{xy}^2 + \tau_{xz}^2}\ ;\ S_y = \sqrt{\tau_{xy}^2 + \tau_{yz}^2}\ ;\ S_z = \sqrt{\tau_{xz}^2 + \tau_{yz}^2}$$

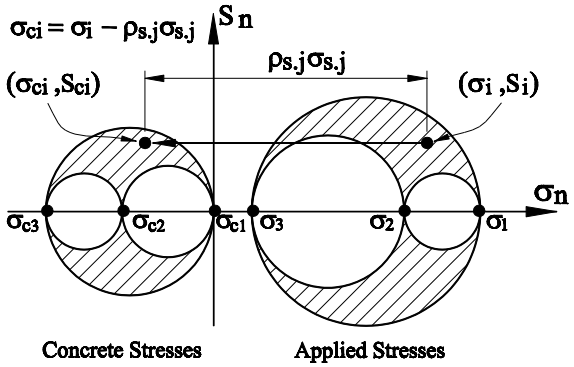


Figure 7.3-47: Compression field for 3D stress at a point

In developing solutions, the designer must "respect" the limitations of the concrete material. In a solid subject to a constant ratio of normal and shear stresses (with at least one tensile principal stress) before cracking the stress field in the concrete remains relatively elastic and the stresses in the

If no reinforcement has yielded and at least one principal stress is in tension then:

$$\nu = \frac{1.18}{1.14 + 0.00166\sigma_{si}} \le 1.0 \qquad (7.3\text{-}108)$$

where σ_{si} is the maximum tensile stress (in MPa) in any layer of reinforcing steel ($i = x, y, z$).

If one or more layers of reinforcement yield:

$$\nu = \left(1 - 0.032\left|\delta_i\right|\right) \cdot \frac{1.18}{1.14 + 0.00166\, f_{yd}} \qquad (7.3\text{-}109)$$

where δ_i is given by Eq. (7.3-116).

If all principal stresses are compressive, ν may be taken as 1.0 or determined in accordance with subclause 5.1.6.

7.3.9.2 Ductility requirements

The ductility demand can be assessed by comparing the principal stress directions (Figure 7.3-48) of the resulting concrete stress tensor with that of the applied, factored, loads.

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reinforcement are negligible. After cracking, the tensile stresses in the concrete reduce while those in the reinforcing steel increase. If the concrete does not fail in compression then the crack directions will remain relatively stable until yielding of the steel in one direction. After yielding in one direction the forces are continuously redistributed to balance the applied tractions until yielding in all directions has occurred. Concrete elements have a limit on the amount of redistribution that can be achieved. As a rule concrete elements should not be pushed far beyond that which is "natural". Designers should critically examine the load path being assumed to ensure that a sufficient level of ductility is available to meet the demands of the imposed tractions. It is suggested to limit $|\delta_i| \leq 15^o$ (Figure 7.3-48).

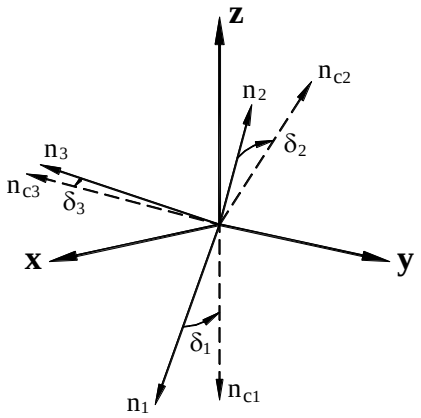


Figure 7.3-48: Comparison of concrete principal stress directions and the principal stress directions due to the applied tractions for the case of optimum reinforcement

The direction cosines of the principal stresses of the loading tensor $\mathbf{n_i} = \{n_{ix}, n_{iy}, n_{iz}\}$ (i = 1, 2, 3) are:

$$n_{ix} = \frac{-c_{iy}\, c_{iz}}{C} \tag{7.3-110}$$

$$n_{iy} = \frac{-c_{ix}\, c_{iz}}{C} \tag{7.3-111}$$

$$n_{iz} = \frac{-c_{ix}\, c_{iy}}{C} \tag{7.3-112}$$

where $C = \sqrt{c_{ix}^2 c_{iy}^2 + c_{ix}^2 c_{iz}^2 + c_{iy}^2 c_{iz}^2}$ and where:

$$c_{ix} = (\sigma_x - \sigma_i)\tau_{yz} - \tau_{xy}\tau_{xz} \tag{7.3-113}$$

$$c_{iy} = (\sigma_y - \sigma_i)\tau_{xz} - \tau_{xy}\tau_{yz} \tag{7.3-114}$$

$$c_{iz} = (\sigma_z - \sigma_i)\tau_{xy} - \tau_{xz}\tau_{yz} \tag{7.3-115}$$

The direction cosines of the principal concrete stresses $\mathbf{n_{ci}}$ are calculated from Eqs. (7.3-110) to (7.3-112) and Eqs. (7.3-113) to (7.3-115) with σ_{cx}, σ_{cy} and σ_{cz} substituted for σ_x, σ_y and σ_z, respectively.

The enclosed angles between the concrete stresses and those from the applied loading δ_i ($i = 1, 2, 3$) are given by:

$$\delta_i = \cos^{-1}\left|n_{ix}\, n_{cix} + n_{iy}\, n_{ciy} + n_{iz}\, n_{ciz}\right| \tag{7.3-116}$$

The rotational demands as indicated by the differences in the stresses resulting from the applied loading and those in the concrete should be limited to ensure sufficient ductility capacity to meet the demands.

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The given fatigue strength of concrete is valid for concrete tested under sealed conditions (see subclause 5.1.11). The fatigue strength of steel is given both for a normal environment and for a marine environment.

7.4 Verification of structural safety (ULS) for non-static loading

7.4.1 Fatigue design

7.4.1.1 Scope

The following design rules apply for the entire service life of concrete structures. The rules for reinforcing and prestressing steel should be applied if more than 10^4 load repetitions are expected; low-cycle fatigue is not covered.

The verification of the design principle (see subclause 4.5.2.3) can be performed according to the three methods given in subclauses 7.4.1.3, 7.4.1.4 and 7.4.1.5, with an increasing refinement. The models for the analysis of stresses in reinforced and prestressed concrete members under fatigue loading are treated in subclause 7.4.1.2 as well as concrete stress gradients.

Subclause 7.4.1.6 deals with shear design and in 7.4.1.7 a method for calculating the increased deflections under fatigue loading is given. The relevant combination of loads is treated in subclause 4.5.2.3.1.

7.4.1.2 Analysis of stresses in reinforced and prestressed members under fatigue loading

Linear elastic models may generally be used, and reinforced concrete in tension is considered to be cracked. The ratio of moduli of elasticity for steel and concrete may be taken as $\alpha = 10$.

In the case of prestressed members it should be verified if the relevant section is sensitive to cracking. This holds true if any combination of loads (see subclause 4.5.2.5) causes tensile stresses at the concrete surface. In that case the stress ranges for reinforcing steel and prestressing steel should be calculated assuming the cracked state.

The effect of differences in bond behaviour of prestressing and reinforcing steel has to be taken into account. Unless a more refined method is used, this can be done using a linear elastic model for stress calculation which fulfils the compatibility in strains and multiplying the stress in the reinforcing steel by the following factor:

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$$\eta_s = \frac{1+(A_p / A_s)}{1+(A_p / A_s)\sqrt{\xi(\phi_s / \phi_p)}} \qquad (7.4\text{-}1)$$

where:

η_s is the factor which increases the stress in the reinforcing steel due to differences in bond behaviour between prestressing and reinforcing steel;

A_s is the area of reinforcing steel;

A_p is the area of prestressing steel;

ϕ_s is the smallest diameter of the reinforcing steel in the relevant cross-section;

ϕ_p is the diameter of the prestressing steel (for bundles an equivalent diameter $1.6\sqrt{A_p}$ is chosen, where A_p is the cross-section area of the bundle);

ξ is the ratio of bond strength of prestressing steel and high-bond reinforcing steel.

For post-tensioned members the following values may be used:

ξ = 0.2 for smooth prestressing steel;

ξ = 0.4 for strands;

ξ = 0.6 for ribbed prestressing wires;

ξ = 1.0 for ribbed prestressing bars.

For pretensioned members the following values may be used:

ξ = 0.6 for strands;

ξ = 0.8 for ribbed prestressing steels.

The stress gradient for concrete in the compression zone of a cracked section may be taken into account by multiplying the maximum stress in the compression zone by a factor η_c, equal to:

$$\eta_c = \frac{1}{1.5 - 0.5|\sigma_{c1}|/|\sigma_{c2}|} \qquad (7.4\text{-}2)$$

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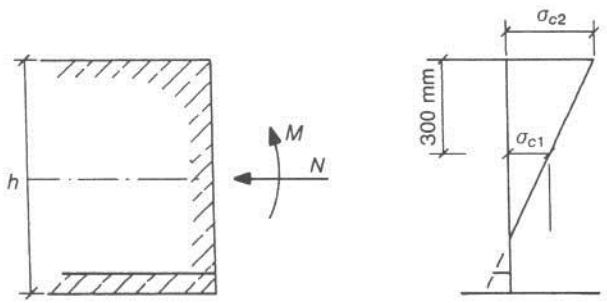


Figure 7.4-1: Definition of stress σ_{c1}, σ_{c2}

Values for $\gamma_{s,fat}$ and $\gamma_{c,fat}$ are given in subclause 4.5.2.3.

The fatigue reference strength is defined as follows (see also subclause 5.1.11).

where:

η_c is the averaging factor of concrete stresses in the compression zone considering the stress gradient;

$|\sigma_{c1}|$ is the minimum absolute value of the compressive stress within a distance of 300mm from the surface under the relevant load combination of actions;

$|\sigma_{c2}|$ is the maximum absolute value of the compressive stress within a distance of 300mm from the surface under the same load combination as that for which $|\sigma_{c1}|$ was determined.

7.4.1.3 Level I of Approximation: the simplified procedure

This procedure is only applicable to structures subjected to a limited number ($\leq 10^8$) of low stress cycles.

Steel

The fatigue requirements will be met, if the maximum calculated stress range under the frequent combination of loads, max $\Delta\sigma_{Ss}$, satisfies the condition

$$\gamma_{Sd} \max \Delta\sigma_{Ss} \leq \Delta\sigma_{Rsk} / \gamma_{s,fat} \qquad (7.4\text{-}3)$$

where:

$\Delta\sigma_{Rsk}$ is the characteristic fatigue strength at 10^8 cycles.

Values for $\Delta\sigma_{Rsk}$ are given in Table 7.4-1 and Table 7.4-2.

Concrete

Detailed fatigue design needs not be carried out if the maximum calculated stresses under the frequent combination of loads, $\sigma_{c,\max}$ (compression), $\sigma_{ct,\max}$ (tension), respectively, satisfy the following criteria:

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Compression

$$f_{cd,fat} = 0.85\beta_{cc}(t)\left[f_{ck}\left(1 - \frac{f_{ck}}{25 f_{ck0}}\right)\right] / \gamma_{c,fat}$$

where

$\beta_{cc}(t)$ is the coefficient which depends on the age t of the concrete in days when fatigue loading starts (see subclause 5.1.9.1);

$f_{ck0} = 10$ MPa (reference strength).

Tension

$$f_{ctd,fat} = f_{ctk,0.05} / \gamma_{c,fat}$$

For value of $\gamma_{c,fat}$, see subclause 4.5.2.3.

For $\sigma_{c,\max}$, $\sigma_{ct,\max}$, see subclause 4.5.2.3

When the unique value Q can be chosen satisfactorily, (e.g. as fatigue equivalent) this method is a more precise assessment than the simplified procedure.

When it is considered necessary to carry out fatigue tests to determine the performance of reinforcing steel, the tests should be made according to standardized procedures, as described in ISO 15630-1, 2, 3.

The characteristic fatigue strength function for steel consists of segments (see Figure 7.4-2) of the form $(\Delta_{Rsk})^m \cdot N = \text{const.}$ Values for the S-N curves are given in Table 7.4-1 and Table 7.4-2.

Compression

$$\gamma_{Sd}\sigma_{c,max}\eta_c \le 0.45 f_{cd,fat} \qquad (7.4\text{-}4)$$

where:

$\sigma_{c,max}$ is the maximum compressive stress;

η_c is an averaging factor considering the stress gradient Eq. (7.4-2);

$f_{cd,fat}$ is the design fatigue reference strength for concrete in compression.

Tension

$$\gamma_{Sd}\sigma_{ct,max} \le 0.33 f_{ctd,fat} \qquad (7.4\text{-}5)$$

where:

$\sigma_{ct,\max}$ is the maximum tensile stress in the concrete;

$f_{ctd,fat}$ is the design fatigue reference tensile strength of the concrete.

7.4.1.4 Level II of Approximation: verification by means of a single load level

This method takes account of the required service life with a foreseen number, n, of cycles. This number intervenes in the verification with the maximum fatigue effects of the action, Q, as defined in subclauses 4.5.2.3(b), subclause 7.4.1.2 and the paragraphs below.

Steel

The fatigue requirement will be met if the calculated maximum acting stress range, max $\Delta\sigma_{Ss}$, satisfies the condition:

$$\gamma_{Sd} \max \Delta_{Ss} \le \Delta\sigma_{Rsk}(n) / \gamma_{s,fat} \qquad (7.4\text{-}6)$$

where:

$\Delta\sigma_{Ss}$ is the steel stress range under the acting loads;

$\Delta\sigma_{Rsk}(n)$ is the stress range relevant to n cycles obtained from a characteristic fatigue strength function.

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The values given in Table 7.4-1 and Table 7.4-2 are characteristic and do not incorporate partial safety factors. These values or higher values shall be validated by appropriate approval documents.

The code does not cover coiled and re-straightened bars.

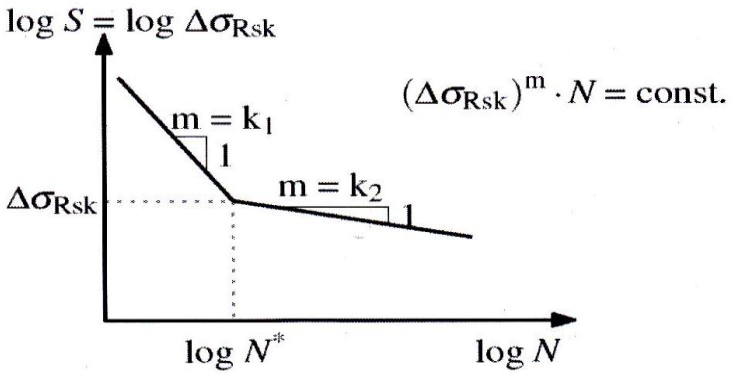


Figure 7.4-2: Shape of the characteristic fatigue strength curves (S-N curves) for steel

In case appropriate information is provided by specific approval documents for the steel to be used, higher fatigue strength values may be used accordingly.

Data on the fatigue behaviour of bars with diameters larger than 40 mm are scarce. Therefore no data for these bars are given here.

Table 7.4-1: Parameters of S-N curves for reinforcing steel (embedded in concrete)

	N^*	**Stress exponent**		$\Delta\sigma_{Rsk}$ (MPa)[e]	
		k_1	k_2	at N^* cycles	at 10^8 cycles
Straight and bent bars $D \geq 25\phi$					
$\phi \leq 16$ mm	10^6	5	9	210	125
$\phi > 16$ mm[a]	10^6	5	9	160	95
Bent bars $D < 25\phi$[b]	10^6	5	9	–[c]	–[c]
Welded bars[b] including tack	10^7	3	5	50	30
Welding and butt joints					
Mechanical connectors					
Marine environment[b],[d]	10^7	3	5	65	40

[a] The values given in this line represent the S-N curve of a 40 mm bar; for diameters between 16 and 40 mm interpolation between the values of this line and those of the line above is permitted.

[b] Most of these S-N curves intersect the curve of the corresponding straight bar. In such cases the fatigue strength of the straight bar is valid for cycle numbers lower than that of the intersection point.

[c] Values are those of the according straight bar multiplied by a reduction factor ξ depending on the ratio of the diameter of mandrel D and bar diameter ϕ: $\xi = 0.35 + 0.026D/\phi$.

[d] Valid for all ratios D/ϕ and all diameters ϕ.

[e] In cases where $\Delta\sigma_{Rsk}$ values calculated from the S-N curve exceed the stress range $f_{yd} - \sigma_{min}$, the value $f_{yd} - \sigma_{min}$ is valid.

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The values given in Table 7.4-2 are at the safe side compared to the strength values for the basic material given in subclause 5.1.11.

The reduction of the $\Delta\sigma_{Rsk}$ values for curved tendons compared with the values of straight tendons is due to fretting corrosion which results from the lateral pressure and slip between prestressing strands and/or ribs of the steel sheaths.

Table 7.4-2: Parameters of S-N curves for prestressing steel (embedded in concrete)

	N^*	**Stress exponent**		$\Delta\sigma_{Rsk}$ (MPa)[(a)]
		k_1	k_2	at N^* cycles
Pretensioned steel				
Straight strands and wires	10^6	5	9	185
Postensioned steel				
Single strands in plastic ducts	10^6	5	9	185
Straight tendons or curved tendons in plastic ducts	10^6	5	10	150
Curved tendons in steel ducts	10^6	5	7	120
Splicing devices	10^6	5	5	80

[(a)] In cases where the S-N curve intersects that of the straight tendon, the fatigue strength of the straight tendon is valid.

Characteristic S-N curves for concrete can be used without any restriction for frequencies higher than 0.1 Hz. For lower frequencies, the fatigue life should be reduced, see chapter 3 in CEB Bulletin 188 "Fatigue of Concrete Structures" (CEB, 1988) for guidance.

In the case of compression-tension the criteria for compression as well as for tension shall be fulfilled.

Concrete

The fatigue requirements under cyclic loading will be met if the required lifetime (number of cycles) is lower than or equal to the number of cycles to failure:

$$n \leq N$$

where:

n is the foreseen number of cycles during the required design service life;

N is the number of resisting stress cycles, to be calculated from the fatigue strength functions given below.

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For γ_{Ed} see subclause 4.5.2.3.

For η_c see Eq. (7.4-2).

For the assessment of $\sigma_{c,\max}$ and $\sigma_{c,\min}$, see subclause 4.5.2.3 and subclause 7.4.1.2 using the fatigue equivalent or frequent value of the variable action Q.

$\sigma_{c,\max}$ and $\sigma_{ct,\max}$ should be calculated under the upper load effect.

$\sigma_{c,\min}$ is determined as the maximum stress in the compression zone at a distance no more than 300 mm away from the surface where $\sigma_{c,\max}$ occurs, but under the lower load effect.

Compression

For $S_{cd,min} > 0.8$, the S-N relations for $S_{cd,min} = 0.8$ are valid.

For $0 \le S_{cd,min} \le 0.8$, Eqs. (7.4-7a)-(7.4-7b) apply.

$$\log N_1 = \frac{8}{Y-1} \cdot (S_{cd,\max} - 1) \tag{7.4.7a}$$

$$\log N_2 = 8 + \frac{8 \cdot \ln(10)}{Y-1} \cdot (Y - S_{cd,\min}) \cdot \log\left(\frac{S_{cd,\max} - S_{cd,\min}}{Y - S_{cd,\min}} \right) \tag{7.4.7b}$$

with

$$Y = \frac{0.45 + 1.8 \cdot S_{cd,\min}}{1 + 1.8 \cdot S_{cd,\min} - 0.3 \cdot S_{cd,\min}^2}$$

where

(a) if $\log N_1 \le 8$, then $\log N = \log N_1$;

(b) if $\log N_1 > 8$, then $\log N = \log N_2$;

where:

$\sigma_{c,\min}$ is the minimum compressive stress;

$S_{cd,\max}$ is the maximum compressive stress level;

$S_{cd,\max} = \gamma_{Ed} \sigma_{c,\max} \eta_c / f_{cd.fat}$;

$S_{cd,\min}$ is the minimum compressive stress level;

$S_{cd,\min} = \gamma_{Ed} \sigma_{c,\min} \eta_c / f_{cd.fat}$.

Compression – tension with $\sigma_{ct,\max} \le 0.026 |S_{td,\max}|$

$$\log N = 9(1 - S_{cd,\max}) \tag{7.4.8}$$

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The partial coefficients are applied in this procedure as follows

For steel, values N_{Ri} are calculated from the S-N curves given in Table 7.4-1 and Table 7.4-2 using an increased stress range $\gamma_{Ed}\gamma_{s,fat}\Delta\sigma_{Ssi}$.

For concrete, values N_{Ri} are calculated directly from the fatigue strength functions given in subclause 7.4.1.4.

For concrete D_{lim} depends on the stress history (Zang et. Al, Mag. Of Concrete Research, 49 (180), 1997, pp. 241-252). Under decreasing stress levels D_{lim} can be significantly smaller than 1.0.

If it can be demonstrated that in the structural member no shear cracks will occur under the relevant combination of loads, fatigue in shear needs not to be verified.

Pure tension and tension-compression with $\sigma_{ct,\max} > 0.026|S_{td,\max}|$

$$\log N = 12(1 - S_{td,\max}) \qquad (7.4.9)$$

where:

$\sigma_{ct,\max}$ is the maximum tensile stress;

$S_{td,\max}$ is the maximum tensile stress level;

$S_{td,\max} = \gamma_{Ed}\sigma_{ct,\max}/f_{ctd.fat}$.

7.4.1.5 Level III of Approximation: verification by means of a spectrum of load levels

This method takes account of the required service life, the load spectrum (which is divided into *j* blocks) and the characteristic fatigue strength functions.

Fatigue damage *D* is calculated using the Palmgren-Miner summation

$$D = \sum_{i=1}^{j} \frac{n_{Si}}{N_{Ri}} \qquad (7.4\text{-}10)$$

where:

D is fatigue damage;

n_{Si} denotes the number of acting stress cycles associated with the stress range for steel and the actual stress levels for concrete;

N_{Ri} denotes the number of resisting stress cycles at a given stress level.

The fatigue requirement will be satisfied if $D \le D_{\lim.}$

Appropriate values for D_{lim} should be adopted for concrete. Under increasing stress levels, D_{lim} can be safely used.

7.4.1.6 Shear design

Members without shear reinforcement

The fatigue requirements will be met, if under cyclic loading the number of cycles corresponding to the required service life is smaller than or equal to the numbers of cycles to failure:

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Eq. (7.4-11) is also valid for beams and slabs in shear and punching.

According to Eq. (7.4-12), the strut inclination θ_{fat} is steeper than the value θ used for design under static loading. This takes account of the higher value of θ in the SLS.

The fatigue reference strength is to be reduced in the same way as the compressive strength of the concrete subjected to simultaneously acting compressive and transverse tensile forces.

$n \leq N$.

N should be calculated from the fatigue strength functions given below.

$$\log N = 10(1 - V_{max} / V_{ref}) \quad (7.4\text{-}11)$$

where:

V_{max} is the maximum shear force under the relevant representative values of permanent loads including prestress and maximum cyclic loading;

$V_{ref} = V_{Rd,c}$ (see subclause 7.3.3).

Members with shear reinforcement

The stress in the shear reinforcement should be calculated according to chapter 6 assuming the following inclination of the compression struts under fatigue loading:

$$\tan\theta_{fat} = \sqrt{\tan\theta} \quad (7.4\text{-}12)$$

where:

θ_{fat} is the angle between the web compression and the chords valid for verification of the reinforcement.

For assessment of the θ value, see subclause 7.3.3.

The resistance of compressive struts can be verified using Eq. (7.4-4) or Eqs. (7.4-7a) and (7.4-7b) reducing the fatigue reference strength given in subclause 7.4.1.3 by a factor of k_c according to Eq. (7.3-27). The compression of web concrete subjected to fatigue loading should be calculated using the angle θ (see subclause 7.3.3).

7.4.1.7 Increased deflections under fatigue loading in the SLS

Under cyclic loading, progressive deflection can occur in reinforced concrete members in addition to the deflection produced by creep. The cyclic effect can be calculated from

$$a_n = a_1[1.5 - 0.5 \exp(-0.03n^{0.25})] \quad (7.4\text{-}13)$$

where:

a_n final deflection after n cycles;

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a_1 deflection in the first cycle due to the maximum load including effects of shear strains;

n is the number of cycles.

7.4.2 Impact and explosion

7.4.2.1 General remarks

Loads caused by impact and explosions are characterized as high dynamic loads with strain rates which are significantly higher than those for example of traffic loads or earthquake loads.

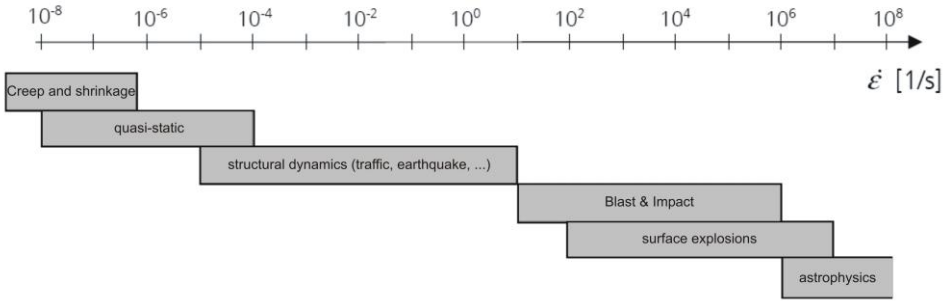


Figure 7.4-3: Strain rates for different loading velocities

High dynamic loads can be caused by:

- impact of vehicles, trains or airplanes;
- impact of projectiles and missiles;
- impact of debris;
- surface burst explosion;
- blast caused by near explosions;
- blast caused by far explosions;
- explosions in rooms, including multiple reflexions.

With increasing strain rates caused by loadings with high velocities, special effects occur which must be taken into account in the design process:

- mass effects;
- resonance effects;
- spalling and scabbing effects;
- punching.

Structures which are designed to resist high dynamic loads are for example:

- nuclear power plants;
- shelters for planes and ammunition;
- office buildings with a high degree of threat like embassies, military buildings, etc.;
- chemical factories;
- piers of bridges which can be hit by trains or vehicles.

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For strain rates lower than 10^{-1}, caused by traffic or earthquake, quasi-static loads can be used to model the additional load effects (level I).

For higher strain rates a dynamic calculation is required. For known phenomena like free air burst explosions or plane impact, idealized load-time-curves or pressure-time-curves are available in codes like UFC 3-340-02 or DIN 25449 which can be adapted to the respective situation.

7.4.2.2 Determination of design loads

Concerning safety factors, high dynamic loads usually belong to the extraordinary loads. For values of the safety factors reference is made to subclause 4.5.2.4. The characteristic loads strongly depend on the type and the intensity of the explosion or impact so that a general definition of the load is not possible. The following different approaches are possible:

- quasi-static equivalent loads (level I);
- standardized pressure-time curves (level II);
- individual load calculation (level III).

For some configurations loads can be obtained from literature or special codes (like UFC 3-340-02). In other cases numerical calculations with the use of hydrocodes, FE-codes or tests are necessary to get load data (level III).

Figure 7.4-4 and Figure 7.4-5 give examples for pressure-time-curves for loads caused by free air burst explosions.

For the determination of internal forces and stresses in the structure three levels of analysis are distinguished:

Level I

For quasi-static equivalent loads a linear or nonlinear static calculation can be done using the static model which is used for dead and live loads.

Level II

For time-pressure curves a linear or nonlinear dynamic calculation using time-step method is required.

Level III

For a detailed modelling of impact or explosion by hydrocode or FE-calculation the internal stresses can be obtained directly from this calculation. As an alternative pressure-time curves can be derived from the results of these calculations which are the basis for a calculation on level 2.

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The effect of a free air burst explosion can be modelled using a standard curve like in Figure 7.4-4. The load value and the time of duration depend on several factors like the distance between the explosive and the concrete structure and the type and the quantity of the explosive material.

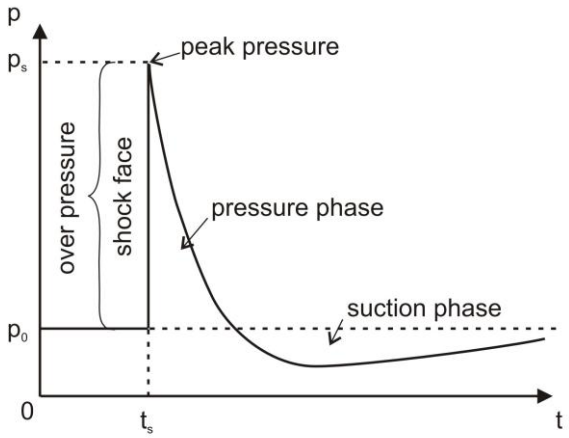


Figure 7.4-4: Principle pressure-time-curve caused by an air burst explosion

Figure 7.4-5 gives a more simplified pressure-time-function without taking account the suction phase which can be used for structures with a high weight and a high stiffness which are loaded by low pressures.

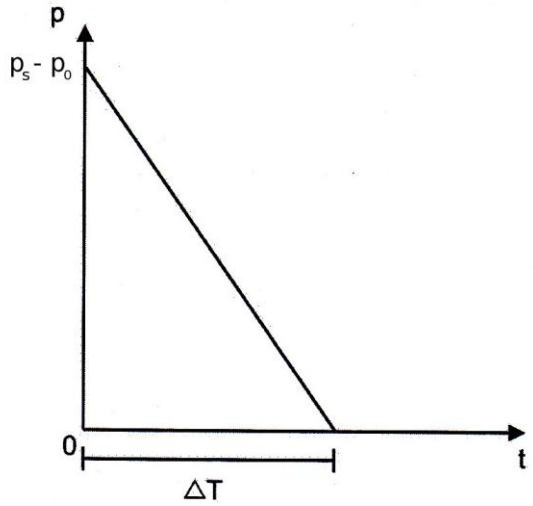


Figure 7.4-5: Free air burst explosion

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The calculation methods for the level 1 and level 2 calculations are similar to those for seismic design, see subclause 7.4.3

Usually very high stresses can occur caused by loading with high strain rates. However, also at the side of the structural resistance special effects can be observed which should be taken into account. The material strength of concrete and of steel under compression and under tension increase with increasing strain rates, see Figure 7.4-6 and Figure 7.4-7. A simplified approach for the increase of material strength is given in Table 7.4-3 and Table 7.4-4.

Table 7.4-3 includes different values for near and far detonations. One possibility to distinguish between the near and far detonations is given by Mayrhofer Chr., "*Grundlagen zu den Methoden der dynamischen Grenztragfähigkeitsberechnung bei terroristischen Ereignissen*", 2. Workshop Bau-Protect, Berichte aus dem konstruktiven Ingenieurbau 06/4, München, 2006.

$z = r/W^{1/3}$

where

7.4.2.3 Dimensioning for overall stresses

For the dimensioning of R/C-structures for overall stresses like bending moments, normal and shear forces, principally the same procedures like for seismic loads can be used. Three different approaches are possible:

- static dimensioning with regard to internal forces (subclause 7.4.2.3.1);
- dynamic dimensioning with regard to internal forces (7.4.2.3.2);
- dynamic dimensioning with regard to ductility (7.4.2.3.3).

7.4.2.3.1 Static dimensioning with regard to internal forces

For static dimensioning with regard to internal forces the following options are available:

- linear or nonlinear calculation of the static internal forces calculated on level 1;
- dimensioning of the structure with the usual methods for static loads;
- no use of the dynamic increase factors, i.e. the normal values for material strength of concrete and reinforcement have to be used.

Table 7.4-3: Dynamic increase factors (DIF) for the design of reinforced concrete elements

Type of stress	**Far design range**			**Close-in design range**		
	Reinforcing bars		Concrete	Reinforcing bars		Concrete
	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_c
Bending	1.17	1.05	1.19	1.23	1.05	1.25
Diagonal tension	1.00	–	1.00	1.10	1.00	1.00
Direct shear	1.10	1.00	1.10	1.10	1.00	1.10
Bond	1.17	1.05	1.00	1.23	1.05	1.00
Compression	1.10	–	1.12	1.13	–	1.16

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r is the distance between explosion epicenter and structure [m];

W is the weight of explosives as TNT equivalent [kg];

with

$z \leq 0.5$ → near explosion;

$z > 0.5$ → far explosion.

The calculation of large structures like nuclear power plants usually is done on level 2. In level 2 calculations the DIF's in Table 7.4-3 and Table 7.4.4 should be used because local strain rates are not known.

For prestressing steel, strands or wires, a dynamic increase factor should not be taken into account ($f_{dy}/f_y = 1$).

Table 7.4-4: Dynamic increase factors (DIF) for the design of reinforced concrete elements

Type of stress	**Type of reinforce-ment**	**Maximum support rotation, Θ_m** (degrees)	**Dynamic design stress**	
			Reinforce-ment f_{ds}	**Concrete,** f_{dc}
Bending	Tension and compression	$0 < \Theta_m \leq 2$ $2 < \Theta_m \leq 6$ $6 < \Theta_m \leq 12$	f_{dy} (1) $f_{dy} + (f_{du} - f_{dy})/4$ $(f_{dy} + f_{du})/2$	f'_{dc} (2) (2)
Diagonal tension	Stirrups	$0 < \Theta_m \leq 2$ $2 < \Theta_m \leq 6$ $6 < \Theta_m \leq 12$	f_{dy} f_{dy} f_{dy}	f'_{dc} f'_{dc} f'_{dc}
Diagonal tension	Lacing	$0 < \Theta_m \leq 2$ $2 < \Theta_m \leq 6$ $6 < \Theta_m \leq 12$	f_{dy} $f_{dy} + (f_{du} - f_{dy})/4$ $(f_{dy} + f_{du})/2$	f'_{dc} f'_{dc} f'_{dc}
Direct shear	Diagonal bars	$0 < \Theta_m \leq 2$ $2 < \Theta_m \leq 6$ $6 < \Theta_m \leq 12$	f_{dy} $f_{dy} + (f_{du} - f_{dy})/4$ $(f_{dy} + f_{du})/2$	f'_{dc} (3) (3)
Compression	Column	(4)	f_{dy}	f'_{dc}

(1) Tension reinforcement only.
(2) Concrete crushed and not effective in resisting moment.
(3) Concrete is considered not effective and shear is resisted by the reinforcement only.
(4) Capacity is not a function of support rotation.

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In the numerical calculation of the internal forces on level 3 the impact or the explosion or the impact process itself is part of the numerical calculation. So special codes are necessary by which the relevant phenomena like wave propagation, large deformations, fracture etc. can be modelled adequately. This can be done for example by hydrocodes or special Finite Element codes. On level 3 the structure is modelled in a very detailed way, local strain rates are available generally.

In Figure 7.4-6 and Figure 7.4-7 curves for the increase of the strength of concrete and of reinforcing steel are given in relation to the strain rates.

The behaviour of concrete in compression and tension is based on equations 5.1-115a and 5.1-115b, 5.1-117a and 5.1-117b respectively in subclause 5.1.11.2.

7.4.2.3.2 Dynamic dimensioning with regard to internal forces

With regard to dimensioning concrete structures for internal forces caused by dynamic loads the following considerations apply:

- the internal forces caused by dynamic loads (level 2, 3) have to be calculated using of a dynamic numerical calculation. The numerical material behaviour of reinforced concrete has to be considered in compression and tension;
- the reinforcement and the concrete in any cross-section can be dimensioned for bending with the same methods as used for static loading, considering the increase of strength of concrete and reinforcement steel according to Figure 7.4-6 and Figure 7.4-7, under high dynamic strain rates;
- the use of the dynamic increase factors for reinforcing bars is only possible if the steel has sufficient ductility, like for seismic requirements, see Table 7.4-3;
- the structure has to be dimensioned for bending with normal forces, shear and torsion for the global internal forces.

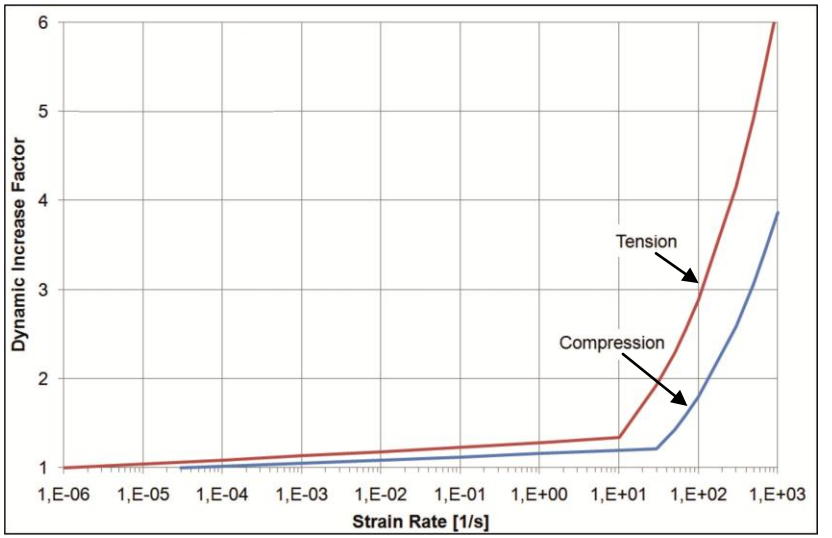


Figure 7.4-6: Increase of concrete strength under high strain rates for compression and tension in a semi-log format

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The equations in that describe the yield and ultimate stress of reinforcing steel used Figure 7.4-7 are listed below and can be found in Malvar L.J., Crawford J.E., "*Dynamic Increase Factor for Steel Reinforcing Bars*", 28. DDESB Seminar, Orlando, Florida, 1998.

$$DIF = (\dot{\varepsilon} / 10^{-4})^{\alpha}$$

where $\alpha_{fy} = 0.074 - 0.040\ (f_y/414)$;

$\alpha_{fu} = 0.019 - 0.009\ (f_y/414)$;

with f_y as static yield strength of reinforcing bar [MPa].

All of these data are derived from tests and can be used in calculations on level 3. More detailed information concerning stress and strain rate effects for concrete are given in subclause 5.1.11.2.

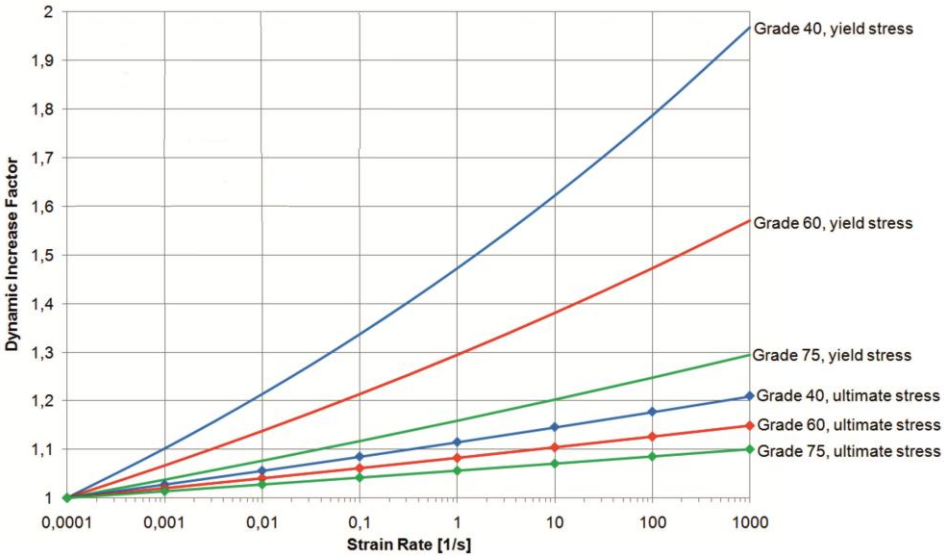


Figure 7.4-7: Increase of reinforcing steel strength under high strain rates

7.4.2.3.3 Dynamic dimensioning with regard to ductility

For dynamic dimensioning with regard to ductility the following considerations apply:

- The deflections caused by dynamic loads (level 2, 3) have to be calculated by use of a dynamic numerical calculation. The material behaviour of reinforced concrete has to be considered in compression and tension.
- As for earthquake design, the plastic rotations are limited (Table 7.4-4).
- The dynamic design stresses are limited to the values given in Table 7.4-4. With these parameters the load-deflection behaviour can be determined for each cross section, for example to establish moment-curvature diagrams.

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In the design phase the engineer has several options to reduce the high dynamic loads. If there is sufficient space this can be done by increasing the distance between the structure and the threat or by barriers. If this is not possible the materials have to be selected and the structure has to be designed for the dynamic loads concerned.

There are two general options for design concerning loads with high strain rates when failure should be taken into account. One possibility is to define alternative load paths in the structure. This leads to a safe design only if the quantity of the loads can be calculated with sufficient accuracy. The alternative solution is the use of defined breaking points or areas in the structure to make sure that no progressive collapse can occur. In this case local damage is accepted but the collapse of the total structure has to be avoided.

Generally it is important that the cross-sections as well as the complete structure have a high ductility. This concerns the choice of the material, the layout of the connections and of the cross-sections as well.

Strengthening of existing structures can be done by the same methods as used in seismic design.

7.4.2.4 Structural detailing

7.4.2.4.1 Possibilities for a reduction of the loads in the design phase

In the design stage also other measures could be considered to reduce the danger of structural failure due to dynamic loads, such as:

- measures to secure sufficient distance between structure and threat;
- security walls for a reduction of pressure;
- sandwich-cross-sections with hard and soft layers to get a good dissipation of energy, adapted to the special case of loading;
- symmetric design of the lateral load resisting elements in plane like for seismic design to avoid high local stresses.

7.4.2.4.2 General recommendations for detailing

With regard to detailing the following possibilities may be considered:

- use of concrete with reduced Young’s modulus;
- use of concrete with high ductility;
- generate alternative load paths and redundant structural systems;
- provide minimum dimensions;
- ensure high ductility of cross-sections and connections;
- use steel quality as required for seismic design;
- use of adequate confinement to secure the cross sections.

7.4.2.4.3 Strengthening of existing reinforced concrete structures

The following methods for strengthening existing structures may be considered:

- increase of strength by implicating external reinforcement, e.g. carbon retrofitting;
- application of sandwich elements for energy dissipation;
- limitation of debris by i.e. an additional layer of textile reinforced concrete.

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Under certain conditions linear elastic analysis may be used for the determination of the deformations due to the seismic action, but not of the internal forces (see subclause 7.4.3.2.2).

7.4.3 Seismic design

7.4.3.1 Format of the verifications

Seismic actions impose deformations on structures. These deformations produce in turn internal forces and moments.

Action effects, S_{Ed}, in seismic situations and the corresponding resistances, R_d, for the verification of the ULS of resistance according to subclause 4.5.2.2 are normally deformations for ductile modes of behaviour and failure (e.g. for flexure with limited axial load), or internal forces for brittle ones (notably, for shear):

– deformation action effects, S_{Ed}, are determined through nonlinear analysis according to 7.4.3.2.4, or – under certain conditions – via linear elastic analysis according to 7.4.3.2.2.

– deformation resistances are normally plastic chord rotations at member ends, established according to 7.4.3.4.

– shear force action effects, S_{Ed}, are determined through nonlinear analysis according to 7.4.3.2.4, or by capacity design calculations according to 7.4.3.2.3.

– shear force resistances, R_d, are determined according to clause 7.3.3, with the special rules of subclause 7.4.3.5 applying in flexural plastic hinges.

For the verification of the SLS of deformations in seismic situations according to subclause 4.5.2.5, a is normally the chord rotation at a member end and is verified according to subclause 7.4.3.6.

7.4.3.2 Determination of seismic action effects through analysis

Nonlinear methods of analysis should normally be used for seismic actions, taking into account the dynamic response of the structure and representing the action through the system of structural deformations or inertia loads it produces.

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As the seismic action induces large inelastic rotation demands to beam ends, the effective slab width on either side of the web at the ends of T-beams in Figure 7.2-2, $b_{eff,i}$, is taken as the lesser of 25% of the beam span and of the mid-distance to the adjacent parallel beam, b_i. All slab bars which are parallel to such a beam and fall within this width are considered as fully effective as longitudinal reinforcement of the beam's end section.

Seismic deformation demands are underestimated, if sources of flexibility are neglected and stiffness is overestimated. To this end, tension stiffening is neglected, as it diminishes with load cycling. Slippage of the tension bars from their anchorage in joints or foundation elements increases the member's apparent flexibility, owing to the fixed-end-rotation of the end section it produces:

$$\theta_{slip} = \frac{\varphi d_{bL} \sigma_s}{8\tau_b} \tag{7.4-13}$$

where:

φ is the curvature at the end section;

σ_s is the stress of the tension bars at the end section;

d_{bL} is the mean diameter of the tension bars;

τ_b is the mean bond stress along the straight anchorage length of the tension bars outside the member length.

At yielding of the end section φ and σ_s may be taken equal to their yield values, φ_y and f_y, while for ribbed bars τ_b (in MPa) may be taken equal to $\sqrt{f_c}$(MPa), giving a value of θ_{slip} denoted as $\theta_{slip,y}$. The increase in the member's apparent flexibility due to this fixed-end-rotation is equivalent to adding a rotational spring at the end section, with stiffness $M_y/\theta_{slip,y}$, where M_y is the yield moment there (proportional to φ_y).

At the background of seismic design based on controlled inelastic response stands a bilinear force-deformation law of the lateral-load-resisting system as a whole and of its individual components in primary (monotonic) loading. The effective stiffness to the yield-point should therefore be used as

7.4.3.2.1 Modelling

The model of the structure should represent the distribution of its stiffness, mass and resistance, so that all significant deformation shapes and inertia forces are properly accounted for.

For both the ULS and the SLS the elastic stiffness should be based on fully cracked sections of those members which are expected to yield in the limit state of interest, without tension stiffening. The fixed-end-rotation of the member's end section due to slippage of longitudinal bars from their anchorage zone outside the member length should also be taken into account.

The secant stiffness to the yield-point should be used as effective elastic stiffness of members which yield at the limit state of interest.

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element elastic stiffness. The default damping of 5% of critical used in linear elastic seismic analysis of concrete structures reflects hysteretic energy dissipation in load cycles up to element yielding.

The chord rotation at the end of a one-dimensional member, θ, is the angle between the normal to the end section and the chord connecting the member ends at the member's displaced position. It includes flexural and shear deformations along the member, as well as the fixed-end-rotation of the end section due to slippage of longitudinal bars from their anchorage zone outside the member length. In the linear elastic regime the member deformations that determine – together with any loading between the ends – the bending moments and shears along the member are the chord rotations at the two member ends.

Estimation of EI_{eff} before the analysis for the seismic action may be based on presumed values of L_s equal to 50% of the clear length of beams between columns and of columns between beams in the plane of bending, as well as of bridge piers fixed against rotation by the deck in the plane of bending. For the strong direction of walls in buildings, the value of L_s within a storey may be taken equal to 50% of the height from the wall's base section in that storey to the top of the wall in the building. In members cantilevering within the plane of bending L_s is the member clear length.

An average value of EI_{eff} at: (a) the two ends where it frames into other members in the plane of bending and (b) for positive and negative moments may be used for a member.

Application of Eq. (7.4-14) presumes that the longitudinal reinforcement of the end sections is known. In new structures this may entail dimensioning first this reinforcement from the non-seismic situations and minimum reinforcement and estimating then how much it may increase for the verifications in the seismic situation. As the value of EI_{eff} depends weakly on the amount of longitudinal reinforcement, use may also be made of empirical expressions giving the ratio of EI_{eff} of different types of members to the uncracked gross section stiffness as a function of known parameters (e.g. the shear-span-to-depth-ratio at the end of the member, the mean axial stress, N/A_c, the ratio of mean bar diameter to the section depth, etc.).

For one-dimensional concrete members (including slender walls) the secant stiffness to the yield-point may be estimated as:

$$EI_{eff} = \frac{M_y L_s}{3\theta_y} \qquad (7.4\text{-}14)$$

where:

M_y is the yield moment;

θ_y is the chord rotation at the yielding member end;

$L_s = M/V$ is the shear span (moment-to-shear-ratio) at that end section in the seismic situation.

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45°-cracking near the member end precedes flexural yielding if the value of $V_{Rd,c}$ is less than the shear force at flexural yielding, M_y/L_s.

The shear deformation of the member at the time it yields in flexural is fairly small, in the order of:

$0.0014(1+1.5h/L_s)$ for beams or rectangular columns;

0.0013 for walls and members with hollow rectangular section;

$0.0022 \max[0; 1-L_s/6D]$ for circular piers or columns.

Eq. (7.4-14) applied with so-computed values of θ_y and with the prestress taken as part of the actions, underestimates, in general, the effective stiffness of prestressed components with bonded tendons. Hence, its use is safe-sided in the context of displacement-based seismic design or assessment.

If a rotational spring with stiffness $M_y/\theta_{slip,y}$ is added at the member end to account for the apparent increase in member flexibility due to this fixed-end-rotation, then the term $\theta_{slip,y}$ is not included in θ_y.

This is the "equal displacement rule" at the level of member deformations. For a single-degree-of-freedom oscillator with elasto-plastic force-deformation law in monotonic loading the "rule" states that the maximum displacement in the inelastic seismic response is about the same as that of an the elastic oscillator with the same mass, damping and elastic stiffness.

Concrete structures have fundamental periods in the range where the "equal displacement" rule gives fairly accurate results for an "equivalent" single-degree-of-freedom oscillator and applies well on average for member deformations (albeit with considerable deviations for individual members).

The end section of a strong column framing into weaker beams, or of a strong beam framing into weaker columns, does not yield in the seismic situation, even when their M_E/M_R-ratio exceeds 1.0. Except at such sections, the ratio M_E/M_R is about equal to the corresponding ductility ratio demand for the chord rotation at the end of the member (peak inelastic chord rotation demand divided by the corresponding value at yielding of the end section).

θ_y is the sum of:

- a flexural component, equal to $\varphi_y(L_s+z)/3$ if 45°-cracking of the member precedes flexural yielding of its end section, or to $\varphi_y L_s/3$ if it doesn't;
- a shear deformation, which normally increases with decreasing member shear-span-to-depth ratio, L_s/h; and
- the fixed-end-rotation due to slippage of longitudinal bars from their anchorage zone outside the member length (the value at yielding of the end section, $\theta_{slip,y}$).

7.4.3.2.2 Linear elastic analysis for the calculation of seismic deformation demands (action effects)

Member inelastic flexural deformations due to the seismic action may be determined through linear elastic analysis with damping 5% of critical, under the condition that they are not concentrated at certain parts of the structure (notably at one side of a building in plan, or in one or few building storeys or bridge piers) but are distributed fairly uniformly throughout the structure.

Applicability of linear elastic analysis for the estimation of inelastic flexural deformations may be checked by inspection of the distribution over the structure of the ratio of the moment from the linear analysis at member end sections, M_E, to the corresponding moment resistance, M_R.

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The directions of the two orthogonal horizontal components should be chosen along two main directions of the structural layout in plan:

– in bridges: along the "longitudinal" direction (connecting the two points on the deck axis at the abutments) and the orthogonal, "transverse" one;
– in buildings or similar structures: along directions of near-symmetry, or to which a large number of plane frames and/or walls are aligned, etc.

The response-history analyses may be carried out either simultaneously for all seismic action components of interest, or separately for each one and the results superposed.

The resultant of the "equivalent" forces in the direction of the seismic action component, V_b, is determined as:

$V_b = m_{eff,1} S_a(T_1)$

where:

$S_a(T_1)$ is the value of the elastic response spectrum at period T_1;

$m_{eff,1}$ is an estimate of the effective modal mass of the dominant mode.

It is safe-sided to take $m_{eff,1}$ as equal to the total vibrating mass, m. In the horizontal direction of a building-type structure, m is the total mass above the foundation or the top of a rigid basement. In the horizontal direction of a bridge, it is the total mass of the deck and of the upper half of the pier height in the relevant part of the bridge. In the vertical direction of a long component on relatively rigid supports, having significant mass distributed along its span, m is the total mass of the component.

The pertinent moment-deformation relations should be used to calculate moments at member ends from inelastic flexural deformations estimated via linear elastic analysis.

The maximum values of member inelastic flexural deformations due to the relevant components of the seismic action may be estimated using one of following methods of linear elastic analysis:

1. Response-history dynamic analysis with damping 5% of critical.

2. Modal response spectrum analysis, using the 5%-damped elastic response spectrum:

   The number of normal modes should be sufficient to capture with their effective modal masses at least 90% of the total vibrating mass in each direction a seismic action component is applied.

   Peak modal responses for any deformation measure of interest should be combined via the Complete-Quadratic-Combination (CQC) rule.

Modal response spectrum analysis may be simplified into separate linear static analyses under "equivalent" forces in the direction of each one of the relevant seismic action components, with the structure considered as an "equivalent" single-degree-of-freedom system having the period of the normal mode, T_1, which has the largest modal mass in that direction.

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If T_1 is shorter than one-half of the period that marks the applicability limit of the simplification of modal analysis as "equivalent static" (see condition iii below), it is allowed to take: $m_{eff,1} = 0.85m$ for:

– the horizontal direction of buildings with more than two storeys above the foundation or the top of a rigid basement;
– the transverse direction of bridges with continuous deck laterally restrained at the abutments (except when the transverse stiffness of piers is large, or it has large differences between adjacent piers or decreases from the abutments to deck mid-length).

The simplification cannot be applied to only one of the two horizontal seismic action components, but it may be applied to the vertical alone.

Condition (i) may be considered to be met if that mode's effective modal mass in the direction of interest accounts for at least 75% of the total.

Condition (ii) may be considered to be met:

– in building-type structures, in the horizontal direction, if:
  (a) the storey mass and lateral stiffness are either constant in all storeys, or decrease gradually and smoothly from the base to the top, and
  (b) no lateral-load-resisting sub-system (frame, wall, etc.) is vertically interrupted before the top of the corresponding part of the structure, and
  (c) any setbacks of each side are limited (e.g. at each storey to 10% of the parallel dimension of the one below and in total to one-third of the building's parallel dimension at the base).
    – The shape of the single mode considered may then be taken proportional to elevation above the top of the foundation or of a rigid basement.
– in bridges with piers having total mass much less (e.g. ≤ 20%) than the deck:
  (a) in any horizontal direction, if lateral stiffness is provided only by piers that are not coupled through the deck (e.g. if spans are simply-supported) and may be considered as structurally independent;

The simplification of modal response spectrum analysis into "equivalent static" analyses in the direction of the seismic action component is allowed, if all of the following conditions are fulfilled:

i. The response to the relevant component is dominated by the normal mode of vibration having the largest effective modal mass in that direction.
ii. It is possible to identify in good approximation the shape of the single normal mode taken into account.

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(b) in the longitudinal direction, if the deck is continuous, almost straight and not restrained in that direction at the abutments;

(c) in the transverse direction, if the deck is continuous and approximately straight, unless the transverse stiffness of piers is of the same order as that of the abutments, or exhibits large differences between adjacent piers, or decreases from the abutments to deck mid-length.

– In cases (a) and (b) the relevant part of the deck is considered to have the same translation in the horizontal direction of interest. In case (c) the mode shape is taken proportional to the elastic displacements due to the gravity loads applied in the horizontal direction of interest.

– in the vertical direction of long components supported on relatively rigid supports and having significant mass distributed along the span; in that case the vertical mode is taken proportional to the component's elastic deflection due to gravity loads.

T_1 is usually the longest among all normal modes with significant effective modal mass in the direction of the seismic action component in question ("fundamental period"). As spectral displacements increase until a value T_D of the period, above which they stay about constant or fall with increasing period T, condition (iii) may be considered as met, if:

$T_1 \leq \min (T_D; 4T_C)$

where:

T_C is the period at which spectral accelerations attain their largest value (normally spectral velocities are about constant between T_C and T_D).

The Rayleigh Quotient estimates T_1 quite accurately, using the displaced shape of the structure from the very "equivalent static" analysis under the force distribution used there (forces proportional to mass times the assumed modal shape).

iii. Response spectral displacements at T_1 are much larger than at the period of any other mode with significant effective modal mass in the direction of the seismic action component.

The "equivalent static" analysis is carried out under a force distribution over the structure (or its relevant part) proportional to the product of mass and the shape of the single normal mode considered. The value of T_1 is estimated on the basis of mechanics, possibly using results of such an analysis.

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No matter whether a full modal response spectrum analysis or its "equivalent static" simplification is applied, peak absolute values of seismic action effects from (concurrent) seismic action components in X, Y, Z are determined via the SRSS rule from the peak values computed for separate application of these components. These peak absolute values should be combined with other action effects with a plus or minus sign.

If modal response spectrum analysis is used, the combination of modal contributions through the CQC rule and of E_X, E_Y (and E_Z) via Eq. (7.4-14) can be done in a single modal response spectrum analysis covering all relevant seismic action components. Such an analysis gives the expected value of peak seismic action effects under concurrent statistically independent seismic action components along X, Y (and Z). Moreover, this value of E is independent of the choice of the horizontal directions X and Y. Values of E_Z from "equivalent static" analysis for the vertical component are still combined via Eq. (7.4-15) with the outcome of the combination of E_X, E_Y within a single modal response spectrum analysis for the two horizontal components X and Y.

Probabilistic models give the values and signs of other action effects (e.g. the column deformation in the orthogonal direction) expected to take place simultaneously with the maximum value of the action effect obtained via the SRSS rule.

If M_R denotes a moment resistance component with vector normal to the shear force of interest, index *b* is used for beams, *c* for columns, *w* for walls and l_{cl} is the clear length of a beam, column or bridge pier, the capacity-design shears are:

Peak values of seismic action effects (i.e., of deformations in this case), E_X, E_Y, E_Z, due to separate application of the seismic action components in directions X, Y, Z, should be combined through the Square Root of the Sum of Squares (SRSS) rule:

$$E = \pm\sqrt{E_X^2 + E_Y^2 + E_Z^2} \tag{7.4-15}$$

7.4.3.2.3 Capacity design for shear forces when linear elastic analysis is used for the estimation of deformation seismic action effects

In capacity design the maximum force demands in brittle components or modes of behaviour or failure are estimated from equilibrium and from the moments delivered to the component in question at its connections to the others. Each of these moments is taken equal to the minimum of:

(a) the moment component with vector normal to the shear force of interest from the analysis, M_E;

(b) the moment resistances at plastic hinge formation – multiplied by $\gamma_{Rd} \geq 1.0$ for overstrengths not explicitly modelled (e.g. due to steel strain hardening) – with plastic hinges assumed to form at those two opposite faces of a joint between the weaker elements.

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If M_R denotes a moment resistance component with vector normal to the shear force of interest, index *b* is used for beams, *c* for columns, *w* for walls and l_{cl} is the clear length of a beam, column or bridge pier, the capacity-design shears are:

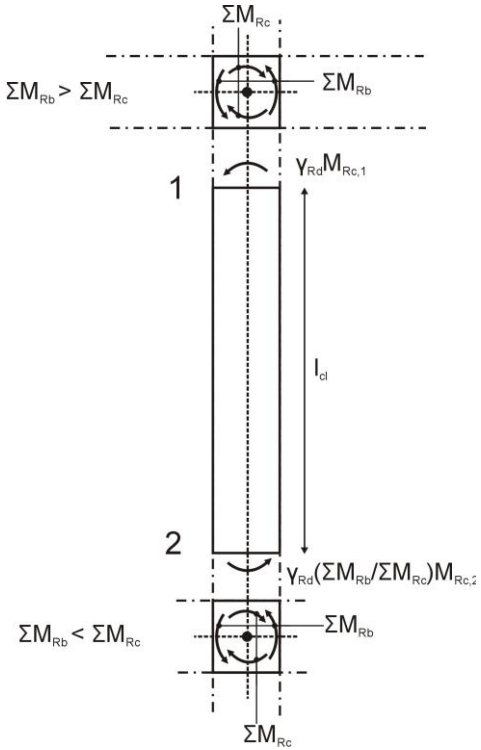


Figure 7.4-8 End moments for capacity design shear in a frame column

In capacity design the maximum force demands in brittle components or modes of behaviour or failure are estimated from equilibrium and from the moments delivered to the component in question at its connections to the others. Each of these moments is taken equal to the minimum of:

(c) the moment component with vector normal to the shear force of interest from the analysis, M_E;

(d) the moment resistances at plastic hinge formation – multiplied by $\gamma_{Rd} \geq 1.0$ for overstrengths not explicitly modelled (e.g. due to steel strain hardening) – with plastic hinges assumed to form at those two opposite faces of a joint between the weaker elements.

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(1) Between ends 1 and 2 of a frame column, bridge pier or similar member:

$$V_{CD} = \frac{\min\left[M_{E,c1};\ \gamma_{Rd} M_{R,c1} \min\left(1;\ \frac{\sum M_{R,b}}{\sum M_{R,c}}\right)_1\right] + \min\left[M_{E,c2};\ \gamma_{Rd} M_{R,c2} \min\left(1;\ \frac{\sum M_{R,b}}{\sum M_{R,c}}\right)_2\right]}{l_{cl}} \quad (7.4\text{-}16)$$

where:

M_E and M_R enter with positive values;

ΣM_{Rc} or ΣM_{Rb} are the sums of moment resistances of the columns or the beams framing into the joint, respectively;

At the connections with the foundation or of a pier with a deck integral with it, it is always: $\Sigma M_{Rb}/\Sigma M_{Rc} > 1$;

For convenience, the moment resistances of columns or piers, M_{Rc}, are computed for the axial force due to gravity loads alone.

(2) At distance x from end section i of a frame beam (the other end denoted by j):

$$V_{CD}(x) = V_{g+\psi 2q,o}(x) + \frac{\min\left[M_{E,bi}^{-};\ \gamma_{Rd} M_{R,bi}^{-} \min\left(1;\ \frac{\sum M_{R,c}}{\sum M_{R,b}}\right)_i\right] + \min\left[M_{E,bj}^{+};\ \gamma_{Rd} M_{R,bj}^{+} \min\left(1;\ \frac{\sum M_{R,c}}{\sum M_{R,b}}\right)_j\right]}{l_{cl}} \quad (7.4\text{-}17)$$

where:

all moments enter as positive, but index (-) is used for moments inducing tension at the top flange of a beam (hogging ones) and (+) for those inducing tension at the bottom (sagging);

$V_{g+\psi 2q,o}(x)$ is the shear force at section x due to the quasi-permanent gravity loads $g+\psi_2 q$, with the beam considered as simply supported (index: o).

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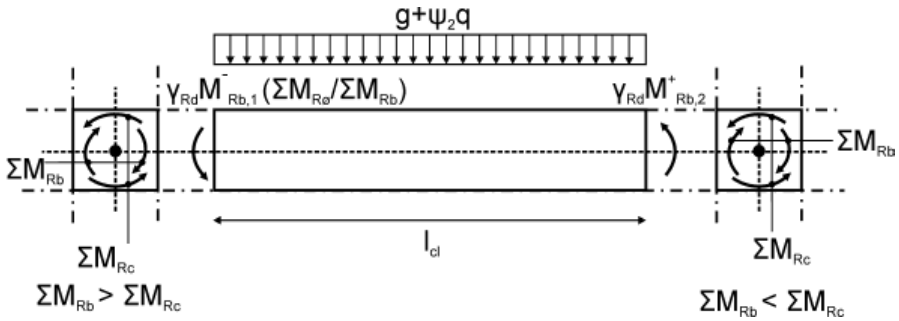


Figure 7.4-9 End moments for capacity design shear in a frame beam

(3) In a joint where a horizontal element (a beam – including a foundation beam – a bridge deck, etc.) and a vertical one (frame column, structural wall, bridge pier, etc) frame into each other:

The nominal shear stress in the joint core, $v_{j,}$ is the same, regardless of whether it is derived:

from the horizontal shear force in the joint, V_{jh}, as: $v_j = \dfrac{V_{jh}}{b_j h_{jc}}$ (7.4-18)

or

from the vertical shear force in the joint, V_{jv}, as: $v_j = \dfrac{V_{jv}}{b_j h_{bj}}$ (7.4-19)

where:

b_j is effective joint width in the orthogonal horizontal direction;

h_{jc} is horizontal distance between the outermost layers of vertical reinforcement in the joint in the direction of V_{jh}; and

h_{bj} is clear depth between the top and bottom reinforcement of the joint.

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If $\sum M_{Rb} < \sum M_{Rc}$, $v_{j,}$ is controlled by the horizontal shear force in the joint:

$$V_{jh} = \sum M_{Rb}\left(\frac{1}{h_{bj}} - \frac{1}{h_{st}}\frac{L_b}{L_{bn}}\right) \qquad (7.4\text{-}20)$$

Otherwise, i.e., if $\sum M_{Rb} > \sum M_{Rc}$, it is governed by the vertical shear force:

$$V_{jv} \approx \sum M_{Rc}\left(\frac{1}{h_{cj}} - \frac{1}{L_b}\frac{h_{st}}{h_{st,n}}\right) + \frac{1}{2}\left|\left[V_{g+\psi 2q,b}\right]_l - \left[V_{g+\psi 2q,b}\right]_r\right| \qquad (7.4\text{-}21)$$

where:

h_{st}, $h_{st,n}$ are theoretical and clear height of vertical element, respectively;

L_b, L_{bn} are theoretical and clear span, respectively, of horizontal element (beam or deck); and

$V_{g+\psi 2q,b}$ is shear force due to quasi-permanent actions at the end of the horizontal element on the left (index: l) or on the right (index: r) side of the joint.

If the moments from the analysis, M_{Eb}, M_{Ec}, are such that $\sum M_{Eb} < \sum M_{Rb}$ and $\sum M_{Ec} < \sum M_{Rc}$, v_j may be estimated either from:

– the horizontal shear force in the joint, V_{jh}, from Eq. (7.4-20) using there $\sum M_{Eb}$ instead of $\sum M_{Rb}$;

or

– the vertical shear force, V_{jv}, from Eq. (7.4-21) using there $\sum M_{Ec}$ instead of $\sum M_{Rc}$.

(4) At elevation z above the base (top of foundation or of a rigid basement) of a structural wall in a building:

Capacity-design shears along the height of a multi-storey wall cannot be established only from equilibrium and the moment capacities of the wall at the base and of beams framing into it at floor levels. They may be conveniently estimated, instead, as follows:

Whenever capacity-design effects cannot be determined solely from the moment capacities at plastic hinge formation on the basis of equilibrium, they may be found assuming that the seismic action effects at the instant the moment capacities at plastic hinge are reached are proportional to the corresponding outcomes of the linear elastic analysis for the seismic situation.

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For "squat" walls (with height-to-length ratio $h_w/l_w \leq 2$):

$$V_{CD,w}(z) = \min\left(1; \frac{M_{R,w}(z=0)}{M_{E,w}(z=0)}\right) V_{Ed,w}(z) \qquad (7.4\text{-}22)$$

For "slender" walls ($h_w/l_w > 2$) with $M_{E,w}(z) > M_{R,w}(z=0)$, taking also into account higher mode effects on the shears of slender walls after yielding at the base:

$$V_{CD,w}(z) = \sqrt{\min\left(1; \frac{M_{R,w}(z=0)}{M_{E,w}(z=0)}\right)^2 + 0.1\left(\frac{\max S_a}{S_a(T_1)}\right)^2}\, V_{Ed,w}(z) \qquad (7.4\text{-}23)$$

where:

$M_{E,w}(z)$, $V_{E,w}(z)$ are moment and shear from the linear analysis for the seismic action of interest and the concurrent quasi-permanent actions;

$M_{R,w}(z=0)$ is moment resistance of the base section for the value of axial load due to the quasi-permanent gravity loads alone;

$\max S_a$ is maximum value of elastic acceleration response spectrum of the seismic action; and

$S_a(T_1)$ is value of elastic acceleration response spectrum at the fundamental period T_1 in the horizontal direction (closest to that) of the wall shear force.

(5) At the interface of foundation elements and the ground:

All linear analysis results for the seismic action effects transferred to the ground are multiplied by a factor a_{CD}, as follows:

(a) For individual footings:

$$a_{CD} = \min\left[\left(1; \frac{M_{R,y}}{M_{E,y}}\right); \left(1; \frac{M_{R,z}}{M_{E,z}}\right)\right] \qquad (7.4\text{-}24)$$

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where:

$M_{E,y}$, $M_{E,z}$ are the two orthogonal moment components at the base of the vertical element from linear elastic analysis for the seismic action of interest and the quasi-permanent gravity loads;

$M_{R,y}$, $M_{R,z}$ are uniaxial moment resistances at the base of the vertical element for the value of axial load due to the quasi-permanent actions alone.

(b) For $N > 1$ vertical elements with a common foundation (a foundation beam, a pile cap, a box-type foundation, a raft, etc.):

$$a_{CD} = \frac{\sum_{i=1}^{N} a_{CD,i} M_{E,i}}{\sum_{i=1}^{N} M_{E,i}} \tag{7.4-25}$$

where:

$a_{CD,i}$ is value of a_{CD} for individual vertical element i;

$M_{E,i}$ is that value among the two moment components $M_{E,y}$, $M_{E,z}$ from linear analysis for the seismic action of interest and the quasi-permanent actions which gives the minimum ratio $M_{R,y}/M_{E,y}$ or $M_{R,z}/M_{E,z}$ at the base of vertical element i and governs plastic hinging there.

(6) In brittle or sensitive components designed to remain elastic after plastic hinging of vertical supports (notably, for the deck, any fixed bearings, any shock transmission units, any abutments flexibly connected to the deck, any seismic links consisting of shear keys, buffers and/or linkage bolts or cables, etc., of bridges with plastic hinging in piers).

Linear analysis results for the seismic action effects in brittle or sensitive components are multiplied by a factor applying for the entire structure under the seismic action direction of interest:

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$$a_{CD} = \frac{\sum V_{CD,i}}{\sum V_{E,i}} \qquad (7.4\text{-}26)$$

where:

i is index that refers to vertical supporting elements that yield in the seismic situation considered and summations extend over all of them;

$V_{E,i}$ is seismic shear force from linear analysis;

$V_{CD,i}$ is capacity design shear in the direction of $V_{E,i}$ calculated as in case 1 above (see Eq. (7.4-16)).

Unlike linear elastic analysis, which may be relied upon – under certain conditions – to estimate seismic deformations but not internal forces, nonlinear analysis may be used to determine both types of seismic action effects.

When bending is mainly in a single plane, it may be sufficient to use for each member end a moment-(chord) rotation model within that plane. The effect of significant variation of the axial load during the response (e.g. in the exterior columns of tall frames) on the moment-rotation behaviour should be taken into account. Simplified treatment of the coupling between the two directions of bending in vertical elements of three-dimensional models is normally possible.

In the end, all components (including those of an existing structure being retrofitted for earthquake resistance) are normally verified to remain elastic if they are brittle, or to have some margin against their ultimate deformation – after which the drop in resistance is significant – if ductile. So, the reduction in resistance after ultimate strength may be neglected and the force-deformation relation in primary loading taken bilinear, especially if the initial post-yield strain-hardening is also neglected, with the full post-yield behaviour taken perfectly plastic. It is on such a straight post-yield branch that the limit deformation is verified, with corrective measures taken if this limit is exceeded.

7.4.3.2.4 Nonlinear analysis for seismic action

Nonlinear dynamic analysis, with solution of the equations of motion in the time-domain, is the reference analysis method for seismic actions. The action should be specified as a suite of independent seismic events in terms of time-histories of the ground motion components (see subclause 4.5.1.4).

Nonlinear models should be employed, at least for those components expected to enter the inelastic regime in the seismic situation of interest.

One-dimensional members (including slender walls) designed or retrofitted for earthquake resistance will have all their inelasticity concentrated in flexural plastic hinges at their ends, where they are connected with other components. As a minimum, the nonlinear model of such a member should employ a force-deformation relation for these end regions – notably, one between the end moment and the corresponding (chord) rotation.

As a minimum, nonlinear models for components should use a bilinear force-deformation law in primary (monotonic) loading. Positive post-yield stiffness (due to strain-hardening) may be neglected and elastic-perfectly plastic behaviour assumed instead. Strong post-yield softening should be taken into account through negative post-yield stiffness.

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Bilinear unloading-reloading parallel to the elastic and post-yield branches in monotonic loading is characteristic of steel but not of structural concrete, producing unrealistically large hysteretic damping.

When part of the deformation is due to bond-slip (e.g. from a joint) or to the effects of shear (as for example in members with low shear-span-to-depth ratio), hysteresis loops are "pinched" in the form of an inverted S and hysteretic energy dissipation is reduced.

Hysteresis rules play also an important role for the estimation of residual deformations of members (for local damage) or of the structure as a whole (e.g. the permanent tilt) after the earthquake.

Under seismic loading material failure at the local level (even rupture of a tension bar) does not constitute by itself member failure. The plastic hinge fails owing to local material failures gradually accumulating during cycling of the imposed deformations.

The dependence of the plastic rotation capacity on each basic variable (material property or geometric dimensions) is not always monotonic and does not lend itself to application of partial factors on these variables.

The plastic part of the chord rotation at a member end is essentially the same as the plastic hinge rotation there, plus the post-yield part of the fixed-end-rotation, θ_{slip}, due to slippage of longitudinal bars from their anchorage zone outside the member length.

The force-deformation law in primary loading should be supplemented with unloading-reloading ("hysteresis") rules, realistically reflecting the amount of post-yield hysteretic energy dissipation and the reduction of unloading and reloading stiffness with increasing peak deformation of a cycle ("stiffness degradation") which characterizes concrete components. The degradation of resistance with load cycling should be included if it is significant (e.g. in brittle or poorly detailed components).

The elastic stiffness of components and the viscous damping associated with the elastic regime should be the same as in linear elastic analysis (i.e. from subclause 7.4.3.2.1 and 5%, respectively).

7.4.3.3 ULS verifications of inelastic flexural deformations

In seismic situations the verification at the ULS of one-dimensional members (including slender walls) takes place in terms of inelastic flexural deformations in plastic hinges forming at member ends.

Deformation measures used in the verifications should represent the behaviour of the plastic hinge region as a whole.

The global safety format in clause 4.6 is appropriate for the ULS verification of a plastic hinge region, considered as an element.

An appropriate deformation measure for the plastic hinge region is the plastic part of the chord rotation at the member end, θ_{pl}.

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Under cyclic loading $\theta^{pl}{}_u$ is the value of θ_{pl} beyond which an increase in imposed deformation cannot increase the moment resistance above 80% of its maximum ever value.

The ultimate value of θ_{pl}, denoted as $\theta^{pl}{}_u$, is conventionally identified with a 20% drop in moment resistance.

The characteristic value (lower-5% fractile) of $\theta^{pl}{}_u$ is obtained from its mean value, $\theta^{pl}{}_{u,m}$, as:

$$\theta^{pl}{}_{u,k} = \theta^{pl}{}_{u,m} / \gamma_{Rd} \qquad (7.4\text{-}27)$$

where the safety factor γ_{Rd} accounts for model uncertainty.

Resistance values that enter the verifications are obtained by dividing the characteristic value of the resistance, $\theta^{pl}{}_{u,k}$, by a global safety factor $\gamma_R{}^*$, which depends on the ULS being verified:

– for the life safety limit state: $\gamma_R{}^* = 1.35$;
– for the near collapse (NC) limit state: $\gamma_R{}^* = 1.0$.

7.4.3.4 Cyclic plastic chord rotation capacity

The expected (mean) value of the plastic part of the ultimate chord rotation at a member end may be estimated as:

$$\theta_{u,m}^{pl} = (\varphi_u - \varphi_y) L_{pl} \left(1 - \frac{L_{pl}}{2L_s}\right) + \Delta\theta_{slip,u-y} \qquad (7.4\text{-}28)$$

where:

φ_u and φ_y are the ultimate and the yield curvature, respectively, of the end section;

L_s is the shear span (M/V ratio) at the member end;

L_{pl} is the "plastic hinge length";

$\Delta\theta_{slip,u-,y}$ is the post-yield part of the fixed-end-rotation due to slippage of longitudinal bars from their anchorage zone outside the member length.

φ_u and φ_y are determined from plane section analysis.

– for φ_y, linear-elastic stress-strain relations may be assumed until yielding of the tension or the compression chord;

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– for φ_u, the parabola-rectangle diagram of subclause 7.2.3.1.5 and Figure 7.2-9 should be used for concrete in compression and the idealized one of subclause 7.2.3.2 and Figure 7.2-15 (with linear strain-hardening) for the reinforcing steel. Calculation of φ_u should take into account all possible failure modes:

  (a) rupture of tension reinforcement in the full, unspalled section;

  (b) exceedance of the concrete ultimate strain ε_{cu2} at the extreme compression fibres of the unspalled section;

  (c) rupture of tension reinforcement in the confined core after spalling of the cover;

  (d) exceedance of the ultimate strain $\varepsilon_{cu2,c}$ of the confined core after spalling.

Failure mode (b) governs over (c) or (d) if the moment resistance of the confined core exceeds 80% of that of the full unspalled and unconfined section.

For the purposes of the determination of φ_u under cyclic loading, the strain at rupture of ribbed tension bars should be taken equal to:

$$\varepsilon_{su,cyc} = (3/8)\varepsilon_{u,k} \qquad (7.4\text{-}29)$$

Owing to the large local inelastic strains and surface cracking that develop in a bar when it buckles in one half-cycle of loading, the bar may rupture during the subsequent tensile half-cycle under a strain much lower than its nominal elongation at maximum force (which in practical applications may be taken equal to its characteristic value, $\varepsilon_{u,k}$). Moreover, being erratic and unpredictable under cyclic loading, bar buckling of one out of several bars in a section may take place early, leading then to bar rupture.

Bonded tendons of prestressed components do not buckle under cyclic loading. It is safe-sided to take their strain at rupture as equal to that applying in monotonic loading, namely:

$$\varepsilon_{su,mon} = \left(1 - \frac{1}{3}\sqrt{\ln N_{t,tension}}\right)\varepsilon_{su,nominal} \qquad (7.4\text{-}30)$$

where

$N_{t,tension}$ is the number of prestressing tendons in the tension zone.

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There is a tendency for the ultimate strain of the extreme fibres of the confined core after spalling of the cover under cyclic loading to increase with decreasing neutral axis depth, x_o, or depth, h_o, of the confined core of the section as:

$$\varepsilon_{cu2,c} = 0.0035 + \left(\frac{1}{x_o(mm)} \right)^{3/2} + 0.4 \frac{\alpha \rho_w f_{yw}}{f_{cc}} \quad (7.4\text{-}32)$$

or:

$$\varepsilon_{cu2,c} = 0.0035 + \left(\frac{10}{h_o(mm)} \right)^{2} + 0.4 \frac{\alpha \rho_w f_{yw}}{f_{cc}} \quad (7.4\text{-}33)$$

For the full, unspalled section the confinement term does not apply and the dimensions x and h of the full section are used, in lieu of those of the confined core.

while the ultimate strain of the extreme compression fibres of a concrete core confined by closed ties with 135°-hooks may be taken as:

$$\varepsilon_{cu2,c} = 0.0035 + 0.4 \frac{\alpha \rho_w f_{yw}}{f_{cc}} \quad (7.4\text{-}31)$$

where:

ρ_w is the ratio of transverse reinforcement in the direction of bending (or the minimum in the two transverse directions for biaxial bending)

f_{yw} is the yield stress of the transverse reinforcement;

α is the confinement effectiveness factor:

α may be estimated as:

– for rectangular sections:

$$\alpha = \left(1 - \frac{s_h}{2b_o}\right)\left(1 - \frac{s_h}{2h_o}\right)\left(1 - \frac{\sum b_i^2 / 6}{b_o h_o}\right) \quad (7.4\text{-}34)$$

– for circular sections with circular hoops:

$$\alpha = \left(1 - \frac{s_h}{2D_o}\right)^2 \quad (7.4\text{-}35)$$

– for circular sections with spiral reinforcement:

$$\alpha = \left(1 - \frac{s_h}{2D_o}\right) \quad (7.4\text{-}36)$$

where:

s_h denotes the centreline spacing of stirrups;

D_o, b_o, h_o are the confined core dimensions to the centreline of the hoop;

b_i is the centreline spacing along the section perimeter of longitudinal bars (indexed by i) engaged by a stirrup corner or a cross-tie.

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Between yielding of the end section and the ultimate curvature there under cyclic loading, the yielding of the tension bars penetrates into their anchorage zone, increasing the fixed-end-rotation of the end due to slippage of longitudinal bars from their anchorage by:

$$\Delta\theta_{slip,u-y} = 5.5 d_{bL} \phi_u \tag{7.4-37}$$

For φ_u, φ_y and $\Delta\theta_{slip,u-,y}$ determined as above, the "plastic hinge length" L_{pl} for cyclic loading may be estimated as:

– for beams, rectangular columns or walls and members of T-, H-, U- or hollow rectangular section:

$$L_{pl} = 0.2h\left[1+\frac{1}{3}min\left(9; \frac{L_s}{h}\right)\right] \tag{7.4-38}$$

– for circular columns or piers with diameter D:

$$L_{pl} = 0.6D\left[1+0.15\,min\left(9; \frac{L_s}{D}\right)\right] \tag{7.4-39}$$

With $\theta^{pl}{}_{u,m}$ from Eqs. (7.4-18) to (7.4-38), the safety factor for its conversion to a characteristic value via Eq. (7.4-17) is γ_{Rd} =2.

$\theta^{pl}{}_{u,m}$ may also be estimated through purely empirical expressions, which have – or can be extended to – a wider scope and are associated with smaller model uncertainty and lower values of the γ_{Rd} safety factor.

Two practically equivalent purely empirical expressions for beams, rectangular columns or walls and members of T-, H-, U- or hollow rectangular section, are

$$\theta_{u,m}^{pl} = \alpha_{st}^{pl}(1-0.4a_{w,r})\left(1-\frac{a_{w,nr}}{4}\right)(0.25)^{\nu}\left(\frac{max(0.01;\omega_2)}{max(0.01;\omega_1)}\right)^{0.3} f_c^{\,0.2}\left(\frac{L_s}{h}\right)^{0.35} 25^{\left(\frac{\alpha\rho_w f_{yw}}{f_c}\right)} 1.275^{100\rho_d} \tag{7.4-40}$$

$$\theta_{u,m}^{pl} = \alpha_{st}^{hbw}\left(1-0.05\,max\left(1.5; min\left(10; \frac{h}{b_w}\right)\right)\right)(0.2)^{\nu}\left(\frac{max(0.01;\omega_2)}{max(0.01;\omega_1)}\frac{L_s}{h}\right)^{\frac{1}{3}} f_c^{\,0.2} 25^{\left(\frac{\alpha\rho_w f_{yw}}{f_c}\right)} 1.225^{100\rho_d} \tag{7.4-41}$$

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where:

$\theta^{pl}_{u,m}$ is in rads, f_c in MPa, and:

a^{pl}_{st}, a^{hbw}_{st} are coefficients for the type of steel:

- for Class B, C or D steel: a^{pl}_{st}=0.0143 and a^{hbw}_{st}=0.017;
- for Class A: a^{pl}_{st}=0.0069, a^{hbw}_{st}=0.0073.

$a_{w,r}$ is zero-one variable for rectangular walls:

- $a_{w,r}$ = 1 for rectangular walls,
- $a_{w,r}$ = 0 otherwise;

$a_{w,nr}$ is zero-one variable for non-rectangular sections:

- $a_{w,nr}$ = 1 for T-, H-, U- or hollow rectangular section,
- $a_{w,nr}$ = 0 for rectangular ones;

$\nu = N/bhf_c$

where:

b is the width of the compression zone,

N is the axial force, positive for compression;

$\omega_1 = (\rho_1 f_{y1} + \rho_v f_{yv})/f_c$ is the mechanical reinforcement ratio for the entire tension zone, including the tension chord (index 1) and the web longitudinal bars (index v);

$\omega_2 = \rho_2 f_{y2}/f_c$ is the mechanical reinforcement ratio for the compression zone;

$L_s/h = M/Vh$ is the shear-span-to-depth ratio at the section of maximum moment;

α is the confinement effectiveness factor from Eq. (7.4-34);

$\rho_w = A_{sh}/b_w s_h$ is the ratio of transverse steel parallel to the plane of bending;

ρ_d is the steel ratio of diagonal bars (if any) in each diagonal direction of the member;

b_w is the width of one web, even in cross-sections with two or more parallel webs.

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With $\theta^{pl}_{u,m}$ from these expressions, the safety factor for its conversion to characteristic value via Eq. (7.4-27) is γ_{Rd}=1.75.

For prestressed components with bonded tendons Eq. (7.4-40) and Eq. (7.4-41) may be applied (as a matter of fact, they are safe-sided), if the prestressing tendons are included in the calculation of ω_1 and ω_2 and the prestressing force in N.

7.4.3.5 Cyclic shear resistance at the ULS in members with shear reinforcement

The design value of shear resistance within flexural plastic hinges forming at member ends in the seismic situation is determined according to 7.3.2, but using the following limiting values for the inclination of the compression struts, ϑ:

– $\cot\vartheta = 1$, wherever the plastic part of the chord rotation demand at the yielding member end is more than two-times the elastic part, θ_y;

– $\cot\vartheta = 2.5$, for elastic flexural response (zero plastic chord rotation);

with interpolation in-between these values.

The shear resistance of walls for web crushing, $V_{Rd,max}$ is computed with a reduction coefficient k_c applied on the concrete compressive strength equal to:

$$k_c = 0.25\left(\frac{30}{f_{ck}}\right)^{\frac{1}{3}} \le 0.25 \qquad (7.4\text{-}42)$$

7.4.3.6 ULS verification of joints between horizontal and vertical elements

Joints between horizontal and vertical elements (beams and columns in frames, piers and the deck in a bridge) may be verified at the ULS as plates under in-plane loading consisting of:

– n_x, n_y: mean axial stresses in the joint core from the vertical and the horizontal element;

– v_j,: nominal shear stress in the joint core.

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In the ULS verifications a reduction coefficient k_c is applied on the concrete compressive strength to account for the reinforcement running obliquely to the direction of compression.

$$k_c = 0.55\left(\frac{30}{f_{ck}}\right)^{\frac{1}{3}} \leq 0.55 \qquad (7.4\text{-}43)$$

Vertical reinforcement between the extreme bars of the vertical element in the plane of loading the frame counts as joint vertical reinforcement. Hoops should be placed as joint horizontal reinforcement.

7.4.3.7 SLS verifications of flexural deformations

For the verification of the SLS of deformations in seismic situations according to subclause 4.5.2.5, a is normally the chord rotation at a member end. Its value is determined through nonlinear analysis according to subclause 7.4.3.2.4 or via linear elastic analysis according to subclause 7.4.3.2.2, if applicable.

– if the operational limit state in subclause 3.3.1.1 is verified, it is appropriate to use as C_d the value of the chord rotation at yielding, θ_y, of the member end of interest;

– if the immediate use limit state in subclause 3.3.1.1 is verified, an appropriate value for C_d is equal to $2\theta_y$.

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Several nominal fire curves are proposed in the codes to be used in the design process for representing the action of the fire. The most often used are the ISO 834 fire curve, the ASTM E119 curve, the hydrocarbon fire and the external fire curve. All are formed of a simple relationship giving one temperature (the temperature of the gases in the compartment) as a function of time. They are thus representing a fully developed fire. For a large compartment, such a situation is not encountered before a significant amount of time has elapsed since the very beginning of the fire. This initial period of time is thus not taken into account in the calculated fire resistance whereas, as far as safety of people is concerned, this is the most important period; in fact the only one during which evacuation from the fire compartment is possible.

The cooling down phase of the fire is generally not taken into account in the nominal fire curves. In fact, when a certain fire resistance time is required, no consideration is given to the period beyond this duration.

These relationships hardly depend on the particular characteristics of the situation for which the design is performed. The quantity of combustible material, the dimensions of the compartment, the conditions of ventilation, for example, are not taken into account.

If these characteristics shall be taken into account, it is necessary to apply fire safety engineering, where the fire risk is considered and the exposure is determined based on the actual conditions and utilization. Hereby a fire curve is estimated for the actual room/structure. It should be noted that the fire safety in this case is dependent on the assumptions for this determination of the fire exposure.

7.5 Verification of structural safety (ULS) for extreme thermal conditions

7.5.1 Fire design

7.5.1.1 Introduction

Fire design includes consideration of a transiently changing material in interaction with the exposure: the fire, the temperatures, the thermal characteristics and the impact on the structural behaviour.

Fire design is necessary in order to achieve a suitable performance in case of fire and to minimize the damage from a fire event.

The present chapter on fire contains the fires design principles, and the calculation methods and it outlines the design of selected important structural elements. Finally the topic of compartmentation is briefly discussed.

In the present chapter the fire risk and the fire exposures have not been specified. Even though these topics are very important, it is out of the scope of the present code to specify loads and exposures.

Normal structural design is based on design of components. The safety format takes the component considerations as a basis.

In the fire design of concrete structures it is strongly recommended to consider the behaviour of the entire structure and the interaction between deformations and lost strength in one part of the structure. This may in some case lead to critical load situations in other parts of the structure not directly exposed to the fire.

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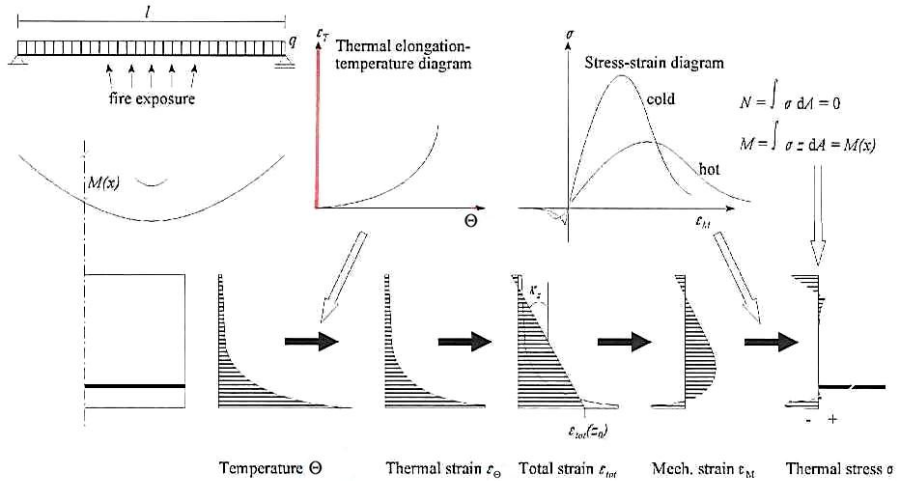


Figure 7.5-1: Graphical representation of strains and stresses in a simply supported beam exposed to the fire along the bottom face

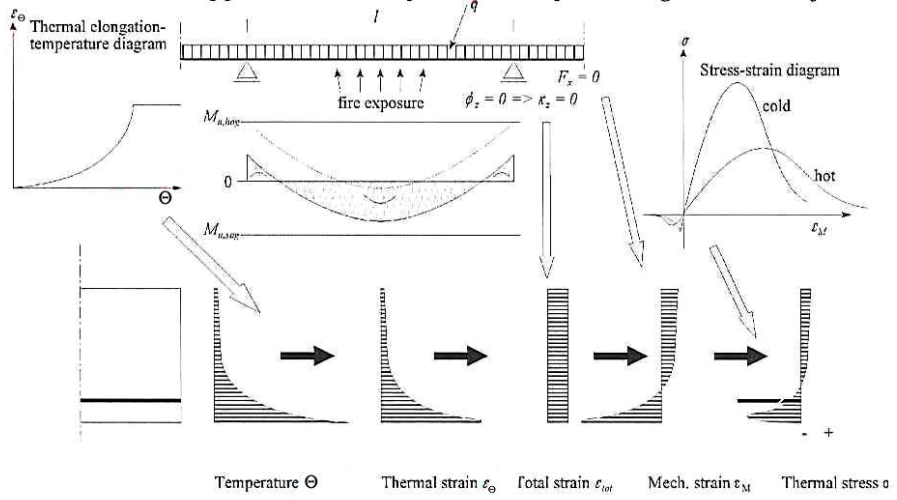


Figure 7.5-2: Graphical representation of strains and stresses in a continuously supported beam exposed to the fire along the bottom face The indication of zero rotation and zero curvature is valid for a point in the adjacent spans

7.5.1.1.1 Direct and indirect effects of fire

The most direct effect of a fire on a structure is that the temperature in the structure will increase, in a first phase, then decrease progressively as the fire decreases until extinction. During the heating phase, heat is introduced into the structure by a combination of convection from the surrounding gas and radiation (Figure 7.5-1 and 7.5-2).

During a fire the temperature is far from uniform in the concrete elements. The main reason for this is the massive dimensions of the structural elements. Because the material characteristics change as a function of the temperature, the material properties at various locations in the cross section will transiently change. This influences the cross sectional behaviour and may result in restraint forces, internal stresses and deformations.

Indirect actions are those effects of actions that arise from restrained thermal expansion. For example, a beam that cannot freely expand longitudinally will be subjected to an increase in the axial force especially during the first minutes of the fire.

In fact, even in a single member that is completely free to expand, indirect effects do appear at the local level.

Thermal gradients in concrete slabs generate the large deflections that are required for the membrane tension effect to develop. In this case, thermal expansion has a positive effect on the stability.

Restraint to axial expansion induces axial compression forces in beams which, depending on the position of the restraint, can have a favourable effect (restraints close to the bottom of the end sections), or an unfavourable effect (restraints close to the top of the end sections), in both simply-supported and continuous beams (favourable effect = reduction of the tensile stresses and of the bending moments).

Restraint to thermal expansion increases the compressive force in columns but the effect may not be as detrimental as expected if the behaviour of the building as a whole is taken into account.

Restraint to thermal bowing dramatically modifies the bending moment diagram in continuous beams or slabs with a clear tendency to have more negative bending than under ambient conditions.

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The symbols in Figure 7.5-2 are defined as:

$M_{u,hog}$ is the moment at support;

$M_{u,sag}$ is the mid-span moment;

F_x is the axial force in x-direction;

κ_z is the curvature in z-direction;

φ_z is the angular rotation in z-direction.

7.5.1.2 Fire design principles

7.5.1.2.1 Ultimate limit state

The fire situation is an accidental situation that requires only verifications against the ultimate limit state (as opposed to the serviceability limit state). Ultimate limit states are these states associated with structural collapse or other similar forms of structural failure such as loss of equilibrium, failure by excessive deformation, formation of a kinematic mechanism, rupture or loss of stability.

In the semi probabilistic approach, the design against the ultimate limit state is based on the comparison between the resistance of the structure calculated with the design values of the material properties, on one hand, and the effects of the design values of the actions on the other hand, as in Eq. (7.5-1).

$$R_{d,fi}(X_{d,fi}) \geq E_{d,fi}(F_{d,fi}) \qquad (7.5\text{-}1)$$

where

$R_{d,fi}$ is the design value of the resistance in case of fire;

$X_{d,fi}$ is the design value of the material properties in case of fire;

$E_{d,fi}$ is the design value of effects of actions in case of fire;

$F_{d,fi}$ is the design value of the actions in case of fire.

The resistance and the effects of actions are both based on characteristic values of geometrical data, usually the dimensions specified in the design, e.g. cross sections. Geometrical imperfections such as bar out of straightness or frame initial inclinations are represented by design values.

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The design value of the accidental action that has been mentioned previously does not appear in Eq. 7.5-1 because, in case of fire, the fire action is not of the same form as the other actions. It does not consist of some N or some N/m² that could be added to the dead weight or to the wind load. The fire action consists of indirect effects of actions induced in the structure by differential and/or restrained thermal expansion.

Table 7.5-1 given here (Table A1.1 of Eurocode 0 (EN 1990, 2003, Table A1.1 in Annex A1)) gives the relevant ψ factors for the fire situation in buildings.

Table 7.5-1: Coefficients for combination ψ for buildings

Action	ψ_1	ψ_2
Imposed load in buildings		
category A: domestic, residential	0.5	0.3
category B: offices	0.5	0.3
category C: congregation areas	0.7	0.6
category D: shopping	0.7	0.6
category E: storage	0.9	0.8
Traffic loads in buildings		
category F: vehicle weight ≤ 30kN	0.7	0.6
category G: 30kN < vehicle weight < 160kN	0.5	0.3
category H: roofs	0.0	0.0
Snow loads for H < 1000 m amsl	0.2	0.0
Wind loads	0.5	0.0

Different actions generally occur simultaneously on the structure. In an accidental situation, they have to be combined as follows:

- design values of permanent actions;
- design value of the accidental action;
- frequent or quasi-permanent value of the dominant variable action;
- quasi-permanent values of other variable actions.

When it is not obvious to determine which one amongst the variable actions is the dominant one, each variable action should be considered in turn as the dominant action, which leads to as many different combinations to be considered.

In case of fire, and if the variability of the permanent action is small, i.e. in most cases, the following symbolic equations hold:

$$\sum_{j\geq 1} G_{k,j} + P + \psi_{1,1}\, Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad (7.5\text{-}2a)$$

$$\sum_{j\geq 1} G_{k,j} + P + \psi_{2,1}\, Q_{k,1} + \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad (7.5\text{-}2b)$$

where

G_k, Q_k, P_k	characteristic values of the permanent, variable and prestressing action;
$G_{d,fi}$, $Q_{d,fi}$, $P_{d,fi}$	design values of these actions in case of fire;
ψ_1	coefficient for frequent value of a variable action;
ψ_2	coefficient for quasi-permanent value of a variable action.

Eq. 7.5-2a is based on the frequent value and Eq. 7.5-2b on the quasi-permanent value for the dominant variable action.

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The rationale for using 1.0 as a partial safety factor for material properties and for the actions lies in the theory of conditional probabilities. Assuming that the probability of failure at ambient condition meets a particular target value, the following possibilities exist:

P(failure at ambient conditions) ≤ Target value

The probability that the structure ever fails in a fire is the product of two probabilities: the probability that a severe fire occurs and the probability that this fire causes failure.

P(failure in fire condition) = P(there is a fire) x P(failure caused by this fire)

The coupled probability of having a fire and having a failure under that fire should be the same as the probability of having a failure at ambient temperature. This is why more favourable values of the partial safety factors are used in the fire situation, as well as in any accidental situation.

P(failure in fire condition) ≤ Target value

From the above equations it follows:

P(failure caused by the fire) ≤ Target value / P(there is a fire)

An explanation of why the materials' factors are taken equal to 1.0 in fire, instead of 1.2 in accidental conditions, is that in many accidental conditions (like earthquakes) the uncertainty is taken care of partly in the material ($\gamma_m = 1.2$) and partly in the actions. In fire, however, the uncertainty is totally taken care of in the actions, by means of very severe temperature-time curves (standard fire), or rather refined and realistic temperature-time curves (natural fires). In the specific case of concrete, it is well known that high temperature reduces the random scattering of the material properties, and so the uncertainties.

The design values of the material properties, $X_{d,fi}$, are described by the general equation:

$$X_{d,fi} = \frac{X_k(\Theta)}{\gamma_{M,fi}} \qquad (7.5\text{-}3)$$

with

$\gamma_{M,fi}$ is the partial safety factor for material property in fire design, normally taken as 1.0;

Θ is the temperature.

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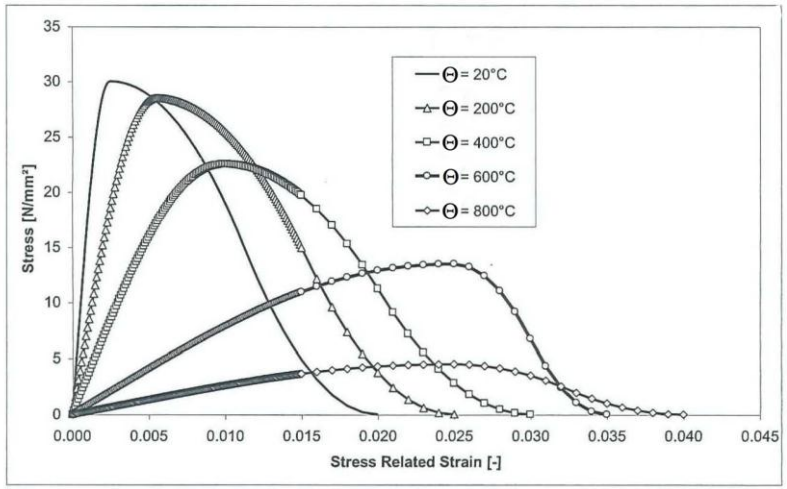


Figure 7.5-3: Example of stress-strain relationships of concrete under compression at elevated temperatures

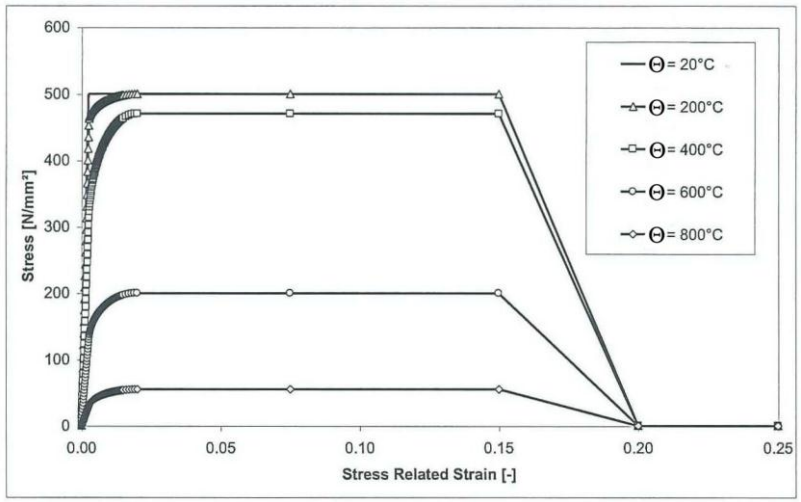


Figure 7.5-4: Stress-strain relationships of reinforcing steel at elevated temperatures

7.5.1.2.2 Concrete and steel

The present chapter concerns design of concrete structures, and the detailed description of concrete as a material is beyond the scope of this sections. Therefore only some principal features are mentioned with respect to concrete material and fire design.

Concrete is in itself a versatile material, which – based on the mix-design including the admixtures – can exhibit many different characteristics depending on the actual requirements. In case of a fire the material properties change because of the progressive thermal damage; hence the structure or the cross section actually consist of a multitude of different concrete layers with different material properties. These properties constantly change during and after the fire.

The strength as well as the stiffness of steel and concrete are reduced by a temperature increase. The evolution of the strength and stiffness characteristics with temperature is yet not sufficient to describe the modification of the material characteristics because the whole stress-strain relationships are modified.

The modifications of the material properties comprise the following:

Compressive strength is the most extensively analysed property of concrete. The strength at room temperature, the water/cement ratio, the type of cement, the maximum aggregate size and the rate of heating appear to have little influence on the relative reduction, in percent of the original strength. The type of aggregate has an influence, the decrease being less important with calcareous or lightweight aggregates compared to siliceous aggregates. The aggregate/cement ratio has also an effect, with the reduction being proportionally smaller for lean mixes than for rich mixes. Finally the reduction highly depends on the testing procedure, with more favourable results obtained when a certain compression stress level is maintained during heating, as it is generally the case in reinforced concrete columns.

The modulus of elasticity of concrete is influenced in the same way by the factors previously mentioned for the compressive strength. The reduction as a function of the temperature is larger than for the compressive strength because of the simultaneous decrease of the compressive strength and increase of the strain at the peak stress.

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Steady state creep is of importance essentially for service conditions, i.e. temperatures below 150°C applied for very long periods, in concrete reactor vessels for example. In a fire situation, the creep rates observed under steady state conditions are very much less important than the creep values observed under transient temperature conditions.

Load-Induced Thermal Strain is the particular deformation that occurs in concrete during first heating under load. It is influenced mainly by the aggregate type, by the aggregate/cement ratio and by the curing conditions; air-cured and oven-dried specimen exhibit a significantly lower transient strain than water-cured specimens.

The tensile strength of concrete has a tendency to decrease faster with the temperature than the compressive strength, but less than the elastic modulus.

The fracture energy of concrete has a tendency to keep constant or to slightly increase with the temperature, but how to define and measure the fracture energy is still a highly-debated issue, in spite of the many test results.

The stress-strain relation of concrete in compression reflects the modifications due to elevated temperatures of the compressive strength, of the modulus of elasticity, and of the strain at the peak stress. As for the ultimate strain at 20°C, its value is defined in such a way that the M-N envelopes at the ultimate limit state obtained by means of an incremental-iterative procedure be in agreement with those obtained with the traditional approach based on strain limitations, and on the usual parabola-rectangle (or bilinear) stress-strain curve.

The thermal properties of concrete are also modified by a temperature increase. The thermal conductivity is normally reduced whereas the specific heat is increased by an elevation of the temperature. As a result, the thermal diffusivity decreases with increasing temperatures.

The yield strength of steel is reduced by an elevation of the temperature. The relative reduction does not depend on the value of the yield strength at room temperature but it varies with the type of steel, hot rolled (less temperature-sensitive) or cold worked (more temperature-sensitive) reinforcing steel, quenched and tempered or cold worked prestressing steel (very temperature-sensitive).

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The elastic modulus of steel is also reduced at elevated temperatures, somewhat faster than the yield strength.

The thermal properties of steel are also modified by a temperature elevation, with a decrease of the thermal conductivity and a slight increase of the specific heat. This is not particularly relevant in concrete structures because the amount of steel is generally so low that it hardly influences the temperature distribution.

The geometrical size of concrete and steel is modified by a temperature increase. This is the well known *thermal expansion*. The expansion is not a linearly increasing function of the temperature. The order of magnitude of the thermal expansion can reach 1% at very high temperatures in the range of 800°C. This phenomenon plays an important role in the behaviour of structures because it induces either large displacements that may generate geometrical second order effects or indirect effects of actions if the expansion is restrained.

The bond strength between concrete and steel has been shown to decrease with temperature similarly to the tensile strength rather than to the compressive strength. Experience has yet very rarely produced evidence of failures by debonding in reinforced structures. The problem is more critical in prestressed pretensioned members.

Spalling in concrete structures is a very important characteristic linked to high temperatures. Different types of behaviour are usually named as spalling, from the very progressive sloughing off at the surface that progressively exposes the inner part of the section and the reinforcing bars to elevated temperatures, to the explosive spalling that suddenly destroys completely the material. Extensive research activity is still going on in order to understand and mitigate this phenomenon, the problem becoming more crucial with the introduction of high strength concretes because they have a more closed and dispersed porosity, which favours the increase of vapour pressure in the pores. The factors most often mentioned as playing a negative role in spalling are: fast temperature increase, high moisture content, high compression stress level, young age, low porosity, thin members, geometrical effects (corner spalling). Some of these are related to the material itself, but it seems that structural effects also play a role in this phenomenon.

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For the approach based on the use of tabulated data reference can be made to EN 1992-1-2, Section 5.

For the approach based on the use of the 500°C isotherm method reference can be made to EN 1992-1-2, Annex B1. For the approach based on the zone method reference can be made to EN 1992-1-2, Annex B2.

For the approach based on the use of stress-strain, temperature-dependent laws, reference can be made to EN 1992-1-2, Section 3.2.2.1.

7.5.1.2.3 Heating of materials

After having defined or determined the fire exposure, and in order to be able to evaluate the modifications of the material and the structural effects, it is necessary to investigate the thermal field, which depends on the fire exposure and on the thermal properties of the material, as well as on the structural context.

The thermal properties of the material will not be discussed here.

The direct consequence of the heating of the concrete is a modification of the material characteristics like a decrease of the strength and of the stiffness, as well as the generation of additional deformations associated with the stress level during first heating. These deformations are usually called Load-Induced Thermal Strain (LITS) or transient creep in the literature. Furthermore, thermal elongation is a direct result of the temperature increase, which is not uniformly distributed on the section. These strains and their non-uniform distribution have several effects on the behaviour of concrete members and the effect may be different depending on section type. The most important effects have to do with: spalling, elongation and deflections.

7.5.1.3 Calculation method

7.5.1.3.1 Sectional analysis

Sectional analysis is a common approach to verify the fire resistance. The bearing capacity of reinforced concrete sections subjected to a fire is usually evaluated by means of different approaches:

- by using tabulated data (first-level method);
- by using the reference-isotherm method or the zone method (second-level methods);
- by using temperature-dependent stress-strain relationships within the framework of an incremental-iterative procedure (third-level method).

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Reference is made to EN 1992-1-1.

Reference is made to EN 1992-1-2.

The tabulated data are based on past experience and on the theoretical evaluation of tests (Naranayan and Beeby, 2005). These data provide a set of admissible values for the main geometric parameters of a section, including the cover of the reinforcement, as a function of the fire duration that the element is required to withstand.

The reference isotherm method is based on reasonable assumptions, and was originally devised for reinforced concrete sections subjected to pure bending, where the failure is generally controlled by the yielding of the tensile reinforcement. The possible extension to sections subjected to an eccentric axial force is still under discussion.

Investigations show that the larger is the fire duration, the worse is the agreement between the 500°C isotherm method and the exact method, particularly under large axial forces. Of course the two methods give almost the same results in the case of less temperature-sensitive large sections. The bearing capacity in pure compression at low values of the fire duration is slightly overestimated by the isotherm method, which on the whole is rather conservative.

In the present clause focus is given to nonlinear analysis. Four issues are addressed:

- the use of nonlinear analysis implemented with simplified constitutive laws, as an alternative to realistic (but more complex) laws, at room temperature;
- the use of incremental-iterative procedures ("exact" method) and nonlinear analysis in fire conditions;
- the validity of the well-known 500°C isotherm method in fire conditions, under an eccentric axial force;
- the relevance of the eigenstresses generated by the thermal gradients.

Tabulated data

The approach of tabulated data allows the designer to give a quick response in many practical cases with well-defined boundary conditions. On the other hand, this approach does not allow the designer to refer to materials' properties and fire scenarios other than ordinary concrete and the standard ISO834 Fire Curve. Neither the mechanical, nor the thermal aspects of the problem are explicitly addressed by this approach.

Reference-isotherm method (500°C isotherm)

The "reference-isotherm" method (or "effective-section" method) is based on the assumption that concrete is fully damaged above the temperature of 500°C, while it is fully effective (fully undamaged) for temperatures below 500°C. On the contrary, the mechanical decay of the reinforcing steel is explicitly introduced.

This method can be applied within the context of nonlinear analysis, by assuming the parabola-rectangle stress-strain curve at ambient temperature for the concrete, with the usual strain limitations, and by considering only the undamaged part of the concrete section.

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For the temperature-dependent stress-strain curves reference is made to EN 1992-1-2, Section 3.2.2.1.

Zone method

The zone method retains the philosophy of the 500°C isotherm method, but considers a more complex and realistic effective section, whose size depends on the temperature distribution. Also the characteristics of the concrete in the effective section (compressive strength and Young's modulus) depend on the temperature distribution. In order to perform the calculation, the section is divided into a finite number of zones. The temperature is determined in the centroid of each zone, on the basis of the thermal analysis. The method is more complex than the 500°C isotherm method, but yields better results, especially in the case of pure compression and thin webs exposed to the fire on both sides. Moreover, the method allows to consider second-order effects, by introducing proper correction coefficients.

Exact method – Incremental-iterative procedure

The incremental-iterative procedure is based on the temperature-dependent stress-strain curves. At first a thermal analysis is performed in order to determine the temperature distribution in the section, and thus the level of the thermal damage at each point, for any given fire duration. The mechanical properties of concrete and steel in each point can then be related to the maximum temperature reached locally by means of the temperature-dependent stress-strain curves. In this way, the section is considered as a composite section, consisting of many different materials, whose properties and spatial distribution are related to the thermal field.

The next step is to determine the ultimate value of the bending moment M_u for a suitable set of values N_u of the axial force. This is done by working out, for each value N_u, the corresponding moment-curvature diagram of the section, by means of an incremental-iterative procedure (see also 7.5.1.3.2). Once the moment-curvature diagram is known, the maximum value of the bending moment is the ultimate bending moment M_u, corresponding to the assigned value of the axial force N_u. The couples of values (N_u,M_u) identify as many points in an M-N domain, and the interaction envelope is obtained by connecting these points.

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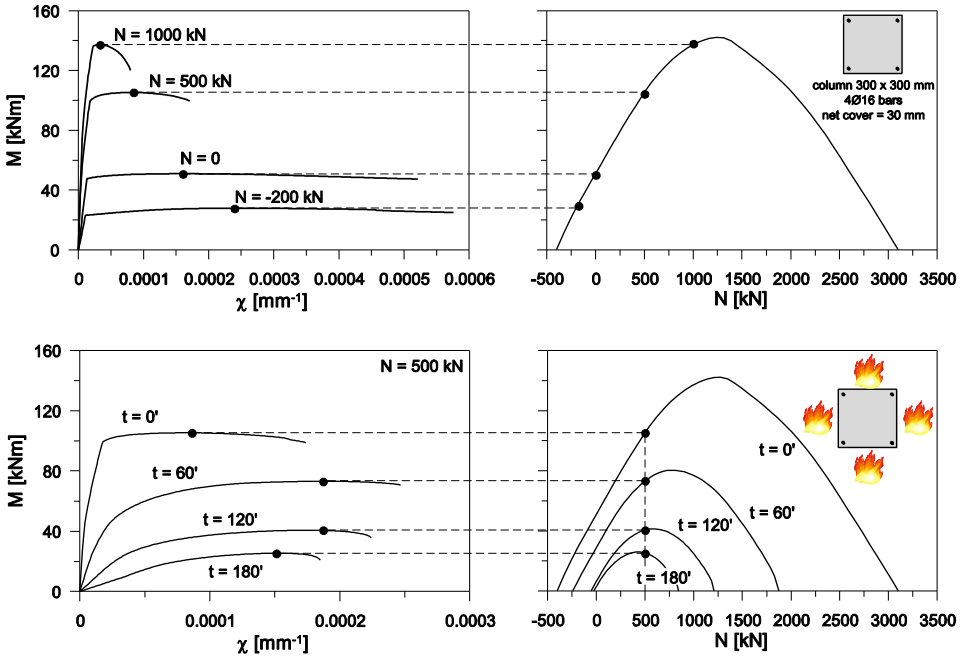


Figure 7.5-5: Examples of the application of the incremental-iterative procedure at room temperature (a); and for different values of the fire duration (b)

This procedure is rather time consuming when compared with the previous methods (500°C isotherm and zone methods). Nevertheless, nonlinear analysis based on strain limitations cannot be used with the stress-strain curve proposed in EN 1992-1-2, Section 3.2.2.1, because the attainment of the ultimate strains in one of the two materials does not correspond – in general – to the attainment of the ultimate bearing capacity.

Note that the strain-limitation method, that is commonly adopted in ambient conditions, can be extended to high temperature, only if suitable monotonic stress-strain curves are introduced (see Strain-limitation method below).

The sectional capacity is underestimated if the fire curve is not used properly, i.e. by adopting the afore-mentioned incremental-iterative procedure. It seems that in order to achieve a reliable result with non-linear analyses, the incremental iterative procedure will have to be followed.

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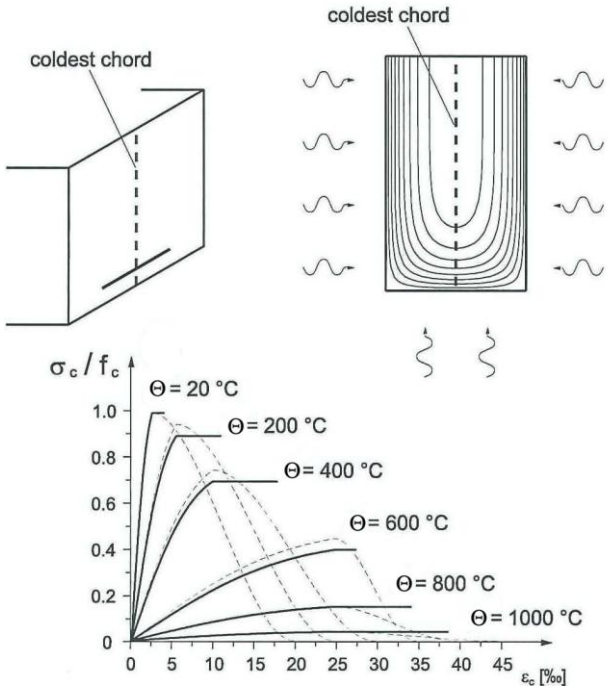


Figure 7.5-6: Strain-limitation method and qualitative parabola-rectangle diagrams

These curves can be easily worked out from the softening curves proposed by EN 1992-1-2.

Note that once the deformation capacity of the coldest chord is respected, this is also true of any other point.

Strain limitation model

The strain-limitation method is based on simple monotonic stress-strain curves, that are the extension of the well-known parabola-rectangle – or similar – curve used in the design at room temperature. Each curve is valid for a given value of the temperature. After (a) the thermal analysis of the section has been performed for a given value of the fire duration; (b) the "coldest" chord of the section has been identified (coldest chord = farthest chord from the heated surface, according to the inflexion plane; the coldest chord is characterized by the least deformation capacity); (c) the distribution of the ultimate strains along this chord has been identified; and (d) a number of linear strain profiles have been plotted considered along the same coldest chord, so as to respect the ultimate strain in each point of the section, the stresses can be worked out for each strain profile, and – by integration – M_u and N_u can be evaluated. For each strain profile there is a single couple of values for M_u and N_u. The couples M_u - N_u make it possible to draw the envelope for the given value of the fire duration.

This method does not require iterative procedures and is as reliable as the exact method, provided that a suitable set of monotonic, temperature-dependent stress-strain curves is available for the concrete.

7.5.1.3.2 The role of thermal strain

Including the effect of the thermal strains requires a clear understanding of the various assumptions concerning the different strain components acting on the section, i.e. the total strain ε_{tot}, the thermal strain ε_Θ and the stress-induced strain ε_σ:

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– the hypothesis that plane sections remain plane is still valid, but with reference to the total strain ε_{tot};

– the usual nonlinear analysis with strain limitations can still be applied, provided that the limitations are referred to the stress-induced strain ε_{σ}.

To work out the moment-curvature diagrams for any given value N of the axial force, the following incremental-iterative procedure is applied, for a number of values of the curvature χ:

– for each value of χ, a tentative deformation ε_0 is assumed in the centroid of the section, and the corresponding total deformation ε_{tot} is determined at each point of the section, assuming that plane sections remain plane;

– the corresponding stress-related strain ε_{σ} is determined at each point of the section:

$$\varepsilon_{\sigma} = \varepsilon_{tot} - \varepsilon_{\Theta}$$

– the value of the stress σ is determined at each point, by using the constitutive laws;

– the axial force N' is evaluated: if N' = N (within a prefixed tolerance), the bending moment M is calculated, and a next value of χ is considered;

– if N' $\neq$ N, the procedure is repeated until N' = N, by varying ε_0; then, the bending moment M is calculated, and a next value of χ is considered.

For the constitutive laws reference can be made to the EN 1992-1-2.

7.5.1.3.3 Plastic analyses

It is stated in Eurocode 2 (EN 1992-1-2, 2004) that an important question is whether load redistributions between different sections of a member in bending can be accepted in case of fire, these redistributions being allowed by the plastic behaviour of both the reinforcement and the concrete not only at room temperature but (even more) at high temperature. One of the key conditions for this plastic behaviour is the ductility of the section, i.e. the capacity of the section to keep on developing the plastic bending moment, when the curvature increases to very high values. This seems to be the case

The theory of plasticity gives a theoretical validation to the fact that several effects leading to self-equilibrated stress distributions can be neglected in nonlinear numerical analysis. Among these effects, (a) those occurring either in the construction phases or during the service life at room temperature, before the fire starts (due for instance to shrinkage, creep and thermal strains), and (b) those occurring during the fire (due to creep and thermal expansion) should be mentioned. A consequence of neglecting these effects is that the strains, stresses and tangent moduli that are computed in

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according to some numerical examples that show how the ductility of a section tends to increase during a fire.

The main difference between the hot and cold situations is the ratio between the ultimate plastic moment and the first-yielding moment. This ratio is much higher in fire, which means that much higher rotations have to take place before the full plastic moment is reached. This is in no contradiction with what is generally observed during laboratory tests, where the failure of R/C structures is often accompanied by very large displacements.

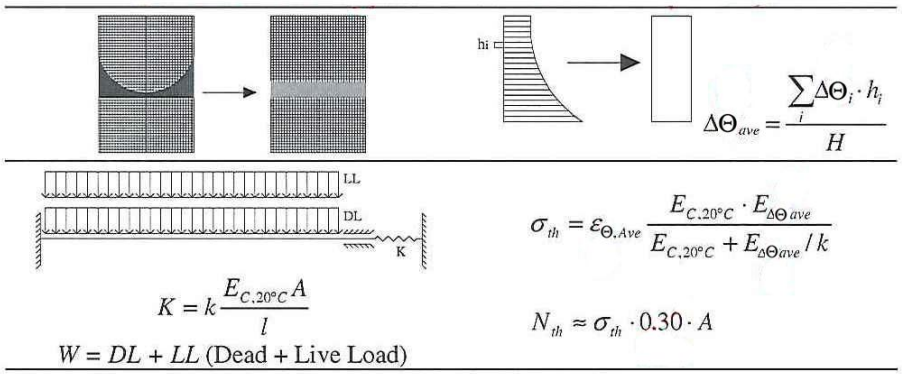


Figure 7.5-7: Evaluation of the axial force in axially-restrained beams

any given point of a structure are only approximate – or "mean" – values compared to the "true" values that would be computed if all these effects were taken into account. The computed values are indeed based on the hypothesis of a "virgin" initial stress distribution, which is far from reality.

Neglecting these self-equilibrated stress distributions is subjected to some limitations, since it is justified as long as the ensuing displacements are small. Hence, the effects of thermal expansion during the fire must be taken into account.

Another strain component that may affect deformations – second-order effects included – in concrete structures submitted to fire is the transient-creep strain.

Use of plastic analysis

Within the scope of fire design applications, the objective of plastic analysis is generally to evaluate the load-carrying capacity of a beam.

According to plastic analysis the verification is positive if the ultimate load at the requested fire duration is larger than the applied load, at the onset of beam collapse because of the formation of a suitable number of plastic hinges.

The plastic (e.g. ultimate) moments at critical sections may be determined according to various sectional-analysis methods, such as the 500°C isotherm method or the zone method.

In axially-unrestrained beams, the application of plastic analysis is straightforward and generally leads to a conservative estimate of the ultimate load.

In case of axially-restrained beams, the ultimate bending moment depends on the axial force developed during the fire (Figure 7.5-7). Its value may be determined by performing the analysis of the entire structure. Since this procedure involves sophisticated models and heavy computing, the axial force has to be evaluated in a simpler way based on fire effects (i.e. thermal field of the section), and on the actual axial restraints of the member in question, before performing the mechanical analysis of the section. An approximate estimate of the axial force, to be adopted in plastic analysis, can be performed via the simplified procedure outlined in the following:

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Comments to verification of the method:

In the case of no axial restraint (K=0), the ultimate bending capacity of the critical sections at any given fire duration is generally underestimated. As a result, plastic analysis underestimates the ultimate load-carrying capacity of a beam, thus leading to conservative results;

The assumed axial force in axially-restrained beams results in a lower-bound estimate of the actual force (as given by nonlinear analysis). Hence, the proposed method underestimates the effects of the axial restraint;

In most cases, the ultimate bending moment of fully axially-restrained beams is overestimated, particularly close to the end sections. As a result, plastic analysis leads – in most cases – to a non-conservative estimate of the ultimate load-carrying capacity.

For partially-restrained beams, plastic analysis leads in most cases to acceptable results, i.e. conservative or slightly non-conservative results.

It is observed that in plastic analysis, by completely neglecting the effects of the axial restraint, the estimate of the ultimate load-bearing capacity of a beam is always on the safe side.

- based on the results of the thermal analysis after a time t of exposure to the fire, the average temperature distribution at each level along the section is determined and the average temperature in the section is found;
- the axial force ensuing from the restrained thermal elongation, corresponding to the mean temperature, is evaluated (a) by computing the longitudinal stress σ_{th} arising in a beam of stiffness $K_{beam} = (E_{\Theta ave} A)/L$, axially restrained by a spring of stiffness $K = k (E_{20} A)/L$, as a consequence of the average thermal elongation $\varepsilon_{\Theta ave}$; and (b) by multiplying such a stress by a suitably-reduced cross section A'. The following definitions/values should be adopted: $A' = 0.30A$, where A is the cross section area of the beam; $E_{\Theta ave}$ (for the evaluation of σ_{th}) is the elastic modulus corresponding to the average temperature of the section; k has to do with the number of equal spans connected to the heated beam (for example, k = 1.00, 0.50, 0.33 for a two-span, three-span and four-span continuous beam, with one end spam in fire); $N_{th} = \sigma_{th} A'$ applied in the geometric centroid of the section. At any temperature T, excluding creep and transient strain, the elastic modulus may be calculated by multiplying the elastic modulus in virgin conditions (E_{20}) by the normalized decay inferred from the σ-ε curves given in the EN 1992-1-2: $E_{\Theta} = E_{20} \cdot (E_{\Theta}/E_{20})_{EC2\text{-}Fire}$;
- the plastic (i.e. ultimate) moments of the critical sections are determined by means of either the 500°C isotherm method or the zone method considering also the axial force.

Plastic analysis is a simple and straightforward method, that is very sensitive to the evaluation (a) of the plastic moments at the inner intermediate supports, and (b) of the effects of the axial restraints.

Neglecting the effects of the axial restraints always leads to a conservative estimate of the ultimate load- carrying capacity in statically-redundant beams.

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For further reference and explanation reference is given to *fib* Bulletin 46: "Fire design of concrete structures – structural behaviour and assessment" (*fib*, 2008).

For the integrity requirements reference can be made to EN1991-1-2.

7.5.1.4 Structural elements

In the following sections some important conclusions concerning the fire design of structural elements are given.

Not only the structural resistance (R) will have to be estimated and documented according to the fire design principles (see subclause 7.5.1.2) but also the insulation requirements (I) and the integrity requirements will have to be documented. This may require a detailed investigation of the deformations.

7.5.1.4.1 Beams and slabs

The following requirements and recommendations are the result of recent studies:

It is recommended to thoroughly consider all the sectional forces when designing beams and frames. It has been observed, that axial forces in the design of reinforced concrete beams (predominantly exposed to bending moment) under fire conditions may contribute to an increased fire resistance.

Hence, neglecting the axial force in the design of reinforced concrete beams under fire conditions leads to a conservative estimate of the fire resistance.

The collapse of continuous reinforced concrete beams is generally controlled by the intermediate support sections, where concrete damage may lead to anticipated section failures.

During the fire, the bending moment can shift upwards due to the negative bending moment induced by thermal gradients. This effect will occur at an early stage in fire before the stiffness degradation takes place.

In such a situation, top reinforcement is required in the mid-span sections to avoid a premature collapse during a fire, especially in (one-way) slabs.

The shear capacity in continuous beams may become critical even though it would not be critical at ambient temperature. Especially for (one-way) slabs the shear forces may become critical as result of the fire.

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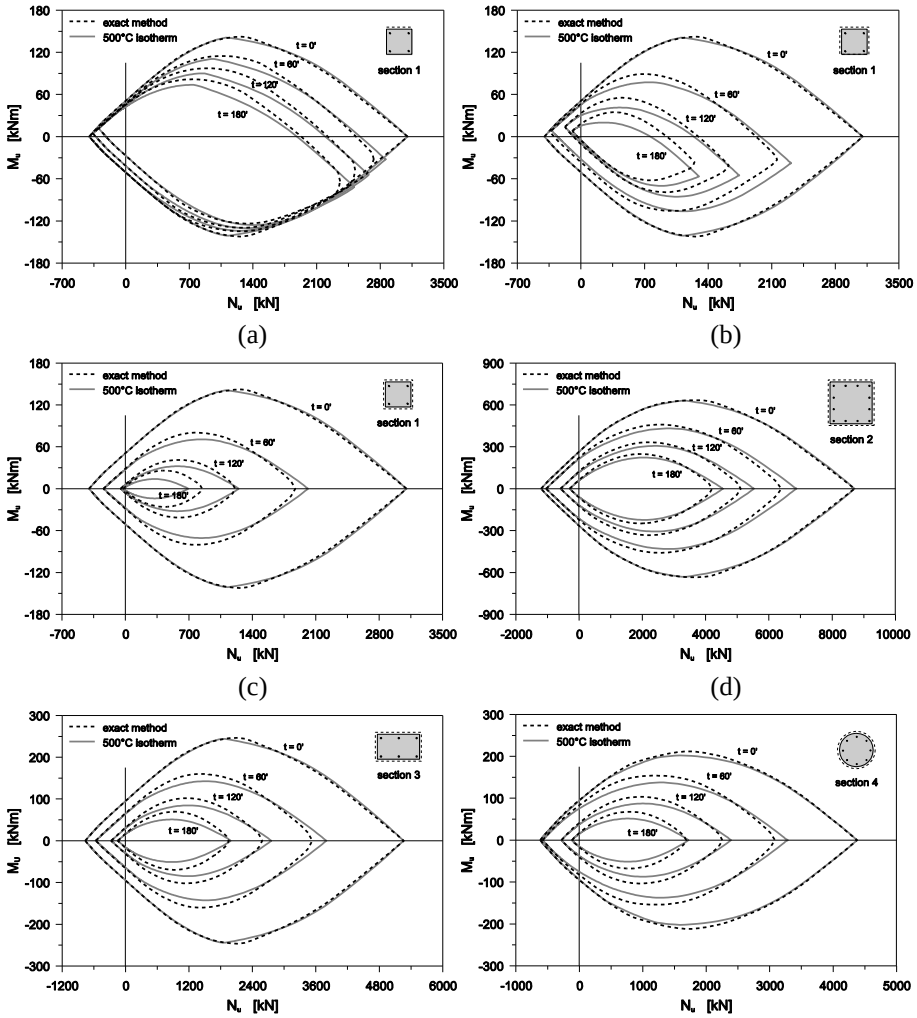


Figure 7.5-8: Example of M-N envelopes for different fire exposures (a,b,c) and for different section geometries (d,e,f). The heated sides of the sections are indicated with dashed lines. The M-N curves illustrate a comparison between the reference isotherm method (using 500°) and the exact method.

For beams which are partly restrained in the axial direction, axial displacements increase rapidly in earliest phases of the fire, whereas they keep constant with rather low values after the fire-induced stiffness degradation has taken place. Also the axial force increases rapidly at the beginning of a fire and moderately later on.

Second-order effects have a marginal influence on rather massive R/C members subjected to bending; on the contrary, in rather thin members like one-way slabs, second-order effects may induce shear forces larger than those at ambient temperature.

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It should be observed that – even if concrete above 500°C has little residual strength – still it contributes markedly to the inertia of the section, because of its peripheral location.

7.5.1.4.2 Frames

The following requirements and recommendations are the result of recent studies:

All the sectional forces should be thoroughly considered in the design of frames.

Neglecting the effects of beam thrust in the design of R/C frames may lead to highly non-conservative results, because of the increasing bending and shear in the columns during the first 30 minutes of fire exposure.

It is required to thoroughly consider and model the support and foundation conditions of the frames in the estimation of the fire resistance of the structure. The thermal effects on bending and shear depend on the type of the foundation adopted in column design, since – for instance – isolated footings provide less rotational stiffness than continuous foundation beams and even less than 2D foundations mats. Consequently, bending and shear in fire increase less in the first case.

Second-order effects have a marginal influence on rather massive reinforced concrete frames subjected to bending.

Detailing rules for the columns similar to those generally adopted in seismic design seem to be suitable also in fire design. In fact, the adoption of densely-spaced closed stirrups (hoops) is instrumental in improving section strength and ductility under combined bending and axial force, and helps in controlling concrete spalling.

7.5.1.4.3 Columns

Reduced M-N envelopes taking into account second-order effects are still hardly available in the literature, and only qualitative sketches can be found.

It may be possible – as a simplified and rough model – to evaluate the slenderness on the basis of the effective section using the reference isotherm method; in the case of NSC (normal strength concrete) the 600°C isotherm may be adopted, whereas a lower value should be chosen in the case of HPC (high performance concrete; see also subclause 7.5.1.3.1.

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For horizontal members, failure in a standard test is assumed to have occurred when the deflection reaches a value of L/20 (L = clear span of the specimen) or the rate of deflection (mm/min) exceeds a prefixed value (for instance $L^2/9000d$ where d is the effective depth in mm). The rate-of-deflection criterion only applies once the deflection has reached a value of L/30.

The origin of the deflection limits are unclear and tests have demonstrated that the load-bearing capacity can be maintained for deflection values much larger than this limit. For concrete-floor members compartmentation failure is generally related to the insulation capacity rather than to the load-bearing capacity.

The origins of the definitions and of the values concerning the limits of deformability, integrity and insulation are unclear and may deserve to be worked over in order to reflect the situation in real fires.

However, the basic idea is that for any given fire duration structural members should behave properly according to the original design philosophy, which means – for instance – that slabs and walls should remain almost flat during a fire.

In the design of structures for storing refrigerated liquefied gases (RLG) generally a distinction is made between "double containment" tank systems and "full containment tank systems". Double containment tank systems consist of a closed primary and an open-top secondary container. The primary container is a single containment RLG structure. Under normal operating conditions it contains both the refrigerated liquid and the associated vapours.

For a more detailed consideration the reduction of elasticity should also be taken into account.

Except for the slenderness considerations the columns may be considered in accordance with the M-N envelopes discussed above.

7.5.1.5 Compartmentation

Compartmentation has traditionally been defined according to the concept of fire resistance, with reference to collapse (R criterion), fire penetration (E criterion), and excessive heat transfer (I criterion).

The purpose of sub-dividing spaces into separate fire compartments is twofold. Firstly, to prevent any rapid fire spread, which could trap occupants of the building and secondly to restrict the overall size of the fire.

Continuity at the junctions of the fire-resisting members enclosing a compartment must be guaranteed. Typically this would be the junction between a wall (either loadbearing or non-loadbearing) and a floor.

Loss of integrity is deemed to occur when gaps and fissures allow the ignition of a cotton pad on the unexposed side, or when gaps are wider than 25 mm.

Insulation failure is deemed to occur when the mean temperature on exceeds by more than 140°C its initial value, or the local temperature on the unexposed face exceeds by 180°C its initial value.

7.5.2 Cryogenic design

7.5.2.1 General

Design of structures under cryogenic circumstances is most of all relevant for the design of containment structures for refrigerated liquefied natural gas (LNG). Concrete is particularly well suited for the storage of cryogenic liquids because most of its properties and its behaviour improve substantially as temperature is lowered into the cryogenic range. In this section the most important properties of concrete in cryogenic conditions are treated.

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The secondary containment is often an open-top concrete wall serving two basic functions:

- protection of the primary container from external loads under normal operating conditions;
- containing the leakage from the primary container, but not the vapour generated from such leakage under accidental spill conditions.

A full containment tank systems consists of an open-top primary and a closed top secondary container. Under normal operating conditions the primary tank contains the refrigerated liquid and vapour, while the secondary tank contains vapour and provides protection to the primary container. Under accidental spill conditions the secondary container contains the leakage from the primary container and contains or controls the vapour generated from such leakage.

Storage structures for refrigerated liquefied gases require taking into account different scenarios for coping with the increased risk of such structures.

7.5.2.2 Design loads to be regarded in the design of structures for refrigerated liquefied gases.

Design loads to be regarded for the design of structures containing refrigerated liquefied gases are:

(a) *Regular loads:* Dead load, product pressure and weight, prestressing forces, installation loads, construction specific loads, wind.

(b) *Incidental loads:* testing, commissioning and decommissioning loads.

(c) *Accidental loads*: seismic loads, explosion and impact, fire, environmental loads.

(d) *Imposed deformation*: loads due to thermal and moisture gradients, loads induced by shrinkage and creep, differential settlement, thermal and moisture gradient loading under spill conditions.

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A primary container shall at least remain liquid-tight under hydrotesting, operation loads, and operation loads in combination with a specified basic seismic load. The primary container should also retain its containment capability under specified seismic loading (SSE = Safe Shutdown Earthquake) and aftershock events (SSE_{aft}. – Safe Aftershock Event). For the secondary container, under all conditions, including but not limited to spill, SSE and SSE_{aft} plus spill events, the structural integrity of the wall should be maintained. The secondary containment shall be designed for the SSE_{aft} event while containing the total volume of spilled product with sufficient prestressing force and reinforcement provided to prevent through thickness cracking of concrete during the SSE_{aft} event.

Details about these scenarios can be found e.g in ACI 376 Code Requirements for Design and Construction of Concrete Structures for the Containment of Refrigerated Liquefied Gases", American Concrete Institute, Jan. 2010

Thermal shock loading will not cause any appreciable loss in strength if the relative humidity (RH) is less that a critical value of about 85%. For higher values of RH, a thermal shock can cause a significant loss of strength only if: a) the concrete is water-saturated and/or b) a high w/c ratio has been used.

The influence of the moisture content is demonstrated in Figure 7.5-9.

7.5.2.3 Failure mechanisms to be regarded in the design of structures for storing refrigerated liquefied gases

The following ULS criteria apply:

- loss of overall equilibrium;
- failure of critical regions;
- instability resulting from large deformations;
- instability resulting from plastic or creep deformation.

The following SLS criteria apply:

- cracking and spalling;
- deformations;
- corrosion of reinforcement or deterioration of concrete;
- vibrations;
- tightness.

7.5.2.4 Concrete material properties under cryogenic conditions

7.5.2.4.1 Concrete compressive strength

The concrete cylinder compressive strength under a temperature Θ can be formulated as:

for Θ > -120°C $$f_c(\Theta, mc) = f_c + 12 \cdot mc \cdot [1 - (\frac{\Theta + 170}{170})^2] \qquad (7.5\text{-}4a)$$

for Θ ≤ -120°C $$f_c(mc) = f_c + 10.7 \cdot mc \qquad (7.5\text{-}4b)$$

where

Θ is the temperature in °C;

mc is the percentage of moisture content by weight.

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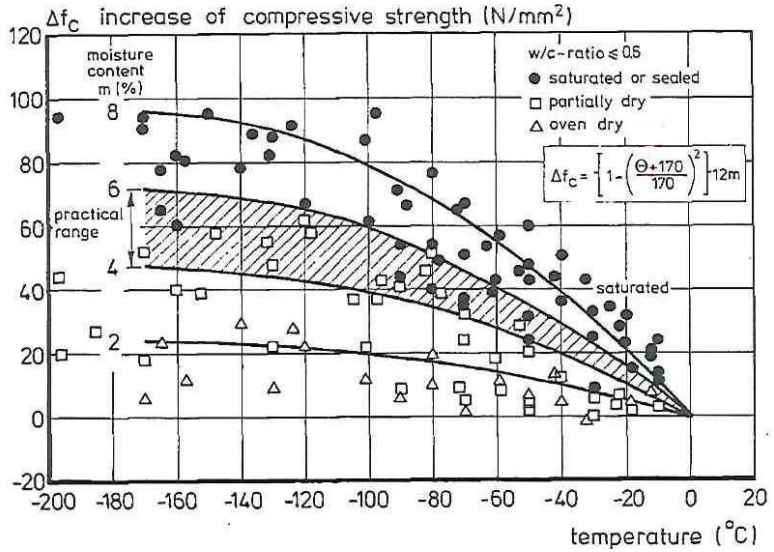


Figure 7.5-9: Increase of compressive strength as a function of temperature and moisture content (Van der Veen, "Cryogenic bond stress – strain relationship" PhD thesis TU Delft, 1990)

7.5.2.4.2 Compressive stress-strain relation

The compressive stress-strain relation is a function of the temperature and the moisture content. The strain $\varepsilon_{fc}(\Theta)$ at the ultimate strength at a temperature Θ can be expressed by:

$$\varepsilon_{f_c}(\Theta) = \varepsilon_{f_c}(\Theta = 20°C) + \kappa \cdot \Delta\varepsilon_{f_c}^{\max}(\Theta = -60°C) \qquad (7.5\text{-}5)$$

where:

$\Delta\varepsilon_{f_c}^{\max}(\Theta = -60°C)$ is the maximum increase in strain experienced at -60°C and the coefficient κ is defined as:

for Θ > –60°C $$\kappa = 1 - \left(\frac{\Theta - 60}{60}\right)^2 \qquad (7.5\text{-}6a)$$

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For an overview of the mechanical properties of concrete under cryogenic conditions reference is made to Rostasy, F.S., "Assessment of mechanical properties of structural materials for cryogenic application" (Braunschweig, iBMB, Institut für Baustoffe, Massivbua und Brandschutz, Technical University of Braunschweig, Report H 216, ISBN 978-3-89288-200-8).

for -170°C < Θ < -60°C $\kappa = \frac{\Theta - 170}{110}$ (7.5-6b)

In Eq. (7.5-5) it can be assumed that $\varepsilon_{f_c}(\Theta = 20°C) = 0.2\%$ and $\Delta\varepsilon_{f_c}^{\max}(\Theta = -60°C) = 0.1\%$ for Portland cement-based concretes and 0.15% for blast furnace cement-based concretes.

The following relationship can be used for predicting the effect of temperature on the stress-strain relation:

$$\sigma_c(\Theta) = f_c(\Theta, mc) \cdot [1 - (1 - \frac{\varepsilon_c(\Theta)}{\varepsilon_{f_c}(\Theta)})^n] \quad (7.5\text{-}7)$$

where:

σ_c (Θ) and ε_c (Θ) are stress and strain at a given point of the stress-strain curve;

$\varepsilon_{f_c}(\Theta)$ is the strain at ultimate strength defined in Eq. (7.5-5);

n is an exponent defined in Eq. (7.5-8a, b, c):

at Θ = 20°C $n = 2$ (7.5-8a)

for -170°C < Θ < 20°C $n = 1 + \frac{\Theta + 170}{170}$ (7.5-8b)

at Θ = -170°C $n = 1$ (7.5-8c)

7.5.2.4.3 Splitting tensile strength

The splitting tensile strength $f_{c,split}(\Theta)$ can be derived from the cylinder compressive strength $f_c(\Theta, mc)$ according to the relation:

$$f_{c,split}(\Theta, mc) = C \cdot f_c(\Theta, mc)^{0.67} \quad (7.5\text{-}9)$$

The coefficient *C* varies between 0.3 and 0.56 in case of air-dry and water saturated concrete.

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Neville (1995) reported *E*-moduli at a temperature of -190°C which were about 1.7 times as high as measured at room temperature. Van der Veen (1990) reported an increase of 1.15, 1.5 and 1.8 for air dried, sealed and saturated samples, respectively, exposed to -165°C .

For background information about bond and crack width control reference is made to Van der Veen, "Cryogenic bond stress – strain relationship" (PhD thesis TU Delft, 1990)

7.5.2.4.4 Modulus of elasticity

The modulus of elasticity increases with decreasing temperature, but not as significant as the concrete strength. However, there is significant scatter.

7.5.2.4.5 Coefficient of thermal expansion

The coefficient of thermal expansion decreases with decreasing temperature. For dry concrete this decrease is about 10%. The response of concrete stored at a RH above 86% will show a sudden decrease in the coefficient of thermal expansion. This decrease is dominated by the content of free moisture. For moist concrete, at a temperature of -100°C even an expansion can occur.

7.5.2.4.6 Creep

The creep of concrete decreases with decreasing temperature. At a temperature of -30°C creep is about 50% of that measured at room temperature. Because the elastic strains are also reduced with a reduction of temperature, however, the influence of temperature on the ratio between elastic and creep strains almost cancels itself out. Hence the general relationship between instantaneous (elastic) and long-term strain values still remains the same as at room temperature.

7.5.2.4.7 Bond and crack width control

The bond properties improve at decreasing temperature. Crack width control, carried out for room temperature conditions, gives a reasonable, slightly conservative, approach for the determination of crack widths in structures at cryogenic conditions.

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7.6 Verification of serviceability (SLS) of RC and PC structures

7.6.1 Requirements

It should be demonstrated that the structure and the structural elements perform adequately in normal use. To meet this requirement the serviceability limit states (SLS) should be verified.

Depending on the type and function of a structure or a structural element the verification of different serviceability limit states may be relevant, such as the limitation of:

- stresses (see subclause 7.6.3);
- crack widths (see subclause 7.6.4);
- deformations (see subclause 7.6.5);
- vibrations (see subclause 7.6.6).

7.6.2 Design criteria

For SLS verifications the partial safety coefficients are generally taken equal to 1.0.

The combination of loads to be considered depends on the type of SLS and on the specific problem. It is suitable to utilize one of the combinations given in subclause 4.5.2.5 under representative values of variable actions, being:

- characteristic value;
- combination value;
- frequent value;
- quasi-permanent value.

The serviceability limit states are listed in subclause 3.3.1.

The verification of SLS is performed under service load conditions and the operational failure probability to exceed the limit state is about a thousand times higher than that of ULS (see Table 3.3-5). However, if the SLS criteria are exceeded this does not yet mean that the structure should be rejected.

Exceeding the limit state of stresses or limit state of cracking may lead to limited local structural damage mainly affecting the durability of the structure, its tightness or its appearance.

Excessive deformations may produce damage in non-structural elements or load bearing walls and can affect the efficient use or appearance of structural or non-structural elements.

Vibrations may cause discomfort, alarm or loss of functionality.

For special problems other suitable combinations may be agreed by client and designer.

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In general for the limit state of deformations the mean value of the prestressing force at the time considered is appropriate, while for the limit state of cracking the upper or lower value (P_{sup} or P_{inf}) according to subclause 4.5.1.4.2 is suitable.

If the value of the prestressing force is known (e.g. from site measurement) the mean value of the prestressing force can be also used for the crack width analysis.

Linear or non-linear methods may be used. For most SLS problems linear analysis is sufficient. If, however, a non-linear analysis is carried out for ULS the action effects under service loads may be calculated by the same model. Plastic analysis is unsuitable for SLS calculations.

In selecting appropriate stress limits, the effect of the absolute dimensions of the member should be taken into account. Lower limits will be appropriate for larger members due to size-effects.

Further information is given in subclause 7.6.4.

Stresses are calculated using section properties corresponding to either the uncracked or the fully cracked condition, whichever is appropriate.

Where the maximum tensile stress in the concrete calculated on the basis of an uncracked section under the characteristic combination of loads exceeds f_{ctm} (see Table 7.2-1), the cracked state should be assumed.

Where the section is assumed to be uncracked, the whole concrete section is considered to be active and both concrete and steel are assumed to be elastic, both in tension and compression.

The relevant values of the prestressing force depend on the type of SLS and the problem considered. Prestressing force values to be considered are suggested in subclause 4.5.2.5.

For structural analysis any appropriate method may be used, which takes account of the material behaviour under service loads.

7.6.3 Stress limitation

Under service load conditions limitation of stresses may be required for

- tensile stresses in concrete;
- compressive stresses in concrete;
- tensile stresses in the steel.

The limitation of tensile stresses in concrete is an adequate measure to reduce the probability of cracking.

The limitation of compressive stresses in concrete aims at avoiding excessive compression, producing irreversible strains and longitudinal cracks (parallel to the compressive strains).

Tensile stresses in reinforcement should be limited with an appropriate safety margin below the yield strength, preventing uncontrolled cracking.

In calculating the stress, account shall be taken of whether the section is expected to crack under service loads. Moreover the effects of creep, shrinkage, relaxation of prestressing steel and differential temperatures should be regarded.

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In specific cases, e.g. in incremental launching with precast elements, a minimum compressive stress may be required.

As a rule the limit state of decompression should be required, if cracking or reopening of cracks has to be avoided under a given load combination.

In a beam the state of decompression is reached when the section under consideration is compressed and the extreme fibre concrete stress is equal to zero.

The occurrence of longitudinal cracks may lead to a reduction in durability. In the absence of other measures (such as an increase of concrete cover) it is recommended to limit the compressive stress for exposure classes XD, XS, and XF (section subclause 4.7.2). However, no limitation in service-ability conditions is necessary for stresses under bearings and anchorages through mechanical devices (e.g. anchor plate of prestressing tendons).

The limit of $0.6f_{ck}(t)$ is not a sharply defined value. Consequently, in the corresponding verification the prestressing force may be represented by its mean value, and in transient situations where the magnitude of variable actions is small (especially at transfer of prestressing forces in beams) the quasi-permanent combination of actions may be replaced by the characteristic combination. On the other hand the prestressing force and concrete strength should be considered in the verification with their values at the time at which the maximum stresses are reached.

Measures should be envisaged for deformations if the span/effective depth ratio exceeds 85% of the value given in Table 7.6-6.

If creep is likely to significantly affect the behaviour of the member considered (e.g. with regard to loss of prestress, deformation, validity of the structural analysis) an alternative measure would be to limit the stress to

7.6.3.1 Tensile stresses in the concrete

Depending on the limit state considered, various stress limitations may apply. The limit state of decompression is the most relevant. Stresses may be calculated on the basis of a homogeneous uncracked concrete section. The contribution of reinforcement to the area and the section modulus of the cross section may be taken into account.

7.6.3.2 Limit state of decompression

The limit state of decompression is defined as the state where concrete stresses are below or equal to zero in all principal directions.

7.6.3.3 Compressive stresses in the concrete

Excessive compressive stress in the concrete under service load may lead to longitudinal cracks and high and hardly predictable creep, with serious consequences to prestress losses. When such effects are likely to occur, measures should be taken to limit the stresses to an appropriate level.

If the stress does not exceed $0.6f_{ck}(t)$:

- under the characteristic combination, longitudinal cracking is unlikely to occur;
- under the quasi-permanent combination of actions, creep and the corresponding prestress losses can be predicted with adequate accuracy.

If under the quasi-permanent combination of actions the stress exceeds $0.4f_{ck}(t)$, non-linearity should be regarded for the assessment of creep deformation (see subclause 5.1.9.4 (d)).

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$0.4f_{ck}(t)$. However, the limitation may be taken as a value between $0.4f_{ck}(t)$ and $0.6f_{ck}(t)$ for verifications relating to a transient situation (e.g. during construction) depending on the duration of the loading.

Creep effects in a cracked cross-section may be taken into account by assuming a modular ratio of 15 for situations where more than 50% of the stress arises from quasi-permanent actions. Otherwise, they may be ignored.

This requirement will be met provided that, under the characteristic combination of loads, the tensile stress in the ordinary reinforcement does not exceed $0.8f_{yk}$. Where the stress is due entirely to imposed deformations, a stress of $1.0f_{yk}$ is acceptable.

Stress verifications should be carried out for fatigue effects in partially prestressed members.

Cracks may be due to other causes such as plastic shrinkage or chemical reactions accompanied by expansion of the hardened concrete. The avoidance and the control of the width of such cracks are not covered by this chapter.

It should be noted for the determination of the crack width in practice that the phenomena of cracking are of highly probabilistic nature. Calculated crack widths (w_d) are nominal values for comparison with nominal limit values (see definition of w_d and w_{lim} in 7.6.4.3). The actual crack widths observed on the real structure may differ from these nominal calculated values. Therefore the comparison of calculated crack widths with nominal crack widths limits may only serve as an approximate means to satisfy the design criteria. High accuracy may not be expected.

7.6.3.4 Steel stresses

Tensile stresses in the steel under serviceability conditions, which could lead to inelastic deformations of the steel should be avoided as this will lead to large, permanently open cracks. The allowed stress in steel is in accordance with subclause 5.4.4.2

7.6.4 Limit state of cracking

7.6.4.1 Requirements

It should be ensured that, with adequate probability, cracks will not impair the serviceability and durability of the structure.

Cracks do not, per se, indicate a lack of serviceability or durability; in reinforced concrete structures, cracking due to tension, bending, shear, torsion (resulting from either direct loading or restraint of imposed deformations) is often inevitable and does not necessarily impair serviceability or durability.

Design crack widths can be specified to satisfy requirements concerning functionality (subclause 7.6.4.1.1), durability (subclause 7.6.4.1.2) or appearance (subclause 7.6.4.1.3).

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Loss of functionality occurs, for instance, in liquid retaining or containment structures, where leakage can occur if the crack widths are too large.

For containment structures a concrete compression zone with a specified minimum height can be specified, or a lining can be provided in more demanding cases.

The durability of a structure is predominantly governed by the thickness of the concrete cover and the quality of the concrete if the crack widths are limited to the maximum characteristic crack widths given in subclause 7.6.4.3.

Limit values of crack widths should be agreed with the owner.

If the cover is larger than required for the environmental conditions concerned, those practical limit values for the crack width can even be enlarged proportionally.

In some particular cases more severe demands should be formulated in agreement with the owner, for instance if de-icing agents are frequently used on top of tension zones in reinforced or partially prestressed structures.

The concrete may have a higher strength than was ordered. A higher tensile strength of the concrete may lead to larger crack widths. Another uncertainty is the development of the tensile strength in time due to the continued hydration of the cement, in combination with the loading scenario.

7.6.4.1.1 Functional requirements

The functionality of the structure should not be impaired by the cracks formed.

In relevant cases, nominal crack widths (w_{lim}) may be agreed with the client.

7.6.4.1.2 Durability

The durability of the structure during its intended lifetime should not be impaired by the cracks formed.

7.6.4.1.3 Appearance of the structure

The appearance of the structure should not be unacceptable because of cracking.

7.6.4.1.4 Uncertainties

Uncertainties related to the actual local concrete tensile strength, as well as to unforeseen tensile stresses, should be appropriately considered in design and construction.

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At all sections which are expected to be subjected to significant tension (due to imposed or restrained deformation, combined or not combined with direct loading), a minimum amount of reinforcement should be placed, to ensure that the reinforcement does not yield immediately after cracking in the SLS.

This applies also to prestressed members in regions where tension is expected to develop in the concrete in the SLS.

Different combinations of actions may be considered under particular conditions.

(a) For reinforced concrete members, for exposure classes XC, XD, XF, XS (as specified in subclause 4.7.2), a w_{lim} = 0.30 mm may be assumed satisfactory under the quasi-permanent combination of actions with respect to both appearance and durability, in the absence of specific requirements (e.g. water tightness).
For exposure class X0, this limit may be relaxed provided that it is not necessary for reasons other than durability.

When de-icing agents are expected to be used on top of tensioned zones of reinforced concrete elements, appropriate w_{lim}-values should be specified in accordance with the client, depending on the thickness and quality of the concrete, and of additional protective layers.

(b) For prestressed members, if more detailed data are not available, the crack width limit values presented in Table 7.6-1 may be used.

7.6.4.2 Design criteria versus cracking

(a) The specific requirements of subclauses 7.6.4.1.1 to 7.6.4.1.4 may be met by an appropriate limitation of crack widths. This may be achieved either by means of analytical procedures (subclause 7.6.4.4) or by appropriate practical rules (subclause 7.6.4.7).

(b) The rules given in this chapter only apply if a minimum reinforcement is provided according to clause 7.13 if stress limitation requirements according to subclause 7.6.3 are fulfilled.

7.6.4.3 Limitation of crack width

In order to meet the demands with regard to functionality, durability and/or appearance the crack width has to satisfy the following condition:

$$w_d \leq w_{\lim} \qquad (7.6\text{-}1)$$

where

w_d denotes the design crack width calculated as in subclause 7.6.4.4.1 under the appropriate combination of actions (subclause 4.5.2.5) considered at the concrete surface;

w_{lim} denotes the nominal limit value of crack width considered at the concrete surface, which is specified for cases of expected functional, appearance related or in some particular cases durability related consequences of cracking. The nominal limit value of crack width corresponds to the nominal concrete cover c_{nom} according to subclause 7.13.2.2.

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Table 7.6-1: Crack width limits (mm) for reinforced members and for prestressed members with bonded prestressing steel

	RC	PL1	PL2	PL3
X0	0.3	0.2	0.3	0.3
XC	0.3	0.2	0.3	0.3
XD	0.2	$\sigma < 0$ *	0.2	0.2
XS	0.2	$\sigma < 0$ *	0.2	0.2
XF	0.2	$\sigma < 0$ *	0.2	0.2

For PL1, PL2 and PL3 see subclause 5.4.3.3.

* Stress in concrete at the level of prestressed reinforcement

For structural members subjected to exposure classes XD, XF, XS and bending, no tension is allowed in a distance of 100 mm from the duct or the perimeter of the prestressed reinforcement towards the tension face. This value may be reduced to 25 mm in cases where the overall height of the member is below 400 mm.

With regard to crack width limitation for fluid-tightness, different requirements may apply for various cases (0.15 – 0.25 mm). Special recommendations should be used. The requirement generally depends on the consequences of leakage and the pressure of the fluid:

– If leakage should be limited to a small amount and some surface staining is acceptable w_{lim} = 0.20 mm may be used as a limit for self-healing cracks. Otherwise 0.1 mm may be more appropriate.

– If leakage should be minimal, no continuous cracks are allowed and a compression zone of at least 50 mm should be available.

Whether self-healing of cracks can occur depends on the chemical composition of the fluid, type of cement, water pressure, time after subjecting to water pressure, etc.

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Longitudinal cracks due to the corrosion of steel bars are not covered by these criteria; they should be avoided by provisions for ensuring durability.

Figure 7.6-1 shows the basic behaviour of a reinforced prism subjected to increasing axial deformation. In the cracks the steel has to carry the full tensile load. To both sides of the crack the load is partially transmitted to the concrete. At a distance $l_{s,max}$ at both sides of the crack, the undisturbed situation is reached again. Under increasing deformation more cracks are formed gradually.

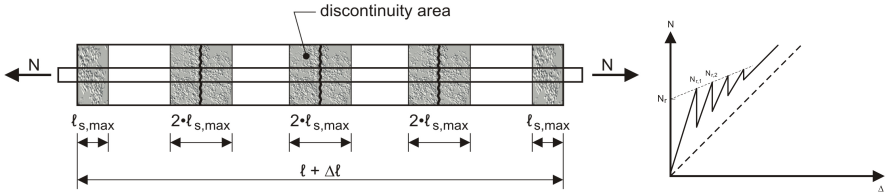


Figure 7.6-1: Behaviour of a reinforced prismatic bar subjected to imposed deformation

For the control of longitudinal cracks (parallel to the main steel bars), the following design criteria apply:

- the thickness of the concrete cover as well as, where necessary, the secondary reinforcement (skin reinforcement) transverse to the main steel bars should be appropriately selected (as a function of their diameter), in order to secure the full development of the bond resistance without any longitudinal cracking;
- for elements reinforced in two directions, tensile stresses generated in sections parallel to the direction of a steel bar should be appropriately limited.

7.6.4.4 Calculation of crack width in reinforced concrete members

7.6.4.4.1 General

The crack width calculations are based on the basic case of a prismatic reinforced concrete bar, subjected to axial tension. With regard to the behaviour under increasing tensile strain four stages are distinguished:

- the uncracked stage;
- the crack formation stage;
- the stabilized cracking stage;
- the steel yielding stage.

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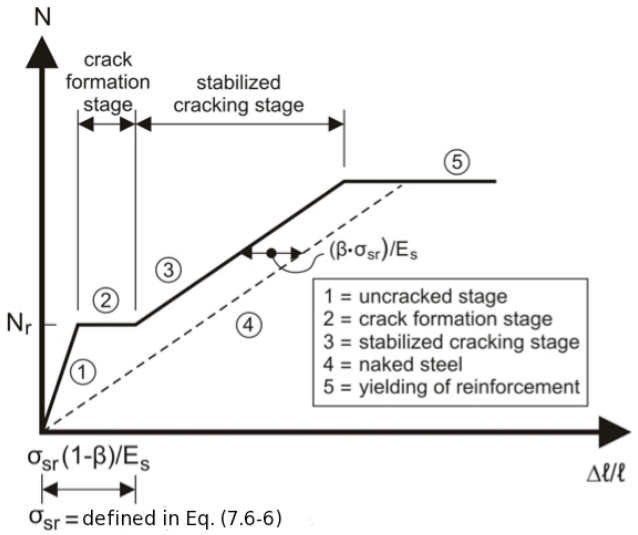


Figure 7.6-2: Simplified load – strain relation for a centrically reinforced member subjected to tension

Figure 7.6-2 shows a simplified representation of the load-deformation behaviour of a centrally reinforced member subjected to tension or imposed deformation. According to the simplification, in the crack formation stage (2) the axial tensile force does not increase. When enough cracks have been formed to ensure that no undisturbed areas (white areas in Figure 7.6-1) are left, the tensile strength of the concrete cannot be reached anymore between the cracks, so that no new cracks can appear. This is the start of the stabilized cracking stage (3). In this stage no new cracks are formed but existing cracks widen. Finally the steel will start yielding at stage (5)

For the model, as a simplification line (2) has been assumed to be horizontal, corresponding to a constant tensile cracking force:

$N_r = A_{c,ef} \cdot f_{ctm}(1 + \alpha_e \rho_{s,ef})$.

In reality this line will not be horizontal but inclined, ranging from the first crack for $f_{ctk,0.05}$ to an upper value of $f_{ctk,0.95}$ (sawtooth line in Figure 7.6-1 right).

For carrying out crack width calculations it is necessary to know whether the crack formation stage or the stabilized cracking stage applies.

According to the simplified approach (Figure 7.6-2) the stabilized cracking stage applies when the load is larger than the cracking load ($N > N_r$). For imposed deformations, the crack formation stage applies when the mean strain satisfies the following condition:

$$\varepsilon = \frac{\Delta L}{L} \le \frac{\sigma_{sr}(1-\beta)}{E_s} \qquad (7.6\text{-}2)$$

where

σ_{sr} follows from Eq. (7.6-6);

β is a coefficient to assess the mean steel strain over $l_{s,max}$, depending on the type of loading (instantaneous, long term, repeated, see Table 7.6-2);

$\rho_{s,ef}$ is effective reinforcement ratio for tensile bar = $A_s/A_{c,ef}$.

If the mean strain is larger than this value, the stabilized cracking stage applies. Under imposed deformation the stabilized cracking stage is usually not reached.

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The approach with the horizontal line (2) is regarded to be accurate enough, considering the influence of a number of uncertainties, such as the accuracy of placement of reinforcement, the real effective tensile strength and the influence of construction quality.

The transmission of forces in a disturbed area adjacent to a crack is shown in Figure 7.6-3. Again the relations are simplified by linearization.

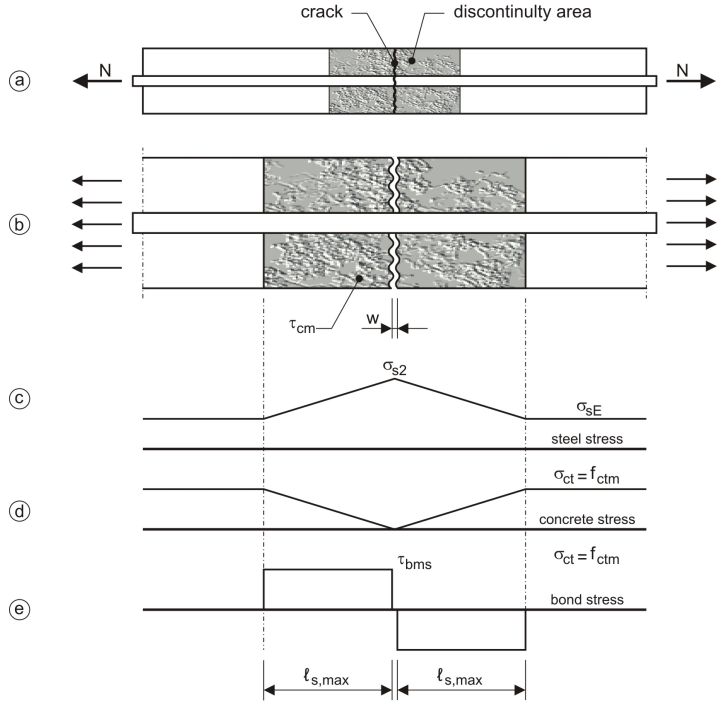


Figure 7.6-3: Steel, concrete and bond stresses in disturbed area in the crack formation stage (simplified representation)
a) Centrically reinforced tensile member with crack; b) Discontinuity area; c) Steel stress development in discontinuity area; d) Concrete stress development in discontinuity area; e) Development of bond stress in discontinuity area

For all stages of cracking, the design crack width w_d may be calculated by:

$$w_d = 2l_{s,\max}(\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs}) \tag{7.6-3}$$

where:

$l_{s,max}$ denotes the length over which slip between concrete and steel occurs. The steel and concrete strains, which occur within this length, contribute to the width of the crack; $l_{s,max}$ is calculated with Eq. (7.6-4);

ε_{sm} is the average steel strain over the length $l_{s,max}$;

ε_{cm} is the average concrete strain over the length $l_{s,max}$;

ε_{cs} is the strain of the concrete due to (free) shrinkage.

For the length $l_{s,max}$ the following expression applies:

$$l_{s,\max} = k \cdot c + \frac{1}{4} \cdot \frac{f_{ctm}}{\tau_{bms}} \cdot \frac{\phi_s}{\rho_{s,ef}} \tag{7.6-4}$$

where:

k is an empirical parameter to take the influence of the concrete cover into consideration; as a simplification, k = 1.0 can be assumed;

c is the concrete cover;

τ_{bm} is mean bond strength between steel and concrete (Table 7.6-2).

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The formulation given for the value of the crack width (Eqs. (7.6-3) to (7.6-5)) provides an estimate for the surface crack width for members subjected to pure tension. For members subjected to bending, the values represent the crack width at the level of the reinforcement. In the latter case crack spacing and crack width will generally be larger at the extreme tensile fibre. In order to estimate the value of the crack width at the extreme tensile fibre, the crack width may be multiplied with a factor (h-x)/(d-x)

Equation 7.6-4 is valid for structures, where the concrete cover is not larger than 75 mm. For a larger concrete cover a more detailed analysis is required. Procedures based on the fracture mechanics approach would be appropriate. According to the contemporary knowledge k = 1 may be used in Eq. 7.6-4.

In the absence of a more refined model Fig. 7.6-4 may be used to assess the effective concrete area in tension.

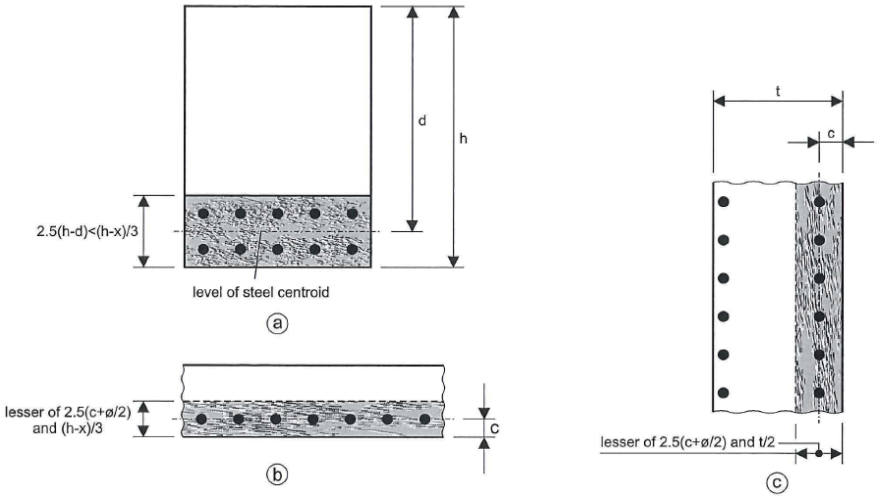


Figure 7.6-4: Effective tension area of concrete $A_{c,ef}$ for: (a) beam; (b) slab; (c) wall in tension (shaded areas)

The relative mean strain in Eq. (7.6-3) follows from:

$$\varepsilon_{sm} - \varepsilon_{cm} - \varepsilon_{cs} = \frac{\sigma_s - \beta \cdot \sigma_{sr}}{E_s} + \eta_r \cdot \varepsilon_{sh} \qquad (7.6\text{-}5)$$

where:

σ_s is the steel stress in a crack;

σ_{sr} is the maximum steel stress in a crack in the crack formation stage, which, for pure tension, is:

$$\sigma_{sr} = \frac{f_{ctm}}{\rho_{s,ef}}(1 + \alpha_e \rho_{s,ef}) \qquad (7.6\text{-}6)$$

where:

$$\rho_{s,ef} = \frac{A_s}{A_{c,ef}}$$

with $A_{c,ef}$ = effective area of concrete in tension (Fig. 7.6-4);

α_e is the modular ratio = $\frac{E_s}{E_c}$;

β is an empirical coefficient to assess the mean strain over $l_{s,max}$ depending on the type of loading (Table 7.6-2);

η_r is a coefficient for considering the shrinkage contribution;

ε_{sh} is the shrinkage strain.

The value for τ_{bms} and the coefficients β and η_r can be taken from Table 7.6-2.

Table 7.6-2: Values for τ_{bms}, β and η_r for deformed reinforcing bars

	Crack formation stage	**Stabilized cracking stage**
Short term, instantaneous loading	$\tau_{bms} = 1.8 \cdot f_{ctm}(t)$ $\beta = 0.6$ $\eta_r = 0$	$\tau_{bms} = 1.8 \cdot f_{ctm}(t)$ $\beta = 0.6$ $\eta_r = 0$
Long term, repeated loading	$\tau_{bms} = 1.35 \cdot f_{ctm}(t)$ $\beta = 0.6$ $\eta_r = 0$	$\tau_{bms} = 1.8 \cdot f_{ctm}(t)$ $\beta = 0.4$ $\eta_r = 1$

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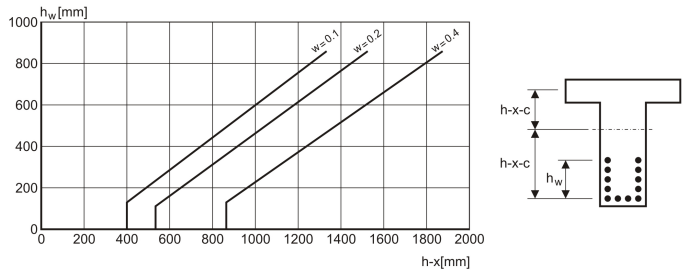


Figure 7.6-5: The height of the web that should be provided with crack distributing reinforcement in order to prevent wide cracks outside the effective tension area (illustrative diagram)

When a more refined model is not available, the following expression for $l_{s,max}$ may be used:

For the stabilized cracking stage, if different bar diameters are used in the tensile area, the value $Ø_s$ in Eq. 7.6-4 may be replaced by $Ø_{eq}$ according to:

$$Ø_{eq} = \frac{\Sigma n_{s,i} Ø_{s,i}^2}{\Sigma n_{s,i} Ø_{s,i}} \qquad (7.6\text{-}7)$$

where:

$n_{s,i}$ is the number of bars with diameter $Ø_{s,i}$.

The effective concrete tension area $A_{c,ef}$ accounts for the non-uniform normal stress distribution by bond forces in the concrete cross-section at the end of the transmission length.

With the method given in this section, the design crack width within the effective tension area is controlled. Outside this area larger cracks may occur; this can be avoided by providing crack distributing reinforcement.

7.6.4.4.2 Combined effects of load and imposed deformations

The effect of cracking may be considered for the analysis of combined effects of loads and imposed deformations. Hence, where cracking is due to imposed deformations and loads, the steel stresses at the cracks due to loads as well as imposed deformations should be taken into account.

7.6.4.4.3 Orthogonal reinforcement directions

If the cracks in a member reinforced in two orthogonal directions are expected to form at an angle which differs substantially (> 15°) from the direction of the reinforcement, the approximation by Eq. (7.6-8) and (7.6-9) may be used to calculate $l_{s,max}$ and w_d.

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$$\ell_{s,\max,\theta} = \left(\frac{\cos\theta}{\ell_{sx,k}} + \frac{\sin\theta}{\ell_{sy,k}} \right)^{-1} \tag{7.6-8}$$

where:

θ denotes the angle between the reinforcement in the x-direction and the direction of the principal tensile stress;

$\ell_{sx,k}$, $\ell_{sy,k}$ denote the slip lengths in the two orthogonal directions, calculated according to Eq. (7.6-4).

The design crack width can then be calculated from:

$$w_d = 2 \cdot l_{s,\max,\theta} (\varepsilon_{\perp} - \varepsilon_{c,\perp}) \tag{7.6-9}$$

where:

$\varepsilon_{\perp}$ and $\varepsilon_{c,\perp}$ represent the mean strain and the mean concrete strain, evaluated in the direction orthogonal to the crack (Figure 7.6-6).

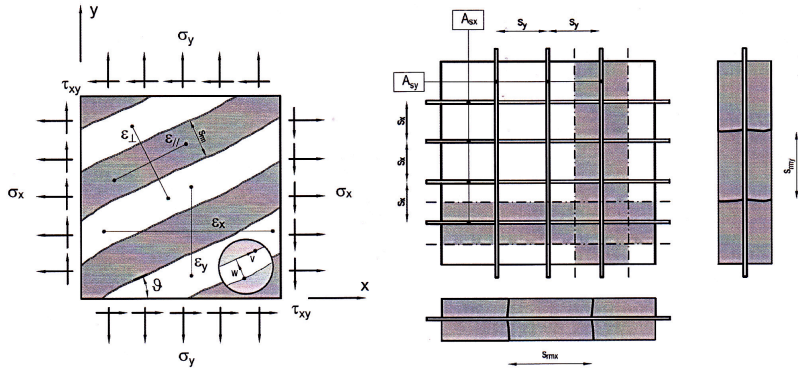


Figure 7.6-6: Basis for calculation of crack width for reinforcement deviating from the direction orthogonal to the crack

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When prestressed and non-prestressed types of steel are simultaneously used, since the bond behaviour of prestressing tendons is different from the bond behaviour of deformed reinforcing bars, different steel stresses will be developed in each type of steel. For the calculation of the equivalent effective reinforcing ratio $\rho_{s+p,ef}$ both equilibrium and compatibility are respected, Figure 7.6-7 and 7.6-8.

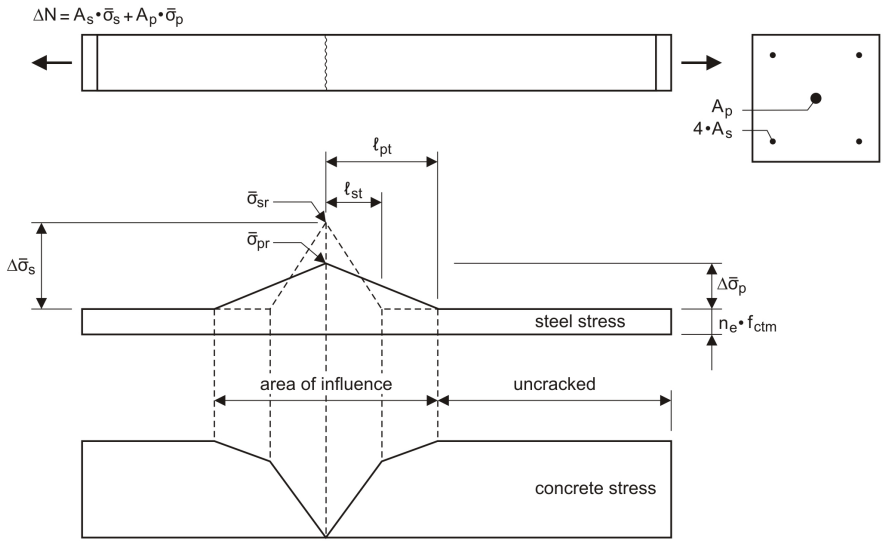


Figure 7.6-7: Crack formation stage: development of the steel and concrete stresses beyond decompression for a combination of reinforcing steel and prestressing steel

7.6.4.5 Calculation of crack width in prestressed concrete members

The calculation of crack width concerns structural members with bonded prestressing reinforcement.

In general, the calculation of crack widths for prestressed structural members follows generally the procedure and the formulae given for reinforced concrete members in subclause 7.6.4.4.

For the general case of combined reinforcement consisting of reinforcing steel and prestressing steel, the prestressing steel is replaced by an equivalent cross sectional area of reinforcing steel, which takes the lower bond quality of the prestressing steel into account. So, Eq. (7.6-3) is extended to the more general case of reinforcing and prestressing steel by replacing $\rho_{s,ef}$ by $\rho_{s+p,ef}$, where:

$$\rho_{s+p,ef} = \frac{A_s + \xi_1^2 \cdot A_p}{A_{c,ef}} = \rho_s + \xi_1^2 \cdot \rho_p \qquad (7.6\text{-}10)$$

where ξ is a bond factor according to:

$$\xi_1 = \sqrt{\frac{\tau_{bmp}}{\tau_{bms}} \cdot \frac{\phi_s}{\phi_{p,eq}}} \qquad (7.6\text{-}11)$$

and the equivalent diameter $\phi_{p,eq}$ of the prestressing steel is given as:

$$\phi_{p,ef} = \frac{4 \cdot A_p}{u_p}$$

where:

$A_p = \Sigma A_{p,i}$ is the total actual area of the prestressing steel;

$u_p = \Sigma u_{p,i}$ is the total equivalent perimeter of the prestressing steel;

where:

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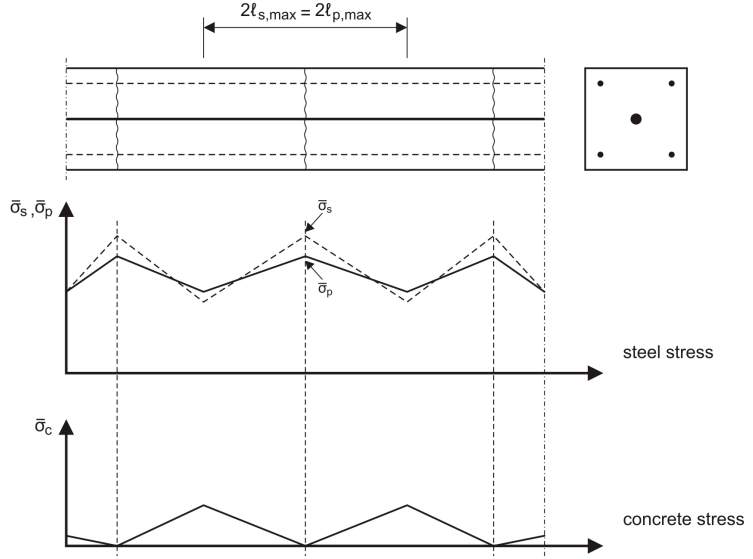


Figure 7.6-8: Stabilized cracking stage: development of the steel and concrete stresses beyond decompression for a combination of reinforcing steel and prestressing steel

For reinforced or prestressed slabs subjected to bending without significant axial tension, no special measures to control cracking are needed, provided that the overall depth of the slab does not exceed 200 mm.

If the values given in Table 7.6-4 and Table 7.6-5 are respected, crack widths do generally not exceed 0.30 mm for reinforced elements and 0.20 mm for prestressed elements.

$u_{p,i} = 1.6 \cdot \pi \cdot \sqrt{A_{p,i}}$ for bundles;

$u_{p,i} = 1.75 \cdot \pi \cdot \phi_{wire}$ for 7-wire strands;

$u_{p,i} = 1.20 \cdot \pi \cdot \phi_{wire}$ for 3-wire strands;

where ϕ_{wire} is the single wire diameter in the strands.

The maximum transmission length is given by Eq. (7.6-4) replacing $\rho_{s,ef}$ by $\rho_{s+p,ef}$.

The relative mean strain in the cracked section beyond decompression is given by Eq. (7.6-5) replacing $\rho_{s,ef}$ by $\rho_{s+p,ef}$.

In the absence of more detailed data approximate values for τ_{bmp}/τ_{bms} can be taken from Table 7.6-3.

Table 7.6-3: Bond factors τ_{bmp}/τ_{bms} for different types of prestressing steel

Surface condition	Pretensioned steel	Posttensioned steel	No bond
Smooth wire	0.40	0.20	0
Strand	0.60	0.40	0
Indented wire	0.80	0.60	0
Ribbed bar	—	1.00	0

7.6.4.6 Control of cracking without calculation

Under well specified conditions, the fulfilment of the requirements in subclauses 7.6.4.1, 7.6.4.2 and 7.6.4.3 may also be achieved by means of appropriate practical rules:

– when small depth elements subjected mainly to bending are considered, no special measures are needed for crack control;

– under the condition that the minimum reinforcement specified in clause 7.13 (minimum reinforcement) is provided, the design crack width may be kept to acceptably low values, if appropriately chosen bar diameters and bar spacings are used.

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– For cracking caused mainly by restraint (crack formation stage), crack widths will generally not exceed the values stated above, provided the bar sizes given in Table 7.6-4 are not exceeded; the σs values referred to in Table 7.6-4 have to be calculated at cracking of the element (σ_{sr}).
– For cracks caused mainly by imposed loads (stabilized cracking stage), crack widths will generally not exceed the values stated above, provided either the provisions of Table 7.6-4 or those of Table 7.6-5 are satisfied.

Table 7.6-4: Maximum bar diameters (deformed bars) for crack width control without calculation for reinforced sections.

Steel Stress σ_s (MPa)	**Maximum bar diameter (mm)**
160	40
200	30
240	20
280	14
320	10
360	8
400	6
450	4

Note: The steel stresses are calculated under the quasi-permanent loads (reinforced concrete). The values in Table 7.6-4 have been obtained under the following assumptions:
(i) The reinforcement provided is the minimum amount of reinforcement in pure bending. Larger reinforcement values will provide larger values for the maximum bar diameter.
(ii) (h-d) is taken as approximately 0.10 h.
(iii) In order to evaluate the reduction in tensile strength due to self-equilibrating stresses, a height of 400 mm has been assumed

Further guidance concerning the choice of the bar diameter is given in clause 7.13.

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As a simplification for prestressed concrete sections, the stress increase of the prestressing steel i.e. the contribution of tendons to the limitation of crack widths, may be disregarded.

For reinforced concrete with an average concrete tensile strength other than f_{ctm} = 2.9 N/mm^2 the maximum bar diameter according to Table 7.6-4 may be modified as follows:

– for restraint cracking (crack formation stage):

$$\phi = \phi_{s,max} \frac{f_{ctm}(t)}{2.9} \tag{7.6-12}$$

– for load induced cracking (stabilized cracking stage):

$$\phi_s = \phi_{s,\max} \frac{h_t}{7.5(h-d)} > \phi_{s,\max} \tag{7.6-13}$$

where:

ϕ_s is the adjusted maximum bar diameter;

$\phi_{s,\max}$ is the maximum bar size given in the Table 7.6-4;

h is the overall depth of the section;

h_t is the depth of the tension zone just before cracking;

d is the effective depth of the cross section;

$f_{ctm}(t)$ is the mean value of the concrete tensile strength at the time t when the first crack appears.

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Table 7.6-5: Maximum bar spacing for crack width control without calculation for reinforced sections

Steel Stress σ_s (MPa)	Maximum bar spacing (mm)
160	300
200	300
240	250
280	150
320	100
360	60

Note: The values in this table have been obtained under the same assumptions as those described in Table 7.6-4, assuming a height of 400 mm for the cross section.

To establish such limits is not within the scope of this Model Code. However, some practical rules are given in subclause 7.6.5.2.3 for some categories of simple buildings.

The deformations must be accommodated by other connected elements such as partitions, glazing, claddings, services or finishes. In some cases a limitation may be required to ensure the proper functioning of machinery or apparatus supported by the structure, or to avoid ponding on flat roofs. Where applicable, acceptable limit values should be established in agreement with the client or his representative.

7.6.5 Limit states of deformation

7.6.5.1 General

7.6.5.1.1 Requirements

In-service deformations (deflections and rotations) may impair:

- the appearance of the structure;
- the integrity of non-structural parts;
- the proper functioning of the structure or its equipment.

To avoid excessive deformations appropriate limiting values should be respected.

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In general, the appearance and general utility of the structure can be impaired, if the calculated sag of a beam, slab or cantilever subjected to quasi-permanent loads exceeds span/250. The sag is assessed relative to the supports. Pre-camber may be used to compensate some or all of the deflection, but any upward deflection incorporated in the formwork should not generally exceed span/250.

Deflections that could damage adjacent parts of the structure should be limited. For the deflection increment after the installation of adjacent construction parts, span/500 is normally an appropriate limit for quasi-permanent loads. Other limits may be considered, depending on the sensitivity of the adjacent parts.

In order to ensure a satisfactory behaviour in the serviceability limit state, deformations should be calculated as follows:

- the long-term deformations are calculated for the quasi-permanent load combinations;
- the instantaneous deformations should be calculated for the rare load combinations.

For the calculation of camber, only the quasi-permanent load combinations are considered.

In order to calculate camber, the mean values of the material properties may be used.

7.6.5.1.2 Combination of actions

The combinations of actions to be considered depend on the criteria in question and are defined in subclause 7.6.2.

7.6.5.1.3 Data for the materials

The values of the material properties to be applied depend on the criteria in question.

In order to prevent damage due to deformations, prudent values of the material properties should be used.

7.6.5.1.4 Modelling

Depending on the precision needed, appropriate deformation models should be used, as described in the following subsections.

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The actual deformations may differ considerably from the calculated values; in particular if the values of the applied moments are close to the cracking moment. The difference will depend on the dispersion of the material properties, the ambient conditions, the loading conditions, the previous loading conditions, the restraints at the supports, etc.

Attention must be paid to cases where the basic assumptions of plane sections and uniformly distributed stresses across the section may not be adequate, such as in the case of shear lag effects in large prestressed structures.

For prestressed concrete members it may be necessary to control deflections assuming unfavourable deviations of the prestressing force and the dead load.

Most prestressed concrete flexural members will have a net positive (upward) camber at the time of transfer of prestress, caused by the eccentricity of the prestressing force. This camber may increase or decrease with time, depending on the stress distribution across the member under sustained loads.

In members composed of parts made of different concretes, such as a precast beam with a cast-in-place slab, the sectional properties of the composite transformed section must be obtained by summing the contribution of the different parts taking into account the different moduli of elasticity of each concrete.

7.6.5.2 Deformations due to bending with or without axial force

7.6.5.2.1 General methods

The most general method to assess deformations is to perform a non-linear analysis capable of calculating the instantaneous and time-dependent deformations, taking into account the effects of applied loads and prestressing, the non-linear behaviour of concrete and steel and the time-dependent deformations due to creep and shrinkage of concrete and the relaxation of prestressing steel. These methods require, in general, iterative and incremental procedures, necessary to fulfil the equilibrium and compatibility conditions and the materials constitutive properties at any load level and instant of time.

In structures, where the construction process requires an accurate estimation of deflections, a step-by-step analysis should be performed including the sequence of loading and possible changes in geometry, structural scheme, support conditions and the evolution of material properties with time.

Deflections and axial displacements are obtained by the integration of curvatures and axial strains along the member length.

(a) Instantaneous deflections

Members which are not expected to be loaded above the level which would cause the tensile strength of the concrete to be reached should be considered to be uncracked, and to have a linear elastic response.

Members loaded above the cracking load level are expected to have a behaviour between the uncracked and the fully cracked stages. Thus, the instantaneous mean axial strains and mean curvatures due to axial loads and bending moments can be obtained with a non-linear sectional analysis, which includes cracking of concrete and tension stiffening, and assumes plane sections and a perfect bond between the concrete and the reinforcement.

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In a cracked section under constant bending moment, changes in the stresses, strains and position of the neutral axis occur due to creep and shrinkage as shown in Figure 7.6-9.

For uncracked members, it can be assumed that the creep deflections are proportional to the instantaneous deflections due to permanent loads, unless a large amount of reinforcement exists.

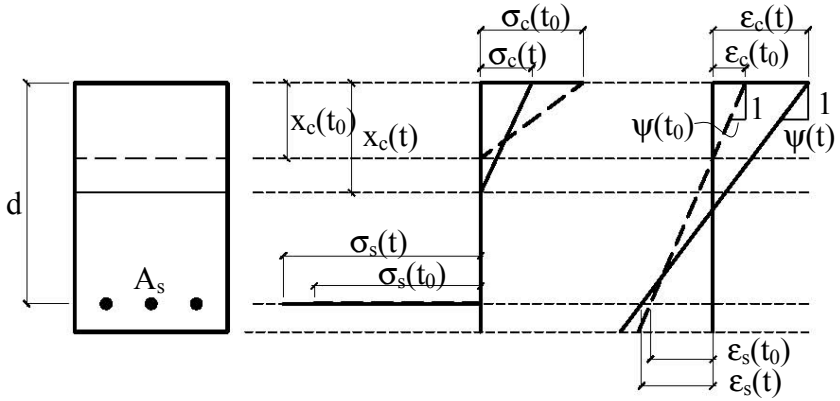


Figure 7.6-9: Stresses and strains at times t_0 and t, due to creep effect, in the presence of a constant bending moment

General methods are described in the CEB Bulletin d'Information No. 158, "Cracking and Deformations", Lausanne, 1985, and CEB Bulletin d'Information No. 235, "Serviceability Models", Lausanne, 1997. These bulletins contain the equations necessary to find the unknown parameters of the problems stated above.

The deflections of composite structures should consider the curvatures resulting from the differential shrinkage of precast and cast-in-situ components.

The time-dependent deflections are influenced by environmental and curing conditions, the age at time of loading, amount of compression reinforcement, magnitude of the stresses due to sustained loading and

(b) Long-term deflections

In order to obtain the delayed deflections, the increment of curvatures and axial strains with time must be obtained and integrated along the member length. Thus, a time-dependent sectional analysis is required which incorporates the effects of creep and shrinkage of concrete and the relaxation of prestressing steel, as well as their interaction with cracking of concrete and tension stiffening, among other non-linearities. For the usual level of concrete stresses at service, the principles of linear visco-elasticity are accepted (see subclause 7.2.4).

For uncracked members, it can be assumed that the creep deflections are proportional to the instantaneous deflections due to permanent loads, unless a large amount of reinforcement exists.

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prestressing as well as strength gain of concrete after release of prestress. In particular, camber is especially sensitive to the concrete properties at the age of release of prestress, level of stresses, storage method, time of erection, placement of superimposed loads and environmental conditions.

The deformation a_I in the uncracked state in bending is calculated using the member stiffness $(E_cI_c)_I$, where E_c is the E-modulus of the concrete for the loading type considered (instantaneous or long-term) and I_c is the moment of inertia of the uncracked cross section.

The deformation a_{II} in the cracked state in bending is calculated using the stiffness of the cracked member, assuming that no tension stiffening occurs. For a member of rectangular section the stiffness in the cracked state is equal to:

$$(EI)_{II} = d^2\left(1-\frac{x}{d}\right)\left(1-\frac{1}{3}\frac{x}{d}\right)A_{s1}E_s + d'^2\left(\frac{x}{d'}-1\right)\left(\frac{x}{d'}\frac{1}{3}-1\right)A_{s2}E_s \qquad (7.6\text{-}15)$$

where

$$\frac{x}{d} = \alpha_e\,\rho_1\left(1+\frac{\rho_2}{\rho_1}\right)\left(-1+\sqrt{1+\frac{2\left(1+\frac{\rho_2 d'}{\rho_1 d}\right)}{n\,\rho_1\left(1+\frac{\rho_2}{\rho_1}\right)^2}}\right)$$

When there is only tensile reinforcement:

$$(EI)_{II} = d^2(1-\frac{x}{d})(1-\frac{1}{3}\frac{x}{d})A_{sl}E_s \qquad (7.6\text{-}17)$$

where

7.6.5.2.2 Simplified method for RC structures

Members which are not expected to be loaded above the level which would cause the tensile strength of the concrete to be exceeded anywhere within the member should be considered to be uncracked. Members which are expected to crack, but may not be fully cracked, will behave in a manner intermediate between the uncracked and fully cracked conditions and, for members subjected mainly to flexure, an adequate prediction of the deformation is given by the expression:

$$a = \zeta a_{II} + (1-\zeta)a_I \qquad (7.6\text{-}14)$$

where

a is the deformation parameter considered which may be, for example, a strain, a curvature, or a rotation. As a simplification, a may also be taken as deflection.

a_I, a_{II} are the values of the deformation calculated for the uncracked and fully cracked conditions respectively;

ζ is an interpolation coefficient (allowing for the effect of tension stiffening at a section) given by the expression:

$$\zeta = 1-\beta\left(\frac{\sigma_{sr}}{\sigma_s}\right)^2 \qquad (7.6\text{-}16)$$

β is a coefficient accounting for the influence of the duration of loading or repeated loading on the average strain:

β = 1.0 for a single short-term loading;

β = 0.5 for sustained loads or multiple cycles of repeated loading;

σ_s is the stress in the tension reinforcement calculated on the basis of a cracked section under the load considered;

σ_{sr} is the stress in the tension reinforcement calculated on the basis of a cracked section under the loading conditions that cause first cracking.

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$$\frac{x}{d} = -\alpha_e \rho + \sqrt{(\alpha_e \rho_l)^2 + 2\alpha_e \rho_l}$$

if the concrete in the compression area is still in the elastic state.

Alternatively, instead of interpolation, an equivalent stiffness deduced from (Eq. 7.6-14) can be used for direct simplified calculation of deflection:

$$(EI)_{eff} = \frac{(EI)_I \cdot (EI)_{II}}{\zeta \cdot (EI)_I + (1-\zeta) \cdot (EI)_{II}}$$

For calculating long-term deflections due to creep and shrinkage the following simplified procedure can be used:

$a_t = (1+\varphi)\, a_g$ for $M_{Ed} < M_{cr}$

$a_t = a_g + a_\varphi + a_{sh}$ for $M_{Ed} \geq M_{cr}$

where

φ is the creep coefficient (see subclause 5.1.9.4.3);

a_g is the instantaneous deflection due to quasi- permanent loads;

a_φ is the creep deflection;

a_{sh} is the shrinkage deflection.

The stresses σ_s and σ_{sr} for the interpolation coefficient ζ are calculated at the most unfavourable section which is usually the section subjected to the maximum bending moment.

σ_{sr} / σ_s in Eq. (7.6-16) may be replaced by M_r / M for flexure and N_r / N for pure tension, where M_r is the cracking moment and N_r is the cracking force. M and N represent the moment and normal force for the load combination considered.

For loads with a duration long enough to cause creep, the total deformation including creep is obtained by using an effective modulus of elasticity of concrete according to:

$$E_{c,ef} = \frac{E_{cm}}{1+\varphi} \qquad (7.6\text{-}18)$$

where:

φ is the creep coefficient corresponding to the load and time interval.

Shrinkage curvatures may be assessed by:

$$\frac{1}{r_{cs}} = \varepsilon_{cs} \cdot \alpha_e \cdot \frac{S}{I} \qquad (7.6\text{-}19)$$

where:

$1/r_{cs}$ is the curvature due to shrinkage;

ε_{cs} is the free shrinkage strain (see subclause 5.1.10.7.2);

S is the first moment of area of the reinforcement about the centroid of the section;

I second moment of area of the section;

α_e is the effective modular ratio = $E_s / E_{c,ef}$.

S and I should be calculated for the uncracked and the fully cracked condition. The final curvature is assessed by Eq. (7.6-14).

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The most rigorous method for assessing deflections is to compute the curvatures at a number of sections along the member and then calculate the deflection by numerical integration. In most cases it will be acceptable to compute the deflections twice, assuming the whole member to be in the uncracked condition and in the fully cracked condition, and then interpolate according to Eq. (7.6-14).

7.6.5.2.3 Simplified method for PC structures

Under the quasi-permanent load combination, the structure is considered to be uncracked.

The curvature at time t is defined as:

$$\frac{1}{r}(t) = \frac{\varepsilon_{c,bottom}(t) - \varepsilon_{c,top}(t)}{h} \tag{7.6-20}$$

where $\varepsilon_{c,bottom}(t)$ and $\varepsilon_{c,top}(t)$ are the strains at time t in the bottom and top fibres of the cross section and h is the height of the section.

The strains at time t are calculated in a way that takes the effect of self weight, the initial prestress, other quasi-permanent actions and the stress variation due to creep, shrinkage and relaxation into account according to the following expression:

$$\varepsilon_c(t) = \varepsilon_{cs}(t,t_0) + \sigma_c(t_0)\cdot\frac{1+\varphi(t,t_0)}{E_{cm}} + \frac{\sigma_c(t)-\sigma_c(t_0)}{E_{cm}}\cdot\left[1+\chi\cdot\varphi(t,t_0)\right] \tag{7.6-21}$$

where

$\sigma_c(t_0)$ is the concrete stress at the initial time t_0 due to prestress and load effects;

$\sigma_c(t)$ is the concrete stress at time t.

For long term loading the ageing coefficient χ can be assumed to be equal to 0.8.

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The approach given in subclause 7.6.5.2.3 is also followed in Eurocode 2.

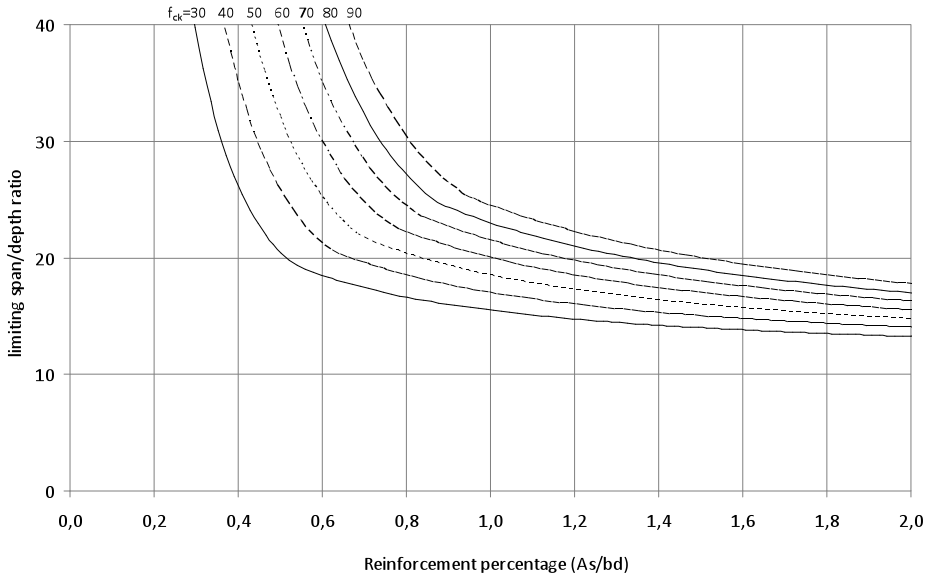


Figure 7.6-10: Graphical representation of Eqs. (7.6-22a) and (7.6-22b) for a simply supported slab bearing in one direction (K = 1.0) and σ_s = 250 N/mm^2

7.6.5.2.4 Cases where calculations may be omitted

Limits to the span/depth ratio may be formulated, which will be adequate for avoiding deflection problems in normal circumstances.

The limiting span to depth ratio l/d may be estimated using the expressions (7.6-22a) and (7.6.22b) and multiplied by correction factors to allow for the type of reinforcement used and other variables.

$$\frac{l}{d} = K\left[11 + 1{,}5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho} + 3{,}2\sqrt{f_{ck}}\left(\frac{\rho_0}{\rho} - 1\right)^{3/2}\right] \text{ if } \rho \le \rho_0 \qquad (7.6\text{-}22a)$$

$$\frac{l}{d} = K\left[11 + 1{,}5\sqrt{f_{ck}}\,\frac{\rho_0}{\rho - \rho'} + \frac{1}{12}\sqrt{f_{ck}}\sqrt{\frac{\rho'}{\rho_0}}\right] \text{ if } \rho > \rho_0 \qquad (7.6\text{-}22b)$$

where:

l/d is the limit span/depth ratio;

K is the factor depending on the structural system;

ρ_0 is the reference reinforcement ratio = $\sqrt{f_{ck}}\cdot 10^{-3}$;

ρ is the required tension reinforcement ratio at mid-span to resist the moment due to the design loads (at support for cantilevers);

ρ' is the required compression reinforcement ratio at mid-span to resist the moment due to de design loads (at support for cantilevers);

f_{ck} is the characteristic concrete compression strength in N/mm^2.

If other stress levels apply than σ_s = 250 N/mm^2, for which the Eqs. (7.6-22a) and (7.6-22b) have been derived, the values obtained by those equations should be multiplied by 250/σ_s. It will normally be conservative to assume that:

$$250/\sigma_s = 500/(f_{yk}A_{s,req}/A_{s,prov}) \qquad (7.6\text{-}23)$$

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The values given by Eqs. (7.6-22a) and (7.6-22b) and Table 7.6-6 have been derived from the results of a parametric study made for a series of beams and slabs simply supported with rectangular cross section, using the general approach given in subclause 7.6.5.2.2. Different values of the concrete strength class and a 500 MPa characteristic yield strength were considered. For a given area of tension reinforcement the ultimate moment was calculated and the quasi-permanent load was assumed as 50% of the corresponding total design load. The span to depth limits obtained satisfy the limiting deflection span/250 for quasi-permanent loads and span/500 for quasi-permanent loads after construction.

Eqs. (7.6-22a) and (7.6-22b) have been derived under the assumption that the steel stress under the appropriate design load at SLS at a cracked section at the mid span of a beam or slab or at the support of a cantilever, is 250 N/mm^2 (corresponding roughly to f_{yk} = 500 N/mm^2)

where:

σ_s is the tensile strain at mid-span (at support for cantilevers) under the design load at SLS;

$A_{s,prov}$ is the area of steel provided at this section;

$A_{s,req}$ is the area of steel required at this section for ultimate load.

For flanged sections where the ratio of the flange breadth to the rib breadth exceeds 3, the values of *l/d* given by Eq. (7.6-22) should be multiplied by 0.8.

For beams and slabs, other than flat slabs, with spans exceeding 7 m, which support partitions liable to be damaged by excessive deflections, the values of *l/d* given by Eq. (7.6-22) should be multiplied by 7/*l* (with *l* in metres).

For slabs where the greater span exceeds 8.5 m, and which support partitions liable to be damaged by excessive deflections, the values of *l/d* given by Eq. (7.6-22) should be multiplied by 8.5/*l* (with *l* in metres).

The values K follow from Table 7.6-6.

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Table 7.6-6: Basic ratios of span/effective depth for reinforced members without axial compression

Structural system	K	**Concrete highly stressed** $\rho = 1.5\%$	**Concrete lightly stressed** $\rho = 0.5\%$
Simply supported beams, one- or two-way spanning simply supported slab	1.0	14	20
End span of continuous beam or one-way continuous slab continuous over one long side	1.3	18	26
Interior span of beam or one-way or two way spanning slab	1.5	20	30
Slab supported on columns without beams (flat slabs) (based on longer span)	1.2	17	24
Cantilever	0.4	6	8

7.6.6 Vibrations

7.6.6.1 General

Vibrations of structures may affect the serviceability of a structure as follows:

- functional effects (discomfort to occupants, affecting operation of machines, etc.);
- structural effects (mostly on non-structural elements, as cracks in partition walls, loss of cladding, etc.).

Vibrations can be caused by several variable actions, e.g.

- rhythmic movements made by people, such as walking, running, jumping and dancing;
- machines;
- waves due to wind and water;
- rail and road traffic;
- construction work such as driving or placing by vibration of sheet;
- piles, compressing soil by means of vibrations as well as blasting work.

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Vibrations that endanger the structure, such as very large deflections due to resonance or the loss of resistance due to fatigue, should be included in the verification for ULS of the structure.

Table 7.6-7: Critical frequency in structures subject to vibrations caused by movements of people

Structures	**Frequency** (s^{-1}) f_{crit}
Gymnasia and sports halls	8.0
Dance rooms and concert halls without permanent seating	7.0
Concert halls with permanent seating	3.4
Critical work areas	1,0
Residence	1.4 – 4.0
Office	4.0
Workshop	8.0
Structures for pedestrians and cyclists	See below*

* Natural frequencies between 1.6 and 2.4 s^{-1} and between 3.5 and 4.5 s^{-1} are to be avoided in structures for pedestrians and cyclists. Joggers can also cause vibrations in structures with natural frequencies between 2.4 and 3.5 s^{-1}.

7.6.6.2 Vibrational behaviour

To secure satisfactory behaviour of a structure subject to vibrations, the natural frequency of vibration of the relevant structure should be kept sufficiently apart from critical values which depend on the function of the corresponding building, see Table 7.6-7. The natural frequency must be far enough from the critical frequency.

The vibrational behaviour of structures can be influenced by the following measures:

- changing the dynamic actions;
- changing the natural frequencies by changing the rigidity of the structure or the vibrating mass;
- increasing the damping features, etc.

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In current numerical models there are two types of crack models:

(1) Discrete crack model. In this model the crack is formed at the interface between two elements. After opening it changes the boundary conditions. The fracture properties are described by the interface constitutive law. This approach requires a re-meshing and mesh refinement near the crack tip during the crack propagation.

(2) Smeared crack model. According to this approach the crack is modeled by assuming orthotropic damage to the concrete within the area assigned to an element, or an integration point. The crack is modeled by a crack band. During the crack propagation the tensile strain localizes within the crack band. This is caused by the softening nature of the stress-crack opening law. The objectivity of the solution (low mesh sensitivity) is ensured by considering the crack band size as a regularization parameter of the strain localization.

The smeared crack model is more suitable for practical applications, where many cracks occur. The discrete crack model is suitable for cases with one distinct crack (Fig. 7.6-11.).

Basically, three parameters describe the constitutive law of crack opening: tensile strength, shape of the softening function and fracture energy.

7.6.7 Verification of serviceability limit state by numerical simulation

7.6.7.1 Fracture mechanics-based models

Analysis of the states of stress and deformation under serviceability conditions can be performed by non-linear finite element analyses based on fracture mechanics. This is a general method and is applicable to structures subjected to general states of stress. However, sufficiently fine numerical models are required in order to capture individual cracks.

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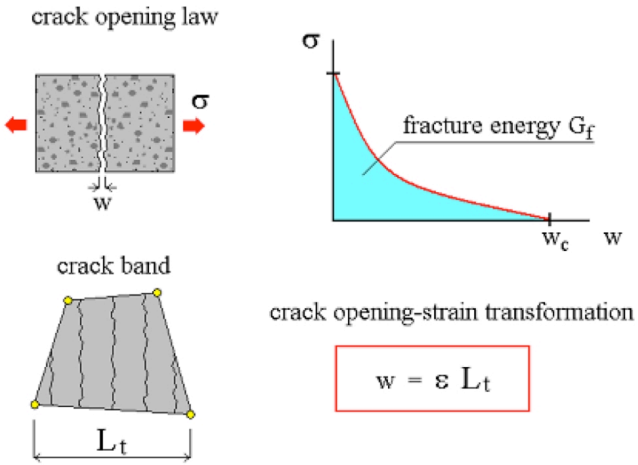


Figure 7.6-11: Constitutive law of crack opening for discrete and smeared crack models

In fracture mechanics based models the cracks are formed as a result of strain localization after reaching the concrete tensile strength. This process is successful only for a sufficiently small size of the finite element mesh.

As a rough estimate the element size should be smaller than half the crack spacing. Consequently, in reinforced concrete structures with small bar spacing, where small crack spacing occurs, the element size should be small, while in plain concrete structures, or locations without reinforcement, the elements can be larger. The upper limit for the acceptable element size is controlled by the fracture energy of the concrete.

7.6.7.2 Tension stiffening-based models

In cases outside of the range suitable for the fracture mechanics-based models mentioned in 7.6.7.1, the crack effect can be described by tension stiffening. In this approach no discrete cracks are considered and the properties of cracked concrete are regarded in an average sense by tension stiffening. This model offers adequate results for the analysis of deformations and strains required for the analysis of crack widths.

The expressions for tension stiffening, crack spacing and crack width, given in Eqs. (7.6-3) to (7.6-5) can be applied for the smeared crack model.

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The rules in this chapter have been derived predominantly for Steel Fibre Reinforced Concrete (FRC). They are based on the assumptions of subclause 5.6.1. Design recommendations especially focussing on high and ultra-high strength fibre concrete are in preparation (*fib* Task Group 8.6)

A distinction should be made between structures with linear elements (statically determinate and indeterminate beams or frames), where the stress redistribution is limited to few sections, where strain localization may occur, and structures with a higher degree of redundancy, where stress redistribution occurs in multiple cracks (like in slabs).

In linear structures there may be critical zones where a model based on a plane section can be assumed, by spreading the localized relative rotation in defined regions characterized by a prescribed length in which the curvature is assumed constant. In these zones the curvature jumps to larger values (post-peak values) with respect to the values reached in the adjacent regions (where the curvature corresponds the unloaded elastic values), although the bending moment is the same to respect the rotational equilibrium.

For thin walled structures, fibre orientation may be influenced by the reduced thickness (wall effect), which is significantly dependent on the casting direction. In these structures, material properties can be better determined by performing "structural tests".

In thin walled structures subjected to bending (shell structures), due to the reduced thickness, the ultimate strain should be significantly reduced in order to satisfy ductility requirements expressed in terms of ultimate strain.

The design rules are not intended to be used for slabs on grade, nor for temporary sprayed concrete linings, nor to design applications in which increased resistance to plastic shrinkage, increased resistance to abrasion, or impact are aimed for. The design rules refer to applications, covered in the scope of EC2.

Due to the nature of FRC, design supported by testing leads to optimized solutions in terms of structural performance versus cost.

7.7 Verification of safety and serviceability of FRC structures

7.7.1 Classification

FRC structures can be classified as:

- structures with linear elements (beams, and columns);
- walls;
- slabs;
- shells (e.g. thin walled members);
- three-dimensional members.

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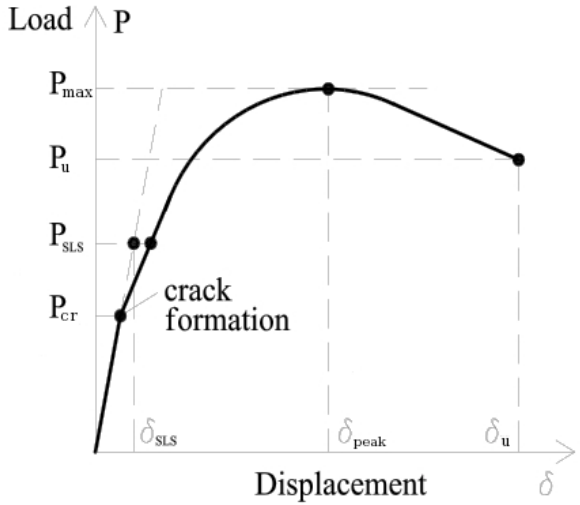


Figure 7.7-1: Typical load (P) – displacement (δ) curve for a FRC structure

In the case of linear elements, without conventional reinforcement, subjected to axial tension with small eccentricity (i.e. without compressive stresses in the section), in addition to the limitations provided by Eqs. (7.7-1) and (7.7-2), FRC should have a hardening behaviour in tension (see clause 5.6.1, Fig. 5.6-2).

In FRC structures, K_{Rd} is mainly affected by:

- the volume of the structure involved in the crack propagation process at failure (V), with respect to that used in the material identification procedure of the post-cracking residual strengths (V_0);
- the ratio between the maximum load P_{max} reached and the first cracking load P_{cr}., that quantifies the redistribution capability

$K_{Rd} = K_{Rd}(V/V_0, P_{max}/P_{cr})$.

7.7.2 Design principles

Structural design must satisfy requirements for resistance and serviceability for the expected service life of FRC elements. The ductility requirement in bending can be satisfied by minimum conventional reinforcement (see also clause 7.13).

In all FRC structures without the minimum conventional reinforcement, one of the following conditions shall be satisfied:

$$\delta_u \geq 20\ \delta_{SLS} \qquad (7.7\text{-}1)$$

$$\delta_{peak} \geq 5\ \delta_{SLS} \qquad (7.7\text{-}2)$$

where δ_u is the ultimate displacement, δ_{peak} is the displacement at the maximum load and δ_{SLS} is the displacement at maximum service load computed by performing a linear elastic analysis with the assumptions of uncracked concrete and initial elastic Young's modulus.

Usually, δ_u is related to the maximum deformation requirement of the structure.

The ultimate load P_u should always be higher than the load at crack initiation P_{cr} and higher than the maximum service load P_{SLS}.

When the structure is able to significantly redistribute the applied loads at failure, a factor K_{Rd}, that takes into account favourable effects due to redistribution, can be assumed:

$P_{Rd} = K_{Rd} \cdot P(f_{Fd})$

$P(f_{Fd})$ is the resistant load computed taking into account the design strength of FRC.

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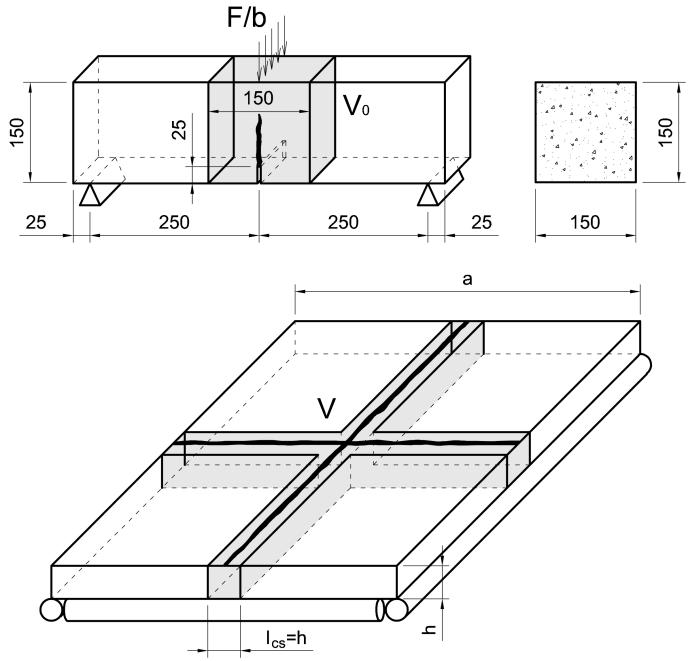


Figure 7.7-2: Volume involved in the failure for the classification test (V_0) and for a structure (V)

In the literature, different methods are proposed to evaluate such a coefficient that considers essentially the experimental evidence of a mechanical global response, which fits the response attainable with the average values of the residual post-cracking strength, when large load redistribution occurs. This is mainly due to the reduction of the standard deviation in the structure's response in relation to that measurable in a standard test, where a limited number of fibres and a specific geometrical location of the notch lead to larger scatter.

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K_{Rd} can be computed by a structural analysis that takes into account a random redistribution of the mechanical characteristics. When a statistical distribution of P_{max} is obtained starting from an assumed standard deviation of the mechanical constitutive law, the factor K_{Rd} can be computed as:

$$K_{Rd} = \frac{P_{\max,k}}{P_{\max,m}} \frac{f_{Ftum}}{f_{Ftuk}} \leq 1.4$$

where:

$P_{\max,k}$ is the characteristic value of the maximum load;

$P_{\max,m}$ is the mean value of the maximum load;

f_{Ftuk} is the characteristic value of the ultimate residual tensile strength of FRC, determined by considering w_u = min ($l_{cs}\varepsilon_{Fu}$, 2.5mm), according to Eq. (5.6-6), and assuming ε_{Fu} equal to 2% for variable strain distribution along the cross section and 1% for constant tensile strain distribution along the cross section;

f_{Ftum} is the mean value of the ultimate residual tensile strength of FRC.

7.7.3 Verification of safety (ULS)

7.7.3.1 Bending and/or axial compression in linear members

The bending failure stage is supposed to be reached when one of the following conditions applies (Figure 7.7-3): the meaning of the variables is found in subclauses 5.6.4 and 5.6.6):

- attainment of the ultimate compressive strain in the FRC, ε_{cu};
- attainment of the ultimate tensile strain in the steel (if present), ε_{su};
- attainment of the ultimate tensile strain in the FRC, ε_{Fu}.

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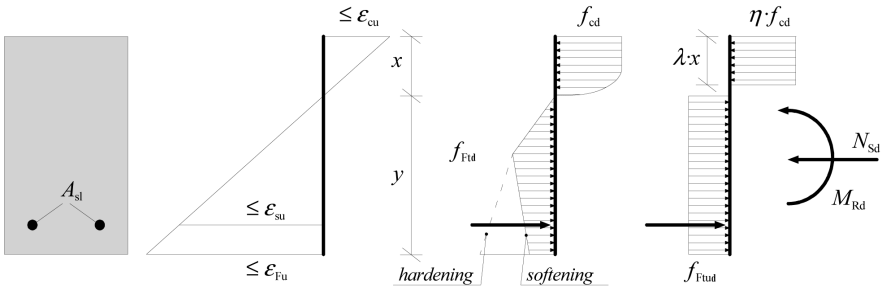


Figure 7.7-3: ULS for bending moment and axial force: use of the simplified stress/strain relationship (λ and η coefficient in accordance Eq. (7.2-15) to (7.2-18) in subclause 7.2.3.1.5)

7.7.3.2 Shear in beams

7.7.3.2.1 Beams without longitudinal and shear reinforcement

When FRC with tensile-hardening behaviour are used and members without both longitudinal and transverse reinforcement are considered, the principal tensile stress, σ_1, shall not be higher than the design tensile strength:

$$\sigma_1 \leq \frac{f_{\mathrm{Ftuk}}}{\gamma_{\mathrm{F}}} \tag{7.7-3}$$

where:

f_{Ftuk} is the characteristic value of the ultimate residual tensile strength of FRC determined with Eq. (5.6-6) for w_u = 1.5 mm [MPa];

γ_F value found in Table 5.6-1.

f_{Ftuk} is to be determined by an axial-tensile test

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This approach was recently developed and validated, and is now reliable, but is not consistent with the approach described in clause 7.3.3. Other approaches are available in the literature; in particular the following that is aligned to the approach presented in the clause 7.3.3 and is more extensively explained in *fib* Bulletin 57 "Shear and punching shear in RC and FRC elements. Workshop proceedings." (*fib*, 2010), although not yet fully validated.

Eq. (7.7-4) is valid for steel fibre reinforced concrete with conventional strength. It has not been validated for other fibre materials nor for non-conventional strength concretes, such as reactive powder concrete.

7.7.3.2.2 Beams without shear reinforcement

The design value for the shear resistance in members with conventional longitudinal reinforcement and without shear reinforcement is given by (in N):

$$V_{\mathrm{Rd,F}} = \left\{ \frac{0.18}{\gamma_{\mathrm{c}}} \cdot k \cdot \left[100 \cdot \rho_{\mathrm{l}} \cdot \left(1 + 7.5 \cdot \frac{f_{\mathrm{Ftuk}}}{f_{\mathrm{ctk}}} \right) \cdot f_{\mathrm{ck}} \right]^{1/3} + 0.15 \cdot \sigma_{\mathrm{cp}} \right\} \cdot b_{\mathrm{w}} \cdot d \qquad (7.7\text{-}4)$$

where:

γ_c is the partial safety factor for the concrete without fibres;

k is a factor that takes into account the size effect and is equal to:

$$1 + \sqrt{\frac{200}{d}} \leq 2.0$$

d is the effective depth of the cross section [mm];

ρ_{l} is the longitudinal reinforcement ratio defined as:

$$\rho_l = A_{sl} / b_w d$$

A_{sl} is the cross sectional area of the reinforcement which extends $\geq l_{bd}+d$ beyond the considered section [mm^2];

f_{Ftuk} is the characteristic value of the ultimate residual tensile strength for FRC, by considering w_{u}= 1.5 mm according to Eq. (5.6-6) [MPa];

f_{ctk} is the characteristic value of the tensile strength for the concrete without fibres [MPa];

f_{ck} is the characteristic value of cylindrical compressive strength [MPa];

$\sigma_{cp} = N_{Ed}/A_c < 0.2\, f_{\mathrm{cd}}$ [MPa] is the average stress acting on the concrete cross section A_c [mm^2] for an axial force N_{Ed} [N], due to loading or prestressing actions ($N_{Ed} > 0$ for compression);

b_w is the smallest width of the cross-section in the tensile area [mm].

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A recent model that follows the approach to shear described in subclause 7.3.3.4, computes the term V_{RdF} as follows:

$$V_{Rd,F} = \frac{1}{\gamma_F}(k_v\sqrt{f_{ck}} + k_f f_{Ftuk}\cot\theta)zb_w \qquad (7.7\text{-}6)$$

where:

f_{Ftuk} is the characteristic value of the ultimate tensile strength for FRC, as determined by direct tensile tests, corresponding to the crack width at ultimate, w_u;

$k_f = 0.8$;

and

$$k_v = \frac{0.4}{1+1500\varepsilon_x}\cdot\frac{1300}{1000+k_{dg}z} \quad \text{for } \rho_w < 0.08\sqrt{f_{ck}/f_{yk}}$$
$$k_v = \frac{0.4}{1+1500\varepsilon_x} \quad \text{for } \rho_w \geq 0.08\sqrt{f_{ck}/f_{yk}} \qquad (7.7\text{-}7)$$

In Eq. (7.7-7) ε_x is the longitudinal strain at the mid-depth of the effective shear depth as determined by either Eq. (7.3-14) or (7.3-16), as appropriate, z is the internal lever arm (in mm) between the flexural tensile and compressive forces (Figure 7.3-9) and k_{dg}, is an aggregate size influence parameter.

The aggregate size influence parameter in Eq. (7.7-7), k_{dg}, is given by:

$$k_{dg} = \frac{32}{16+d_g} \geq 0.75 \qquad (7.7\text{-}8)$$

where d_g is the maximum aggregate size in mm. If the size of the maximum aggregate particles is less than 16 mm, this parameter may be taken as $k_{dg} = 1.0$.

The shear resistance, $V_{Rd,F}$, is assumed to be not smaller than the minimum value, $V_{Rd,Fmin}$, defined as:

$$V_{Rd,Fmin} = \left(v_{min} + 0.15\cdot\sigma_{cp}\right)\cdot b_w \cdot d \qquad (7.7\text{-}5)$$

where $v_{min} = 0.035\cdot k^{3/2}\cdot f_{ck}^{1/2}$.

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The limits of the angle of the compressive stress field, θ, relative to the longitudinal axis of the member, as shown in Figure 7.3-11, are:

$$\theta_{\min} \le \theta \le 45^{o} \tag{7.7-9}$$

where the minimum strut inclination angle is:

$$\theta_{\min} = 29° + 7000\varepsilon_x \tag{7.7-10}$$

Where for the determination of f_{Ftuk}, the crack width at ultimate (w_u) is taken as:

$$w_u = 0.2 + 1000\varepsilon_x \ge 0.125\,\text{mm} \tag{7.7-11}$$

The minimum shear reinforcement should be provided by either stirrups (7.3-22), or fibres.

Eq. (7.7-14) is based on steel fibre concrete research, and should be checked for other types of material.

7.7.3.2.3 Beams with shear and longitudinal reinforcement

For the design of members with shear reinforcement the basic relation Eq. (7.3-23) applies, being:

$$V_{Rd} = V_{Rd,c} + V_{Rd,s} \tag{7.7-12}$$

For FRC elements this equation becomes:

$$V_{Rd} = V_{Rd,F} + V_{Rd,s} \tag{7.7-13}$$

where

$V_{Rd,s}$ is to be taken from Eq. (7.3-29);

$V_{Rd,F}$ follows from Eq. (7.7-4).

7.7.3.2.4 Minimum shear reinforcement

The minimum amount of conventional shear reinforcement (stirrups) is not required if the following condition is fulfilled:

$$f_{Ftuk} \ge 0.08\sqrt{f_{ck}} \tag{7.7-14}$$

where:

f_{Ftuk} is the characteristic value of the ultimate residual tensile strength for FRC, by considering w_u = 1.5 mm according to Eq. (5.6.-6) [MPa].

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This allows limiting the development and the diffusion of the inclined cracking and, as a consequence, can ensure sufficient member ductility.

When the above-mentioned limitation is not applied, conventional shear reinforcement (stirrups) shall be introduced according to Eq. (7.7-13).

7.7.3.3 Torsion in beams

7.7.3.3.1 Beams without longitudinal and transverse reinforcement

When FRC with a hardening tensile behaviour is used in a member without both longitudinal reinforcement and transverse reinforcement, the principal tensile stress shall not exceed the design tensile strength:

$$\sigma_1 \leq \frac{f_{\mathrm{Ftuk}}}{\gamma_{\mathrm{F}}} \tag{7.7-15}$$

where:

f_{Ftuk} is the characteristic value of the ultimate residual tensile strength for FRC at w_u = 1.5 mm according to Eq. (5.6.-6) [MPa];

γ_F is the partial safety factor for the FRC, which follows from Table 5.6-1.

7.7.3.3.2 Beams with longitudinal and transverse reinforcement

The presence of fibres increases the torsion capacity; however, design models are not currently available. Models should be proven by experiments on real size elements.

7.7.3.4 Walls

7.7.3.4.1 Walls without conventional reinforcement

For 2D elements loaded in their plane it is possible to check the limit state criteria for SLS and ULS by means of the biaxial domain of failure, where the uniaxial tension strengths are reduced to f_{Fts} or $f_{Ftu,}$ according to the limit state considered.

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For redundant structures, like slabs, the ultimate deformation determines the ultimate limit state condition. From that the corresponding crack opening (w_u) can be computed, which can be in the order of magnitude of the SLS crack opening (f_{R1})

7.7.3.4.2 Walls with conventional reinforcement

In FRC structures, the fibre contribution can be accounted for by nonlinear finite element analyses or strut and tie models with the constitutive laws defined in subclause 5.6.4

In FRC structure satisfying minimum requirements (Eqs. (5.6-2) and (5.6-3)), secondary conventional reinforcement can be omitted.

7.7.3.5 Slabs

7.7.3.5.1 Members without reinforcement

For slab elements without conventional reinforcement (Figure 7.7-4) with predominantly bending actions, the strength verification can be done with reference to the resisting moment, m_{Rd}, evaluated by considering a rigid plastic relationship (Figure 7.7-3c):

$$m_{\mathrm{Rd}} = \frac{f_{\mathrm{Ftud}} \cdot t^2}{2} \tag{7.7-16}$$

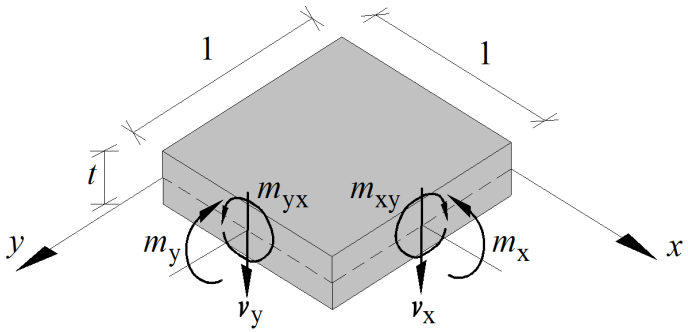


Figure 7.7-4: Actions in a slab element

When a linear analysis is performed, the maximum principal moment should be lower than m_{Rd}.

When a limit analysis is performed, m_{Rd} can be regarded as the reference value.

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For slabs with longitudinal reinforcement, the ultimate crack opening $w_u = \psi d / 6$ is suggested, where ψ is calculated from subclause 7.3.5.4. However this equation has not been validated for all thicknesses.

The design bending moment can be increased, as described in 7.7.2.

Shear in FRC slabs without conventional reinforcement or prestressing is not regarded as dominant unless significant load concentrations occur close to the support.

7.7.3.5.2 Members with reinforcement

The verification of FRC elements with conventional reinforcement can be done with a nonlinear analysis (e.g. limit analysis, nonlinear finite element analysis).

7.7.3.5.3 Punching

For FRC members, Eq. (7.3-60) can be replaced by:

$$V_{Rd} = V_{Rd,F} + V_{Rd,s} \tag{7.7-17}$$

where $V_{Rd,F}$ represents the fibre reinforced concrete contribution to shear.

The design shear resistance attributed to the fibres may be taken as

$$V_{Rd,F} = V_{Rd,c} + V_{Rd,f} \tag{7.7-18}$$

$$V_{Rd,f} = \frac{f_{Ftuk}}{\gamma_F} b_0 \; d_v \tag{7.7-19}$$

f_{Ftuk} is the characteristic value of the ultimate residual tensile strength for FRC, calculated taking into account w_u = 1.5 mm [MPa];

b_0 is the shear resisting control perimeter as defined in subclause 7.3.5.2(4) [mm];

d_v is the shear resisting effective depth defined in subclause 7.3.5.2(2) [mm].

When reinforcement is needed for punching resistance, a minimum amount of fibres (and if applicable transverse reinforcement) is required in order to ensure sufficient deformation capacity at failure as follows:

$$V_{Rd,s} + V_{Rd,f} \geq 0.5 \cdot V_{Ed} \tag{7.7-20}$$

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Eq. (7.7-22) is derived from Eq. (7.6-3) –(7.6-5). It follows from this equation through the modified value of $l_{s,max}$, the length over which slip between concrete and steel occurs, necessary to reintroduce the cracking-force $A_c(1+\alpha_e\rho_{s,ef})f_{ctm}$ back into the concrete by bond.

By virtue of the action of the fibres, which generate a residual tensile strength f_{Ftsm}, the force to be reintroduced by bond is reduced to $A_c(1+\alpha_e\rho_{s,ef})\cdot(f_{ctm}-f_{Ftsm})$.

7.7.3.5.4 Shear in slabs with longitudinal reinforcement

If shear reinforcement is to be provided according to subclause 7.3.3.3, Eq. (7.7-4) may be applied as an alternative.

7.7.4 Verification of serviceability (SLS)

7.7.4.1 Stress limitation

The compressive stresses at the SLS shall be limited in accordance with subclause 7.6.3.3.

Tensile stresses in the rebars shall be limited at the SLS in accordance with subclause 7.6.3.4.

In structural FRC elements having a tension softening behaviour after cracking, the tensile stress verification is not necessary if the element is verified at the ULS.

In structural FRC elements having a tension hardening behaviour after cracking, the tensile stress verification shall be done by imposing the limitation:

$$\sigma_t \le 0.6 f_{Ftsk} \qquad (7.7\text{-}21)$$

where σ_t is the principal tensile stress and f_{Ftsk} is the characteristic value of f_{Fts} given in Eq. (5.6-5).

7.7.4.2 Crack width in members with conventional reinforcement

The design crack width w_d in FRC elements can be calculated by:

$$w_d = 2\left\{k\cdot c+\frac{1}{4}\frac{\phi_s}{\rho_{s,ef}}\cdot\frac{\left(f_{ctm}-f_{Ftsm}\right)}{\tau_{bm}}\right\}\cdot\frac{1}{E_s}\cdot\left(\sigma_s-\beta\cdot\sigma_{sr}+\eta_r\cdot\varepsilon_{sh}\cdot E_s\right) \qquad (7.7\text{-}22)$$

where:

f_{Ftsm} follows from Eq. (5.6-5).

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So, the introduction length $l_{s,max}$ is reduced to:

$$l_{s,max} = k \cdot c + \frac{1}{4}\frac{(f_{ctm} - f_{Ftsm})}{\tau_{bm}} \cdot \frac{\phi_s}{\rho_{s,ef}}$$

For the other symbols reference is made to Eq. (7.6-5).

With $\sigma_{sr} = (f_{ctm} - f_{Ftsm}) \cdot (1 + \alpha_e \cdot \rho_s) / \rho_{s,eff}$ (7.7-23)

In σ_s the effect of the fibres (f_{Ftsm}) needs to be taken into account.

$f_{Ftsm} = f_{Ftsk} / 0.7$ (7.7-24)

where f_{Ftsk} is the characteristic value of f_{Fts} given in Eq. (5.6-5).

7.7.4.3 Minimum reinforcement for crack control

For controlling the crack width in the elements under bending, if needed, a minimum reinforcement should be applied, at least equal to:

$$A_{s,\min} = k_c \cdot k \cdot (f_{ctm} - f_{Ftsm}) \frac{A_{ct}}{\sigma_s} \quad (7.7\text{-}25)$$

where:

f_{ctm} is the average value of the tensile strength of the concrete matrix;

f_{Ftsm} is the average value of the residual strength of FRC;

A_{ct} is the tensile part of the concrete cross section, evaluated by considering a stress field at elastic limit;

σ_s is the maximum tensile stress in the reinforcement in the cracked state, that can be considered equal to the yielding stress of the steel;

k_c is a coefficient taking account of the stress distribution in the cross section just before cracking and the change of the inner lever arm; for rectangular cross sections $k_c = 1$;

k is a coefficient taking into account of non-uniform self-equilibrating stresses, leading to a reduction of the cracking force:
k =1.0 for webs with $h \leq 300$ mm or flanges with width ≤ 300 mm;
k = 0.65 for webs with $h \geq 800$ mm or flanges with width ≥ 800 mm;
for intermediate values interpolation can be applied.

When $A_{s,min}$ is negative, the minimum reinforcement can be due only to the fibre reinforcement.

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The forthcoming standard ISO 16204, "Durability – Service life design of concrete structures" is based on the principles given in *fib* MC SLD, see *fib* Bulletin 34, "Model Code for Service Life Design" (*fib*, 2006) and is also fully in line with the provisions given in clause 7.8 of this Model Code.

Subclause 3.2.2 gives the basis for verification of the design service life in terms of performance requirements (length and target reliability levels).

Traditionally, national and international concrete standards give requirements to achieve the desired design service life based on the "deemed-to-satisfy" and the "avoidance of deterioration" approach. Such operative requirements have to be calibrated by the responsible standardization body. This clause gives guidance for such calibration

In subclause 4.7.2 the durability related exposure conditions in the design situations are addressed. Reference is made to ISO 22965-1 "Concrete Part 1: Methods of specifying and guidance for the specifier", which gives an example of how to differentiate the environmental loads with respect to deterioration on the structure by 17 "Exposure classes". The same classification is adopted by the European CEN standards on design of concrete structures. Such classification is qualitative in nature and is by the local standardization body often linked directly to "deemed-to-satisfy" and "avoidance of deterioration" requirements in operational standards.

If more refined service life designs are to be undertaken by the use of deterioration modelling, this classification of the environmental load must be related to quantified parameters, for instance chloride concentrations for marine structures. When publishing this Model Code, such quantified parameters were not available in any operational standard. Information must therefore be found by measurements on existing structures and in the literature, for instance in *fib* MC SLD (*fib* Bulletin 34, *Model Code for*

7.8 Verification of limit states associated with durability

7.8.1 General

Verification of limit states associated with durability may be done according to one of the following safety formats given in chapter 4:

- probabilistic safety format;
- partial safety factor format;
- deemed-to-satisfy approach;
- avoidance-of-deterioration approach.

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Service Life Design (*fib*, 2006*)* and Concrete Society Technical Report no 61 (see Bamforth P. Concrete Society Technical report no 61: 2004 *Enhancing reinforced concrete durability*).

Another concept related to the verification of limit states associated with durability is described in "Guidance paper F" to the "Construction Products Directive" of the European Community. This is the use of the so-called "torture tests", where the material is subject to test conditions without doubt harsher than what will be the case in the actual exposure during the design service life. If the material stands the test, it is also accepted that the verification is fulfilled, however, with an unknown margin. However, it is not possible to conclude from a failure to withstand such an onerous "torture test" that the material would underperform during real in-field exposure. Many freeze-thaw tests for concrete are in this category. The "torture tests" is referred to in the Guidance Paper F "Durability and Construction Products Directive" of the European Community.

At the time of publishing this Model Code, the European standardization body CEN is working on a concept named "Equivalent Durability Concept – EDC". This implies that a material composition not dealt with in the operational standard may be compared to one reference with a proven long-term performance. The comparison is made on the basis of testing. Based on these test results the performance of the candidate material has to be assessed at the end of its design service life and then compared with that of the reference. Such an extrapolation of test results involves the use of modelling. The EDC assumes that this modelling is done by the responsible standardization body and that the user of this concept only applies a fixed aging factor for the relevant mix composition authorized by the standardization body.

Within clause 7.8 the following deterioration mechanism are addressed:

- carbonation-induced corrosion;
- chloride-induced corrosion;
- freeze-thaw attack.

For these mechanisms models with a relatively broad international acceptance exist.

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Other deterioration mechanisms as alkali silica reaction, acid and sulphate attack and delayed ettringite formation are not treated in this detail in this Model Code, mainly due to the fact that broadly accepted time-dependent models to be included here are still under discussion.

Requirements to ensure that the design service life is not jeopardized due to these mechanisms are normally based on long term field experience combined with available scientific insight in their nature. The provisions found in operational standards are, based on this, the results of the expert's opinion by the members of the standardization committees.

It is possible that several deterioration mechanisms may occur simultaneously, for instance carbonation with chloride penetration, or freeze-thaw with carbonation with or without chlorides. So there may be a combination of exposure categories (see subclause 4.7.2) leading to the use of the probabilistic safety or partial safety factor formats with deemed to satisfy design.

In the following subclauses, the new aims of the Model Code related to the durability of concrete structures are presented.

7.8.2 Carbonation induced corrosion – uncracked concrete

7.8.2.1 Probabilistic safety format

7.8.2.1.1 Limit state: depassivation

The following requirement shall be fulfilled:

$$P\{\} = P_{dep.} = P\{c - x_c(t_{SL}) < 0\} < P_0 \qquad (7.8\text{-}1)$$

where:

$P\{\}$ is the probability that depassivation occurs;

c is the concrete cover [mm];

$x_c(t_{SL})$ is the carbonation depth at time t_{SL} [mm];

t_{SL} is the design service life [years];

P_0 is the target failure probability.

General equation describing the limit state of depassivation is:

$$P\{\} = P_{dep} = P\{t_{SL} - t_{ini}\} < P_0 \qquad (7.8\text{-}2$$

where:

$P\{\}$ is the probability that depassivation occurs;

t_{SL} is the design service life [years];

t_{ini} is the initiation period [years];

P_0 is the target failure probability.

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This general equation describing the limit state of depassivation has been developed by RILEM, see "Durability design of concrete structures, Report of Rilem Technical Committee 130-CSL", edited by A. Sarja and E. Vesikary.

When publishing this Model Code, no models with broad international consensus were available for predicting the length of the corrosion period till cracking, spalling or collapse of the structure occurs. For this reason service life designs are normally done with the limit state of depassivation (reaching a reduced pH of 8 - 9 at the rebar surface).

The direct consequence of passing this limit state is only that possible future protective measures for repair become more expensive. This rather conservative limit state is therefore normally linked to a corresponding relaxed target reliability level for failing, often in the order of 10^{-1} to 10^{-2}. See also subclause 3.3.3.

To support corrosion of the reinforcement a certain level of humidity is needed. For structural elements solely exposed to a relative dry indoor environment, a limit state 'depassivation' may not be relevant as no significant corrosion will develop.

Subclause 7.8.2.1.2 and Eq (7.8-3) advice the designer to base the design on extrapolated field experience from similar structures to reduce the considerable uncertainty in model and data.

fib Bulletin 34, *Model Code for Service Life Design* (*fib*, 2006*)* gives further information on the use of the model given as Eq. (7.8-3).

The variables c, and $x_c(t_{SL})$, need to be quantified in a probabilistic approach.

7.8.2.1.2 Design model

The ingress of the carbonation front may be assumed to obey the following equation:

$$x_c\left(t\right) = W\left(t\right) \cdot k \cdot \sqrt{t} \qquad (7.8\text{-}3)$$

where

k is a factor reflecting aspects like the execution, basic resistance of the chosen concrete mix (like w/binder-ratio, cement type, additions) under reference conditions and the influence of the basic environmental conditions (like mean relative humidity and CO_2-concentration) against ingress of carbonation;

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The weather function W(t) may be assumed to obey the function:

$$W(t) = \left(\frac{t_o}{t} \right)^w \quad (7.8\text{-}4)$$

where:

t_o is time of reference [years];

t is the considered exposure time [years];

w is weather exponent [-] (w = 0 for indoor conditions and increasing to values of 1 > w > 0 with increasing frequency of wetting events):

$$w = (P_{SR}\ T_O W)^{b_w} \quad (7.8\text{-}5)$$

where:

$T_O W$ is time of wetness [-];

P_{SR} is probability of driving rain [-];

b_W is exponent of regression [-].

fib Bulletin 34, *Model Code for Service Life Design* annex B (*fib*, 2006) gives further information on how these conditions influence the factors governing $W(t)$ and k.

Both the uncertainty of the data and in the model shall be taken into account in the design.

When deriving the product of $W(t)$ and k from existing structures, the influence of these uncertainties will decrease considerably the older the structure is.

Subclause 5.1.13 and Eq. (5.1-141) give guidance in the case the predictions have to be based on accelerated testing on young specimens.

An overview of other models in use, and a database with supporting parameters, is given in Bamforth P. Concrete Society Technical report no 61: 2004 *Enhancing reinforced concrete durability.*

$W(t)$ is a weather function taking the meso-climatic conditions due to wetting events of the concrete surface into account.

For the design of a new structure, the factors W(t) and k, or their product, may be derived from literature data or existing structures where the concrete composition, execution and exposure conditions have been similar to those expected for the new structure.

When assessing the remaining service life of an existing structure, the product of $W(t)$ and k may be derived directly from measurements on the structure.

Alternatively to Eq. (7.8-3), the design may be based on the design model given in subclause 5.1.13.2, Eq. (5.1-141).

Other models may be used, provided that the basic principles formulated in clause 4.4 are fulfilled.

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Reinforcement corrosion leading to cracking, spalling and collapse depend to a high extent on the environment at the concrete surface. The micro environment may vary considerably along the concrete surface of structural elements. Most unfavourable micro environmental conditions are frequent wetting and drying and/or accumulation of aggressive agents (for instance chlorides originating from seawater or de-icing salts). Macro-cell corrosion effects may trigger high corrosion rates in areas with less severe micro environmental condition. For given degrees of corrosion the risk for cracking and spalling depends on the geometry of the cross section. Most vulnerable cross sectional areas, e.g. the edges of beams, should be chosen as decisive for design.

First approaches exist to quantify the variables $\Delta r_{(S),}$ (t_{SL}) and $\Delta r_{(R)}$. Most of the corresponding models are empirically derived, often based on very limited, in consequence insufficient data basis. The correlation between corrosion rates/concrete quality/micro environment is not yet quantified in detail. The same applies to the limit states spalling and collapse. To get first impressions on the propagation period, *fib* TG 5.6, when preparing *fib* MC

7.8.2.1.3 Limit states: corrosion-induced cracking, spalling and collapse

Exemplified with regard to cracking, the following basic limit state function shall be fulfilled:

$$P\{\} = P_{crack} = P\{\Delta r_{(R)} - \Delta r_{(S)}(t_{SL}) < 0\} < P_0 \qquad (7.8\text{-}6)$$

where:

$P\{\}$ is probability that carbonation-induced cracking occurs;

$\Delta r_{(R)}$ is maximal corrosion induced increase of the rebar radius which can be accommodated by the concrete without formation of cracks at the concrete surface [µm];

$\Delta r_{(S)}(t_{SL})$ is increase of the rebar radius due to reinforcement corrosion [µm];

t_{SL} is design service life [years];

P_0 is target failure probability.

An alternative design approach is:

$$P\{\} = P_{crack} = P\{t_{SL} - t_{ini} - t_{prop} > 0\} < P_0 \qquad (7.8\text{-}7)$$

where:

$P\{\}$ is probability that carbonation-induced cracking occurs;

t_{SL} is design service life [years];

t_{ini} is initiation period [years];

t_{prop} is propagation period [years];

P_0 is target failure probability.

The variables $\Delta r_{(R)}$ and $\Delta r_{(S)}(t_{SL})$ or the variables t_{ini} and t_{prop} need to be quantified in a probabilistic approach.

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SLD, organized a Delphic oracle. One result of the exposure dependent output of this Delphic oracle is given in *fib* Bulletin 34, *Model Code for Service Life Design*, Annex R (*fib*, 2006*)*. Together with existing models describing the initiation period and the herewith overall quantified propagation period, probabilistic calculations with regard to corrosion induced cracking, spalling and collapse of concrete structures may be performed, see Eq. (7.8-6) and Eq. (7.8-7)

The nominal value for the concrete cover is the dimension given to the constructor in the project specification (i.e. on drawings) and is assumed to represent the mean value of the cover depth.

The safety margin, Δc is to ensure that the great majority (in operational standards often assumed as the 95% fractile) of the cover thickness for the reinforcement bars are larger than the minimum cover used as a basis for the service life design.

ISO 22966 “Execution of concrete structures” assumes Δc = 10 mm if no other values are given in the execution specification.

Other methods may be used, provided that the basic principles formulated in clause 4.4 are fulfilled.

At the time of publishing this Model Code, no time-dependent model with general international consensus was available for this deterioration process. The time span from initiation to cracking may be estimated from existing structures where the concrete composition, execution and exposure conditions have been similar to those expected for the structure considered.

7.8.2.2 Partial safety factor format

7.8.2.2.1 Limit state: depassivation

The following limit state function shall be fulfilled:

$$c_d - x_{c,d}(t_{SL}) \geq 0 \qquad (7.8\text{-}8)$$

where:

c_d is design value of the concrete cover [mm];

$x_{c,d}(t_{SL})$ is design value of the carbonation depth at time t_{SL} [mm].

The design value of the concrete cover c_d is calculated as follows:

$$c_d = c_{nom} - \Delta c \qquad (7.8\text{-}9)$$

where:

c_{nom} is nominal value for the concrete cover [mm];

Δc is safety margin of the concrete cover [mm].

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To exemplify the design procedure and the quantification of the given quantities, an applicable design method is given in Annex C of *fib* Bulletin 34, *Model Code for Service Life Design* (*fib*, 2006*)*.

For given design service lives, basic requirements with regard to minimum cover to the reinforcement, limiting values for concrete composition, e.g. maximum w/binder ratio, crack width limitation and minimum level of workmanship are given in most operational concrete codes. These sets of requirements should be calibrated according to chapter 4.

An example of such a calibration is given in Maage M., Smeplass S., "*Carbonation – A probabilistic approach to derive provisions for EN 206-1*" DuraNet, Third workshop, Tromsø, Norway, June 2001. Reported in "Betongkonstruksjoners Livsløp" report no 19, Norwegian Road Administration.

The design value of the carbonation depth at time t_{SL}, $x_{c,d}(t_{SL})$ is calculated as follows:

$$x_{c,d}(t_{SL}) = x_{c,c}(t_{SL}) \cdot \gamma_f \qquad (7.8\text{-}10)$$

where:

$x_{c,c}(t_{SL})$ is characteristic value of the carbonation depth at time t_{SL} [mm], e.g. mean value of the carbonation depth;

γ_f is partial safety factor of the carbonation depth [-].

Other methods may be used, provided that the basic principles formulated in clause 4.5 are fulfilled.

7.8.2.3 Deemed-to-satisfy design

Within this approach a trading-off of geometrical (concrete cover to reinforcement), material parameters (indirectly linked to diffusion and binding characteristics) and execution aspects (compaction and curing) is applied.

7.8.2.4 Avoidance-of-deterioration design

Generally, avoidance is achieved if depassivation cannot take place due to infinite resistance of the concrete to carbonation or zero environmental load or infinite corrosion resistance of the reinforcement.

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As with carbonation, there were no available models with broad international consensus available for predicting the length of the corrosion period till cracking, spalling or collapse of the structure occurs when this Model Code was published. For this reason service life designs are normally based on the limit state of depassivation (reaching a critical chloride concentration at the rebar surface). As with carbonation, this rather conservative limit state is then normally linked with a corresponding relaxed target reliability level for failing, often in the order of 10^{-1} to 10^{-2}. See also subclause 3.3.3.

For permanently submerged members, lack of oxygen may hinder the corrosion process. For submerged members in electrical contact with members above sea level, macro-cell corrosion may however occur.

fib Bulletin 34, *Model Code for Service Life Design* (*fib*, 2006*)* gives further information on the use of the model given as Eq. (7.8-12) and the influence of the various conditions discussed above.

Both the uncertainty of the data and in the model shall be taken into account in the design. Suggestions on how to handle in some situations by excluding this convection zone (concrete layer, up to which the process of chloride penetration differs from Fick's second law of diffusion) from the calculations are given in *fib* Bulletin 34.

7.8.3 Chloride induced corrosion – uncracked concrete

7.8.3.1 Probabilistic safety format

7.8.3.1.1 Limit state: depassivation

The following limit state function shall be fulfilled:

$$P\{\} = P_{dep} = P\{C_{Crit} - C\,(c,t_{SL}) < 0\} < P_0 \qquad (7.8\text{-}11)$$

where:

$P\{\}$ is the probability that depassivation occurs;

C_{Crit} is the critical chloride content to achieve depassivation of the reinforcement[wt.-%/binder content];

$C(c,t_{SL})$ is the chloride content at depth c and time t [wt.-%/binder content];

c is the concrete cover [mm];

t_{SL} is the design service life [years];

P_0 is the target failure probability.

The variables c, C_{crit} and $C(c,t_{SL})$ shall be quantified in a probabilistic approach.

7.8.3.1.2 Design model

The ingress of chlorides in a marine environment may be assumed to obey the modified Fick's second law of diffusion:

$$C(x,t) = C_s - (C_s - C_i)\cdot\left[\operatorname{erf}\frac{x}{2\cdot\sqrt{D_{app}(t)\cdot t}}\right] \qquad (7.8\text{-}12)$$

where:

$C(x,t)$ is content of chlorides in the concrete at a depth x (structure surface: x = 0 m) and at time t [wt.-%/binder content];

C_s is the chloride concentration at the concrete surface [wt.-%/binder content];

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When deriving the factor D_{app} (t_0) from existing structures, the influence of these uncertainties will increase considerably the younger the structure is.

This is also the case when the reference diffusion coefficient is derived from a short-term laboratory test on young specimens. It will then not reflect execution or ageing effects and if used for design often result in an uneconomical over-design.

The decrease of $D_{app}(t)$ is due to several reasons:

- continued reaction of the binder, in particular when pozzolans and slag are used;
- influence capillary suction of water in the surface zone (deceasing with time);
- degree of saturation of concrete;
- in this respect the beneficial effect of penetrated chlorides from the sea-water or de-icing salt leading to ion exchange with subsequent pore-blocking in the surface layer.

The uncertainty in determining the aging-factor α increases if the span in time, represented by the observations, is shorter.

The uncertainty in using an aging factor α for design for service life periods far beyond that represented by the observations used to derive α, should also be considered.

C_i is the initial chloride content of the concrete [wt.-%/binder content];

x is the depth with a corresponding content of chlorides C(x,t) [mm];

$D_{app}(t)$ is the apparent coefficient of chloride diffusion through concrete [m²/sec.] at time t (see also subclause 5.1.13.3);

t is time [years] of exposure;

erf is error function.

$$D_{app}(t) = D_{app}(t_0)\left(\frac{t_0}{t}\right)^{\alpha} \qquad (7.8\text{-}13)$$

where:

D_{app} (t_0) is the apparent diffusion coefficient measured at a reference time t_0;

α is the aging-factor giving the decrease over time of the apparent diffusion coefficient. Depending on the type of binder and the micro environmental conditions the aging factor is likely to be between 0.2 and 0.8.

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In the given model, the surface chloride concentration, C_s, for practical reasons is assumed constant over the time of exposure. This is not correct as a build-up period is obviously needed. It is however often assumed that this build-up period is relatively short compared with the normal length of a design service life and that the influence of this simplification is limited on the design result. More sophisticated models where the build-up period of C_s is modelled are also used sometimes.

A realistic C_s for new designs should be based on in-field observations on existing structures with similar material composition and exposure. For assessing remaining service life, C_s may be measured directly on the structure in question.

The critical chloride content, C_{crit}, when the depassivation of the reinforcement occurs, is also a variable depending on factors like steel properties, type of cement and additions, moisture content etc. For practical design, a realistic distribution has to be applied. The fundamental knowledge on this property is still limited, and most numbers reported in the literature are based on in-field observations. Often these observations are grouped in tables with qualitatively described ranges as “negligible risk of corrosion”, “possible risk of corrosion” etc. When performing a probabilistic service life design, it is imperative to make clear if the actual limit state is linked to occurrence of corrosion (based on the mean and the statistical spread of C_{crit}), or to a conservative deterministic value of for example “negligible risk” or “possible risk”.

For structures subject to cyclic exposure of chlorides, in particular where de-icing salts are used only during the winter season, the chloride ions in the surface layer are not only transported by diffusion. Other effects like capillary suction, wash-out etc. will also play a role. The depth of this influenced surface layer will be reduced the more impermeable the concrete is and the shorter the cycles are. Suggestions for how to handle such situations are given in *fib* Bulletin 34, *Model Code for Service Life Design* (*fib*, 2006*)* and in the general literature.

An overview including other models, and a database for supporting parameters, are given in Bamforth P., Concrete Society Technical report no. 61: 2004 *Enhancing reinforced concrete durability.*

For the design of a new structure, the factors C_s, C_i, α and D_{app} (t_0) may be derived from existing structures where the concrete composition, execution and exposure conditions have been similar to those actual for the new structure.

When assessing the remaining service life of an existing structure, the factors, with the possible exception of α, may be derived directly from measurements on the structure.

For both design of new structures, and for the assessment of remaining service life of existing structures, the aging-factor, α, shall be obtained from in-field observations from structures where the concrete composition, execution and exposure conditions have been similar to those for the actual structure. At least observations at two periods of exposure (with a sufficient interval between the observations) are needed for the calculation of the ageing factor.

Other models may also be used, provided that the basic principles formulated in clause 4.4 are fulfilled.

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Subclause 7.8.2.1.3 applies, with "chloride penetration" substituted for "carbonation".

Subclause 7.8.2.2 applies, with "chloride penetration" substituted for "carbonation".

Subclause 7.8.2.3 applies, with "chloride penetration" substituted for "carbonation".

Subclause 7.8.2.4 applies, with "chloride penetration" substituted for "carbonation".

The corrosion rates in the region of cracks crossing the reinforcement are extremely dependent on the micro climatic conditions at the concrete surface and the orientation of the concrete surface. Most severe conditions occur in case of horizontal concrete surfaces and both cracks and chloride ingress from the top. For usual design service lives of more than 10 years and frequent chloride attack (e.g. parking decks in regions where de-icing salts are used) special protective measures are necessary to avoid the rapid penetration of chlorides to the reinforcement (e.g. linings or crack-bridging coatings). In the case of vertical surfaces and soffits with chloride spray or chloride containing water not leaking through cracks a high quality of concrete cover (cover thickness $\geq$ 50 mm, low permeability concrete, w/c $\leq$ 0.5) and ordinary crack width limitation ($w_{k,cal} \leq 0.3$ mm) ensures sufficiently long service life ($\geq$ 50 years) without extra protection.

In case of carbonation induced corrosion an adequate quality of concrete cover and ordinary crack width limitation ensures sufficiently long service life ($\geq$ 50 years) without extra protection.

7.8.3.1.3 Limit states: corrosion-induced cracking, spalling and collapse

See subclause 7.8.2.1.3.

7.8.3.2 Partial safety factor format

See subclause 7.8.2.2.

7.8.3.3 Deemed-to-satisfy design

See subclause 7.8.2.3.

7.8.3.4 Avoidance-of-deterioration design

See subclause 7.8.2.4.

7.8.4 Influence of cracks upon reinforcement corrosion

The minimum structural reliability of a cracked reinforced concrete structure shall be of comparable magnitude to the minimum reliability of a comparable exposed uncracked structure.

A simplified approach is used by most operational standards. This implies that corrosion of the reinforcement is not influenced by crack widths under a certain characteristic value. Depending on the severity of the environment and sensitivity of the structure, this limiting crack width is normally given as a characteristic value (5% upper fractile) in the range of 0.2 to 0.4 mm

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fib Bulletin 33 "*Durability of post-tensioning tendons*" (*fib*, 2006*)* describes multi-barrier systems for the protection of pre-stressing systems. The bulletin describes three different "Protection levels". To ensure the integrity of the component, more barriers are recommended, the harsher the exposure conditions for the structure. These systems are supposed to satisfy the design criteria with ample margin and may be classified according to chapter 4 as "deemed-to-satisfy" requirements based on the "avoidance-of-deterioration" approach.

Similar to the procedure given in subclauses 7.8.2 and 7.8.3, unwanted events with regard to serviceability/ functionality shall be identified (SLS). In addition, it shall be checked whether ultimate limits are affected by continuously corroding reinforcement within the cracked zone or not.

In harsh exposure conditions (for example exposure classes XD3/XS3), if functionality or structural integrity is affected, and if inspection and possible intervention cannot be performed, an avoidance-of-deterioration approach is recommended/needed.

7.8.5 Risk of depassivation with respect to pre-stressed steel

Relevant application rules given in subclauses 7.8.2, 7.8.3 and 7.8.4 to avoid depassivation of pre-stressed steel on an ULS reliability level shall be applied.

Since corrosion of prestressed steel can result in a sudden collapse of the structure, and without pervious warning by cracking and spalling associated with ordinary reinforcement, a higher level of reliability for not passing the depassivation limit state is normally needed than for ordinary reinforcement.

7.8.6 Freeze/thaw attack

7.8.6.1 Probabilistic safety format

7.8.6.1.1 Limit state: freeze/thaw damage causing local loss of mechanical properties, cracking, scaling and loss in cross-section – without deicing agents or sea-water

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To exemplify the design procedure and the quantification of the parameters, an applicable design method is given in Annex B of *fib* Bulletin 34, *Model Code for Service Life Design* (*fib*, 2006).

Equation (7.8-14) is based on the assumption that scaling occurs in the same moment as the concrete surface temperature falls below a certain, critical level, the scaling resistance T_R.

It is assumed that this critical level of scaling resistance changes with age, depending on exposure and type of concrete.

The following limit state function shall be fulfilled:

$$P\{\} = P_{\text{freeze/thaw damage}} = P\{T(t<t_{SL}),\ S_{CR} - S_{ACT}(t < t_{SL}) < 0\} < P_0 \qquad (7.8\text{-}14)$$

where:

$P\{\}$ is the probability that freeze/thaw damage occurs;

S_{CR} is the critical degree of saturation [-];

$S_{ACT}(t)$ is the actual degree of saturation at time t [-];

$T(t)$ concrete temperature in [K];

t_{SL} is the design service life [years];

P_0 is the target failure probability.

The variables S_{CR} and $S_{ACT}(t)$ need to be quantified in a probabilistic approach.

Other methods may be used, provided that the basic principles formulated in clause 4.4 are fulfilled.

7.8.6.1.2 Limit state equation for the salt-freeze/thaw induced surface scaling – with de-icing agents or sea-water

The following limit state function shall be fulfilled:

$$P\{\} = P_{\text{scaling}} = P\{T(t \le t_{SL}) - T_R(RH,\ n_{\text{cycles}},\ t,\ Cl^-,\ldots) < 0\} < P_0 \qquad (7.8\text{-}15)$$

where:

$P\{\}$ is the probability that scaling occurs;

$T(t)$ is the concrete temperature in [K];

$T_R(t,\ldots)$ is critical freezing temperature for scaling to occur at time t;

t_{SL} is the design service life [years];

Cl^- the chloride content [wt. -%/binder content];

RH relative humidity [%];

n_{cycles} number of freeze-thaw cycles [-];

P_0 is the target failure probability.

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To exemplify the design procedure and the quantification of the parameters, an applicable design method is given in Annex B of *fib* Bulletin 34, Model Code for Service Life Design (*fib*, 2006*)*.

Instead of partial factors multiplying the actions or dividing the resistance, for the partial safety factor format of the freeze/thaw attack the same effect is obtained by summing or subtracting a value.

The variables T and T_R shall to be quantified in a probabilistic approach.

Other methods may be used, provided that the basic principles formulated in clause 4.4 are fulfilled.

7.8.6.1.3 Limit states: freeze/thaw-induced deflection and collapse

With regard to load-carrying capacity and deformations, the design shall include the effect of localized changes in mechanical properties due to freeze/thaw damage.

7.8.6.2 Partial safety factor format

The following limit state function needs to be fulfilled:

$$S_{CR,d} - S_{ACT,d}(t < t_{SL}) \geq 0 \qquad (7.8\text{-}16)$$

where:

$S_{CR,d}$ is design value of the critical degree of saturation [-];

$S_{ACT,d}(t<t_{SL})$ is design value of the actual degree of saturation at time t [-];

t_{SL} is design service life [years].

The design value of the critical degree of saturation shall be calculated as follows:

$$S_{CR,d} = S_{CR,min} - \Delta S_{CR} \qquad (7.8\text{-}17)$$

where:

$S_{CR,\,min}$ is characteristic value of the critical degree of saturation (minimum value) [-];

ΔS_{CR} is margin of the critical degree of saturation [-].

The design value of the actual degree of saturation at time t, $S_{ACT,d}(t)$ shall be calculated as follows:

$$S_{ACT,d}(t) = S_{ACT}(t) + \Delta S_{ACT} \qquad (7.8\text{-}18)$$

where:

$S_{ACT,d}$ is characteristic value of the actual degree of saturation at time t [-];

ΔS_{ACT} is margin of the actual degree of saturation (load) [-].

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For structures exposed to freezing and with the potential for a higher level of moisture than the critical degree of saturation, a design according to the deemed-to-satisfy method is the normal procedure.

As an example, the International standard ISO 22965-1 "*Concrete Part 1: Methods of specifying and guidance to the specifier*" suggests the following set of requirements on the concrete composition for such structures:

- cement type and maximum w/c-ratio;
- minimum strength and/or minimum cement content;
- freeze-thaw-resistant aggregates;
- air entrainment or, if allowed by national provisions, adequate performance demonstrated by freeze-thaw testing (scaling).

Similar sets of requirements, based on long-term field experience, are found in most national, regional and international standards.

Due to the complexity in performing design according to the probabilistic method, the calibration of these deemed-to-satisfy requirements is normally based on an avoidance-of-deterioration approach.

This assumes that the freeze-thaw resistance is not reduced over time for a concrete fulfilling the above listed criteria.

Due to absence of freezing, and/or moisture levels below S_{CR}, any detrimental reaction due to freezing will be avoided.

For model for the corrosion depth *d* caused by acid attack reference is made to subclause 5.1.13.6, Eq. (5.1-146).

Other methods may be used, provided that the basic principles formulated in clause 4.5 are fulfilled.

7.8.6.3 Deemed-to-satisfy approach

Within this approach a trading-off of available space for expansion (air entrainment), material (non-freezeable water characteristics) and ageing (carbonation) variables shall be established.

7.8.6.4 Avoidance-of-deterioration method

Generally, avoidance is achieved if freeze/thaw deterioration cannot take place due to infinite material resistance or zero environmental load.

7.8.7 Chemical attack

7.8.7.1 Acid attack

7.8.7.1.1 Probabilistic safety format

At the time of publishing this Model Code, models to be included here are still under discussion. A probabilistic approach for SLD is therefore not feasible.

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The use of linings and protective coatings may avoid contact with acids from the external environment.

7.8.7.1.2 Partial safety factor format

At the time of publishing this Model Code, models to be included here are still under discussion. A partial safety factor approach for SLD is therefore not feasible.

7.8.7.1.3 Deemed-to-satisfy approach

Within this approach one or more of the following remedies are normally required based on calibration to long term experience:

- limitations to the porosity of the concrete (traditionally expressed by max. w/binder ratio, min. cement content and moisture curing);
- composite cements or the use of additions like silica fume, fly-ash and slag;
- type of aggregates.

7.8.7.1.4 Avoidance-of-deterioration approach

Solutions with pH > about 6.5 are considered as not detrimental to the cement paste based on Portland clinker.

In the case of severe acid attack linings or durable coatings should be provided to avoid direct contact of the acid with the concrete surface.

7.8.7.2 Sulphate attack

When hardened cement paste is attacked by sulphates of various bases, two deterioration mechanisms may occur: ettringite formation and thaumasite formation.

Ettringite (sulphoaluminate from C_3A) formation of calcium causes expansion and may result in disruption of the hardened paste.

In presence of Calcium Carbonate and low temperatures (5 – 8ºC) thaumasite may be formed causing disruption of the hardened cement paste without expansion.

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In some national standards for "Sulphate Resistant" cements a requirement on maximum content of ($C_4AF + 2(C_3A)$) is given in addition to the C_3A requirement.

7.8.7.2.1 Probabilistic safety format

At the time of publishing this Model Code, no time-dependent model with general international consensus was available for this deterioration process. A probabilistic approach for SLD is therefore not feasible.

7.8.7.2.2 Partial safety factor format

At the time of publishing this Model Code, no time-dependent model with general international consensus was available for this deterioration process. A partial safety factor approach for SLD is therefore not feasible.

7.8.7.2.3 Deemed-to-satisfy approach

Within this approach one or more of the following remedies are normally required based on calibration to long term experience:

- maximum amount of C_3A in the cement. Cements with C_3A content less than 3-5% and low C_3S content, and blast furnace cements with more than 60% slag are often regarded as “Sulphate Resistant”;
- composite cements or the use of use of additions like silica fume, fly-ash and slag;
- limitations to the porosity of the concrete (traditionally expressed by max. w/binder ratio, min. cement content and moisture curing);
- avoidance of limestone aggregates or fillers in case of the risk of thaumasite formation.

7.8.7.2.4 Avoidance-of-deterioration approach

To support a sulphate attack, sulphate ions and C_3A need to be present in the concrete member. If one or both of these conditions are below certain concentrations, avoidance of deleterious reactions will be achieved. In case of thaumasite formation even low amounts of calcium carbonate may be sufficient to allow for slow deterioration of the hardened cement paste.

In case of severe sulphate attack linings or durable coatings should be provided to avoid direct contact of the acid with the concrete surface.

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7.8.8 Alkali-aggregate reactions

Alkali-aggregate reactions involve deleterious chemical reactions between the alkalis in the paste and silica or carbonate constituents in the aggregate.

7.8.8.1 Probabilistic safety format

At the time of publishing this Model Code, models to be included here are still under discussion. A probabilistic approach for SLD is therefore not feasible.

7.8.8.2 Partial safety factor format

At the time of publishing this Model Code, models to be included here were still under discussion. A partial safety factor approach for SLD is therefore not feasible.

7.8.8.3 Deemed-to-satisfy approach

Within this approach one, or a combination of more, of the following remedies are normally required based on calibration to long term experience:

- maximum amount of alkalis originating from cement and other sources;
- composite cements or the use of additions like silica fume, fly-ash and slag;
- use of lithium nitrite/hydroxide and other chemical additions;
- limitations to the alkali-reactivity of the aggregate;
- accelerated testing of concrete samples (e.g. “torture tests”, see commentary to subclause 7.8.1);
- keeping the humidity in the structure below that needed to support the reaction (for instance dry indoor environment).

Due to possible differences in local geology, care should be taken to transfer experience from one region to another.

7.8.8.4 Avoidance-of-deterioration approach

To support an alkali-aggregate reaction, reactive aggregate, sufficient humidity and alkalis need to be present in the concrete member. If one or more of these conditions missing, avoidance of reaction will be achieved.

The total alkali is often expressed as Na_2O equivalent = $Na_2O + 0.658 \cdot K_2O$

The use of linings and protective coatings may avoid ingress of alkali from the external environment

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7.8.9 Delayed ettringite formation

If concrete is exposed to high temperature over a certain period in its early life, delayed ettringite formation can occur depending on humidity and concrete mix design (alkali content, chemical composition of cement, use of additions etc).

7.8.9.1 Probabilistic safety format

At the time of publishing this Model Code, models to be included here are still under discussion. A probabilistic approach for SLD is therefore not feasible.

7.8.9.2 Partial safety factor format

At the time of publishing this Model Code, models to be included here are still under discussion. A partial safety factor approach for SLD is therefore not feasible.

7.8.9.3 Deemed-to-satisfy approach

Unless specified otherwise, the peak temperature of the concrete within a component exposed to wet or cyclic wet environment shall not exceed 70°C, unless data are provided to prove that, with the combination of materials used, higher temperatures will have no significant adverse effect on the service performance of the concrete.

7.8.9.4 Avoidance-of-deterioration approach

To support delayed ettringite formation, high temperatures at early age, critical contents of SO_3 and C_3A of cement, enough water and sufficient calcium hydroxide in the porous solution need to occur in the concrete member. If one or more of these conditions are missing, avoidance of the reaction will be achieved.

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Examples of accidental and/or exceptional events to be considered are loads by a gas explosion, collision loads, impact loads on roofs, loads by extremely high soil water, loads due to fire or earthquakes, consequences of human errors, deterioration effects, etc.

This strategy accepts some components of the system to fail, but requires that the total damage to the system is limited. When applying this strategy, it shall be verified that the structure has sufficient redundancy and possibilities to mobilise so called alternative load paths. Redundancy significantly increases system robustness, and is often the only viable means of ensuring sufficient robustness. On the other hand, achieving redundancy may be difficult and or expensive, and is therefore often limited to critical parts of the system.

This strategy does not accept that some components of the system fail. When applying this strategy, the structure shall be designed strong enough to withstand every possible load, e.g. by choosing components whose bearing resistance significantly exceeds the expected actions. In general, this strategy leads to disproportionally expensive solutions.

Non structural measures are based on the general concept of avoiding actions (prevention) and reactions (protection), and mitigation of consequences. Non structural measures can be very efficient for specific accidental actions.

According to the approach directed at avoiding actions, one reduces the risk, e.g. by barriers (for collision), quality assurance (for errors), maintenance (for deterioration) or prohibition (e.g. prohibition of transport of dangerous substances through a tunnel).

7.9 Verification of robustness

7.9.1 General

It should be avoided with adequate reliability that accidental and/or exceptional events, or failure of a structural component, cause disproportional damage of a large part of the structure or even total collapse of the whole structure. Hence, the robustness of the system (i.e. adequate safety in relation with accidental and/or exceptional events) shall be analysed.

In order to attain adequate robustness of the system one or more of the following strategies may be followed:

– structural measures:

  • limiting the amount of structural damage;

  • designing the key structural components to withstand every possible load;

– non-structural measures:

  • reducing the probability that the action occurs or reducing the action intensity (prevention);

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According to the approach directed at reducing the effect of reactions, one reduces the risk, e.g. by sprinklers (for fire), relief openings (for explosions) or supplemental damping and/or base isolation (for earthquake).

- reducing the effect of the action on the structure (protection);

- mitigating the consequences of failure.

In design of conventional structures one might eliminate the use of an explicit robustness requirement and avoid verification of redundancy by taking into the design explicitly as many aspects related to specific accidental actions as possible and applying relevant non-structural measures to deal with them. This approach offers the possibility to reduce the risks in the most economical way. However, if robustness requirements are explicitly considered in design, it shall be verified that the structure has sufficient redundancy and possibilities to mobilise alternative load paths. Accordingly, the redundancy of the system shall be verified. For conventional structures, the simplified approach for verification of redundancy is recommended, see subclause 7.9.2.

In design of unconventional structures (e.g. large structures, new materials, new design concepts) robustness shall be achieved by limiting the amount of total damage to the system. Accordingly, redundancy of the system shall be verified. For the verification of redundancy of unconventional structures the simplified approach is recommended, see subclause 7.9.2.

In case of existing structures, robustness of the system may be verified by assessing the effectiveness of applied structural and non-structural measures. In particular, redundancy of the system may be assessed according to he simplified procedure given in subclause 7.9.2.

Strategies for retrofitting an existing structure, which has inadequate safety in relation with accidental and/or exceptional events are in principle similar to those recommended for design and depend on the type of structure. However, the cost of improving the redundancy of the system (i.e. costs of bringing the structure into compliance with the alternative load path principle) may be so high as to make this approach impractical. Often the only rational approach for improving the robustness of an existing system is designing the key structural components so that they can resist any potential load.

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The redundancy of a structural system is related to its strength and shape, where particularly the degree of static indeterminacy (number of potential alternative load paths) is important.

Besides, global design strategy based on classical requirements on sufficient ductility and tying of elements can be applied to achieve some degree of redundancy.

The refined verification procedure is extremely complicated and does not work for unforeseeable hazards.

The probability that the element under consideration is removed by some cause, not yet considered in design, depends on the sophistication of the design procedure and on the type of structure.

For unconventional structures the probability of having some unspecified failure cause is substantial. If in addition new materials or new design concepts are used, unexpected failure causes become more likely.

7.9.2 Specific methods to improve robustness by structural measures

7.9.2.1 Robustness by creating an alternative loading path

It should be made sure that in the case of an extreme event an alternative loading path can be mobilized around an area of local damage.

Two procedures for progressive collapse analysis can be followed to verify the redundancy of a system: a refined procedure and a simplified procedure.

According to the refined procedure, the occurrence and the effect of an accidental and/or exceptional action (impact, explosion, etc) are simulated for all possible action scenarios. The damage of the structural components is calculated and the stability of the remaining structure is assessed. Next, the consequences are estimated in terms of the number of casualties and economic losses. Various measures can be compared on the basis of economic criteria.

According to the simplified approach, the situation is considered that any structural component that has significance for the bearing capacity of the structure can lose its bearing function. It is then verified if the remaining part of the structure can withstand the frequently occurring loads with a defined target reliability for alternative load paths, for a relatively short period of time, necessary to mobilize the users of the structure and/or to carry out necessary repair or strengthening.

This target reliability for alternative load paths depends on the normal safety target for the structure, the period under consideration (hours, days or months) and the probability that the element under consideration is removed (by other causes then already considered in design).

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For conventional structures it should be possible to include all relevant collapse origins in the design. Although it will always be possible to think of failure causes not covered by the design, those failure causes will have a remote likelihood and may be disregarded on the basis of theoretical arguments.

In a building with a frame structural system a storey-sway mechanism can be prevented and a beam-sway one enforced, by satisfying around all beam-column joints the condition:

$$\sum M_{Rc} \geq \gamma_{Rd} \sum M_{Rb} \qquad (7.9\text{-}1)$$

where $\sum M_{Rc}$ and $\sum M_{Rb}$ are the sums of the moment resistances of the columns and the beams framing into the joint and $\gamma_{Rd} \geq 1.0$ a factor for overstrengths not explicitly modelled (e.g., due to steel strain-hardening).

7.9.2.2 Capacity design

Capacity design ensures a hierarchy of resistances of structural components and failure modes such that pre-emptive brittle failure modes are prevented and ductile ones promoted. To this end, brittle failure modes are verified for force action effects obtained not by analysis but from equilibrium, assuming that all relevant ductile components develop their force capacities. Ductile structural components are designed then for the plastic deformations induced in them by the design actions.

Capacity design may also be used – in seismic design in particular – to prevent undesirable plastic mechanisms associated with limited global deformation capacity and enforce the intended one(s), involving only ductile structural components.

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The purpose of the verification of environmental performance is to:

i. ensure that decision-makers consider environmental impacts before deciding whether or not to proceed with new projects, and

ii. to differentiate the performance of alternative solutions with regards to their environmental impact.

The LCA shall consist of four phases:

– Phase I. Goal and scope definition;

– Phase II. Inventory analysis;

– Phase III Impact assessment; and

– Phase IV. Interpretation.

with the boundary of the life cycle assessment (LCA) scoped in Phase I using appropriate assumptions. The life cycle assessment of environmental impacts is conducted in the entire lifecycles, respective stages of the lifecycles, or certain ranges of the lifecycles.

The general information on LCA is obtained in following standards:

– ISO 14040 “Environmental Management – Life cycle assessment – Principles and framework”;

– ISO 14041 “Environmental Management – Life cycle assessment – Goal and scope definition and inventory analysis”;

– ISO 14042 “Environmental management – Life cycle assessment – Life cycle impact assessment”;

– ISO 14043 “Environmental management – life cycle assessment – Life cycle interpretation”; and

– ISO 14044 ‘Environmental management – Life cycle assessment – Requirements and guidelines”.

7.10 Verification of sustainability

7.10.1 Impact on environment

7.10.1.1 General

When conducting design, construction, use, conservation and demolition of a concrete structure, as well as with regard to the process of recycling and disposal, it shall be confirmed at the design stage that the environmental performance required for the structure is satisfied. This is undertaken using a Life Cycle Assessment (LCA) analysis.

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However, the ISO 14000 series was developed for goods and services. The ISO 13315 series will provide the appropriate information in the future.

When an assessment is conducted, the system boundary shall be set according to the purpose of the assessment and reliable inventory data prepared. The general principles, framework and basic rules on environmental management related to concrete and concrete structures are provided in ISO 13315-1.

In the integration of each impact using a unified index, a reasonable weighing among each effect is needed. When using commercially available software for an evaluation, an understanding of the purpose and characteristics of the software is needed in regards to analyses being undertaken.

System boundaries need to be defined with due consideration of each source component and its impact in the final decision. For example, a bridge project maybe sensitive to the materials production, execution and transportation but less to energy cost savings through the life of the structure due to its thermal mass. On the other hand, for a building structure the impact of thermal mass on GHG savings over the life of the structure may be significant and, thus, included within the system boundary.

The range of assessment for consideration of environmental impact should be proportionate with the scale of the structure or project being considered.

Current practice in many countries is assessment through broad-based green building rating schemes. The following are examples of some current national schemes: BREEAM in the UK and BREEAM-NL in the Netherlands, HQE in France, LEED in the USA, CASBEE in Japan, Green Star in Australia, and Green Star-NZ in New Zealand.

Such approaches, however, may not always lead to more sustainable products as they involve many assumptions. For example, in building structures potential benefits of embodied energy, higher durability and lower maintenance are not generally considered in current schemes, although this may change in time.

In conducting an assessment:

- the system boundary shall be set with consideration to the contribution of each component to the total and the sensitivity to the result of excluding any component;
- reliable inventory data shall be prepared with consideration to the location of the structure and the location, supply and manufacturing processes of source materials and products.

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An environmental impact calculation (EIC) through a full or partial life cycle analysis (LCA) at the project, structural or structural element level can be undertaken as a measure of optimization at the level considered. Currently, there are three common measures adopted for such measurements; a measure of the embodied energy (EE), of CO_2 emissions and the global warming potential (GWP). Any one, two or all measures may be used in combination with results benchmarked against existing or common practice. Examples of such an approach are given in Appendix E of *fib* Bulletin 47 "Environmental design of concrete structures – general principles" (*fib*, 2008). Other measures may also be appropriate; ISO/DIS 13315-1 provides the framework and basic rules on environmental management related to concrete and concrete structures.

The GWP equates greenhouse gases and their expected influence on global warming to an equivalent volume of CO_2. It is an indicator of impact over a set time period with time periods typically of 20, 50 or 100 years. For example, the 100 GWP for the three more common greenhouse gas emissions (carbon dioxide, nitrous oxide and methane) is:

100-year GWP = $CO_2 + 298N_2O + 25CH_4$

and has units of kilograms of CO_2 equivalent per kilogram of material.

To undertake an EIC an inventory of materials is needed and will vary from country to country and location to location, depending on local materials and practices.

The calculation shall consider the total quantity of each material (e.g., concrete, reinforcing bar and strand) used in the structure, or structural component, under consideration with the total calculation being the sum of the volume of each material multiplied by their respective unit quantity as per the inventory of materials.

The inventory data should be appropriately collected and evaluated. The JSCE provides an example of current inventory data related to concrete (JSCE Guidelines for Concrete No.7: Recommendation of environmental performance verification for concrete structures (Draft), 2006). However, it should be noted that the inventory data varies depending on various conditions and may change significantly in the future.

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Environmental aspects subject to verification are to be selected and then the set value (S) is determined as the performance requirement. Performance requirements may be given to the designer as a limiting value that can be used for verification, according to legal regulations or by instructions from the owner.

The performance (R), which is to be achieved by the structure or element, is quantified by using the same index or environmental aspect as that adopted for the regulation, limit or target value (S) for which S was set.

Whether or not a structure satisfies the performance requirements for the environmental aspects being the subject of verification, is confirmed by whether or not $R \geq S$ or $R \leq S$, as applicable. The direction of the inequality sign varies by the type of environmental aspect being considered. For example, in cases involving the lifecycle carbon dioxide emission, verification is achieved by setting the upper limit (S) and confirming that it is higher than the estimated carbon dioxide emission (R) of the concrete or structure, whichever is being considered.

In cases where the impact on environment is to be reduced by a factor from a standard value, the verification is conducted by confirming that the ratio of the actual impact to the standard impact is lower than the specified impact reduction factor.

In cases involving recycled product substitution, it may be possible to decide that the requirement is met if the amount used (R) is larger than a predetermined minimum (S).

Impact on the environment shall be evaluated using applicable indexes for each environmental effect to be verified.

Performance requirements shall be specified by criteria such as target indices (resources, energy, emission), means indices (reduce, reuse, recycle), and/or combined indices such as embodied energy or global warming potential.

7.10.1.2 Verification

Environmental performance of a concrete structure shall be verified by confirming that the retained performance (R), defined by using appropriate indexes with regard to the impact on environment, is larger (or smaller) than the set value (S) of the relevant performance requirement.

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Where past results and other data have shown that concrete, reinforced concrete or a concrete structure satisfies a certain range of specifications (such that the constituent materials used for the concrete, the shape/size of the structure and the execution method are similar) and the performance is above a certain level, the specifications that are satisfied for the concrete or concrete structure may be used in place of performance verification, without conducting further quantitative evaluations of an individual structure or structural element.

For elements where the performance has been previously established to meet the specification criteria, the environmental performance may be deemed to be satisfied.

7.10.2 Impact on society

7.10.2.1 General

The primary purpose of the verification of impact on society is to bring about a more sustainable and equitable biophysical and human environment.

When the aesthetics as an impact on society is considered in the design of a concrete structure, it shall be confirmed that the aesthetics required for the structure are satisfied. The primary purpose of the verification of aesthetics is to create graceful structures in harmony with their environment at a reasonable cost.

When conducting, design, construction, use, conservation and demolition of a concrete structure as well as with regard to the process of recycling and disposal, it shall be confirmed at the design stage that social performance required for the structure is satisfied.

Social Impact Assessment (SIA) is a methodology to review the social effects of infrastructure and building projects and other development interventions.

At present, there are no general criteria for evaluating quantitatively the impact on society of a structure. Therefore, depending on the purpose of the structures and the intents of designers, appropriate rating or target indexes should be set (with the weighing).

The impact on society shall be evaluated using appropriate indexes regarding the social effects of objects to be verified.

When the aesthetics is considered, performance requirements for aesthetics shall be specified. At present, there is no general rating index for evaluating quantitatively the aesthetics of a structure. Therefore, depending on the purpose of the structures and the intents of designers, appropriate indexes to evaluate structural beauty, artistic beauty, scenery beauty, elegance, etc., should be set with the weighting

Performance requirements for impact on society shall be specified.

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In the lack of the general rating index for evaluating quantitatively the impact on society of a project, it is possible to evaluate retained performance (R) on the impact on society of a project, for example, from a questionnaire to people. The designer will give social aspects to be considered and set the values (S) based on the regulation, the requests from an owner or his/her intents.

Regarding the aesthetics, it shall be verified by confirming that the retained performance (R) defined by using appropriate indexes regarding the aesthetic aspects to be verified is larger than the set value (S) of performance requirement.

In the lack of the general rating index for evaluating quantitatively the aesthetics of a concrete structure, it is possible to evaluate retained performance (R) on aesthetics of a concrete structure, for example, from a questionnaire to people. The designer will give aesthetical aspects to be considered and set the values (S) based on the regulation, the requests from an owner or his/her intents.

When a specification, in which the examples for the harmony of ratio, mass, light, perspective, colour, and texture between various elements of a structure are provided, is utilized, the verification may be omitted.

7.10.2.2 Verification

Social performance of a concrete structure shall be verified by confirming that the retained performance (R) defined by using appropriate indexes regarding the impact on society is larger (or smaller) than the set value (S) of relevant performance requirements.

Regarding the items for which performance can be ensured by providing specifications, the verification may be omitted.

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7.11 Verifications assisted by numerical simulations

7.11.1 Purpose

Numerical simulation is performed by numerical methods of structural analysis based on non-linear solution methods and material models reflecting the expected real behaviour under a given type of loading and given environmental conditions.

Numerical simulation is another way of checking structural performance and is an alternative to physical testing in a laboratory or on the site. Unlike in real testing, simulation is not limited by structural size, load magnitude and testing facility. However, the power of computer simulation is limited by the chosen numerical model and its objectivity so that adequate checks should be exercised to verify the results and conclusions.

The verification assisted by numerical simulations should ensure that the design leads to the reliability level required by this Model Code.

7.11.2 Methods of numerical simulation

7.11.2.1 Numerical model

The model formulation should be based on principles of mechanics, namely it should satisfy requirements on equilibrium of forces, compatibility of displacements and constitutive laws. The inevitable errors due to numerical approximations should be controlled by solution-error criteria. It is reasonable to maintain a balanced approximation between all model parts.

The complexity of the model grows with each new feature. Therefore, it is appropriate to include only those features, which are significant for the given case, in order to keep the model as simple as possible.

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Typical algorithm for non-linear finite element analysis is schematically illustrated in Figure 7.11-1.

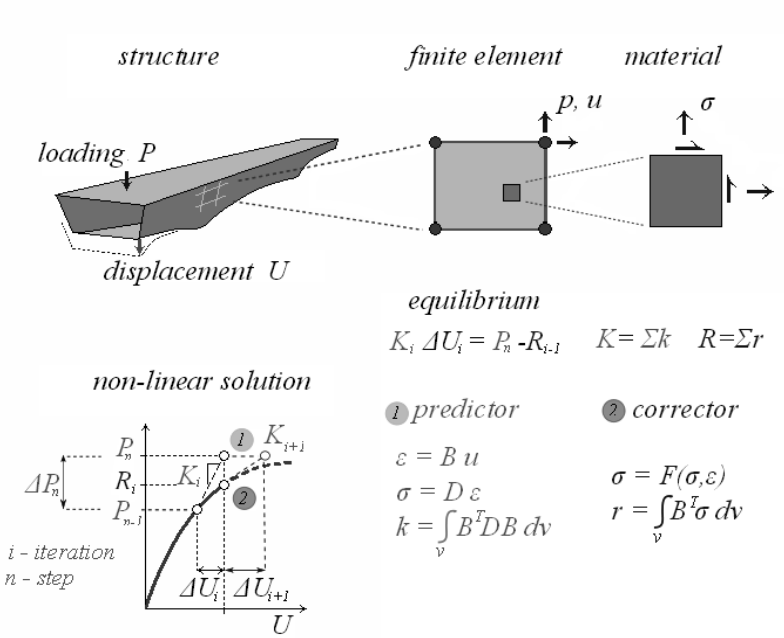


Figure 7.11-1: Typical algorithm for non-linear finite element analysis

The nonlinear solution is performed by predictor-corrector iterative process. In the predictor, point 1, the solution is estimated by a linear analysis based on tangent or initial material stiffness D. The solution is improved in the corrector, point 2, based on non-linear constitutive law F. The iterative process is stopped when difference between predictor and corrector is acceptably small. A balanced approximation of numerical methods in all model parts, that is in structural discretization, element formulation and material laws, should be maintained.

Errors of non-linear solution are controlled by convergence criteria. Solution convergence is satisfied, when the error is within prescribed limits. In case of the stiffness method, as illustrated in Figure 7.11-2, the most

7.11.2.2 Finite element method

The finite element method is typically used for the numerical solution of a continuum problem. Depending on the type of formulation (stiffness, compliance and mixed methods) the results are by definition different from the exact solution.

In case of the most widely used stiffness method, the shape of displacement field is assumed and equilibrium is satisfied only in integral sense. That means, the best possible equilibrium is found for a given approximation (finite element type and size). The internal stresses are lower, compared with an exact solution.

A finite element formulation should satisfy the requirement of convergence to the exact solution by reducing the element size (and/or increasing the number of degrees of freedom).

Thus, independently of the material model, the approximations introduced by the finite element formulation only, can be a significant source of errors in numerical analysis.

Similarly, in other numerical methods, the errors due to approximations should be adequately checked.

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significant convergence criterion is the error in force equilibrium. In addition increments of displacements or residual energy can be checked. Choice of adequate error tolerance is an important aspect of nonlinear analysis. The admissible errors must be appropriately validated, for example by a convergence study, in which results obtained with different tolerances are compared.

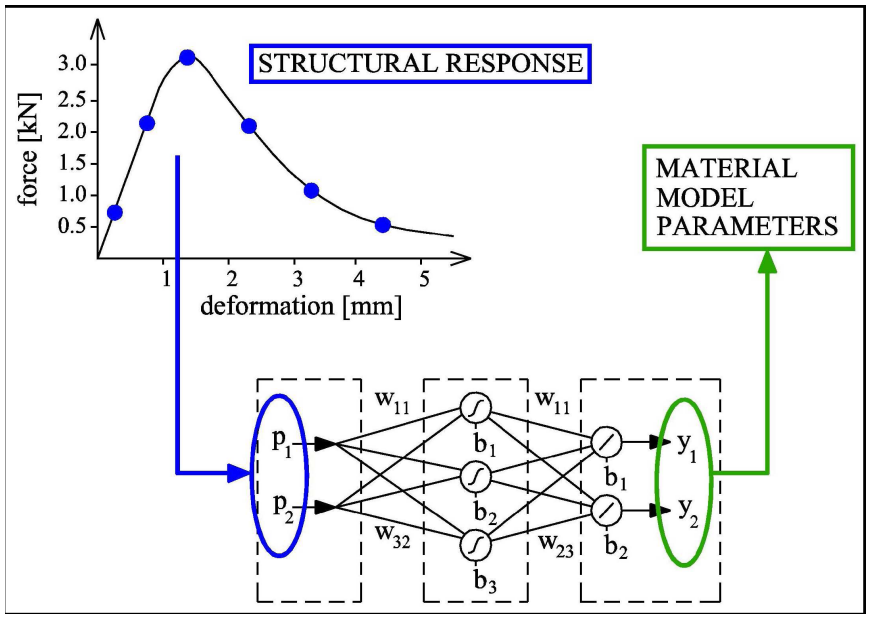


Figure 7.11-2: Typical algorithm for identification of the material model parameters

7.11.2.3 Material models

Material models that are used for concrete, reinforcement and their interaction, shall capture all significant and relevant features of material behaviour for the problem under consideration.

Constitutive laws should be based on principles of continuum mechanics and failure and must ensure objectivity of solution in the context of numerical methods.

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Material models for concrete are usually based on the following theories:

- nonlinear fracture mechanics;
- theory of plasticity with plastic flow rule;
- microplane theory;
- damage theory;
- visco-elastic and visco-plastic theory.

The choice of an appropriate theory depends on the specific application.

Basic material tests serve to validate the constitutive relations. Basic material tests are made on simple structures, with the aim to reduce the influence of geometry and boundary conditions under well-defined stress and strain conditions. Examples of such tests are compressive tests of concrete cylinders, fracture tests of concrete prismatic specimens subjected to three-point loading and tension stiffening tests in uniaxial tension of reinforcing bars embedded in concrete members. Each type of behaviour usually requires a special material test. These test are typically described in codes for material testing, such as those recommended by RILEM.

The aim of structural tests is to validate the ability of algorithm or software to objectively reproduce a certain structural behaviour. This is often performed by means of benchmark calculations For example, if a shear wall should be simulated, a validation of the software by shear wall experiments should be made available. Such studies can be considered as rational basis for choosing adequate material models and software for a given structure.

Mesh sensitivity tests should be performed, or made available, in order to validate the finite element mesh of the numerical model. At least three mesh cases with different element sizes should be tested and their effect on the resistance should be evaluated. In case of a significant mesh sensitivity, when at least two different mesh sizes do not provide sufficiently similar results, the results shall be considered as not objective.

Models for material softening, i.e. materials exhibiting a decrease of strength after reaching certain ultimate stress value, should include appropriate regularization techniques in order to reduce mesh sensitivity of strain-based formulations of constitutive laws.

7.11.2.4 Validation of numerical models

The procedures used in numerical analysis and the choices made with regard to methods (subclause 7.11.2.2) and material models (subclause 7.11.2.3), should be validated by experiments and benchmarks. This validation should include:

- basic material tests;
- structural tests;
- mesh sensitivity tests.

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The objective of non-linear analysis is to simulate the real structural behaviour and to evaluate the representative value of resistance, see subclause 4.6.2.1. Such a task can be also formulated as a prediction of the most probable real resistance, which is the mean value of resistance. Therefore, the mean resistance is chosen as a reference for safety assessment by non-linear analysis. The uncertainty due to random variation of material properties (and possibly to other parameters of resistance) is described by the random variation of resistance. In addition a model uncertainty must be included separately.

The design condition is compatible with the one applied within the partial factor design method, Eq. (4.5-8). However, the design resistance is in this case considered as a design variable and a global resistance method is applied.

The methods for the design resistance assessment are listed according to the level of their probabilistic justification. In the *probabilistic method* design resistance is explicitly evaluated for required failure probability or reliability index. In the *global resistance method* the design resistance is estimated based on simplified probabilistic approach assuming significant approximations. In the *partial safety factor method* the design resistance is determined directly using design values of random variables without evaluation of global safety. The *partial safety factor method* is comparable with the partial factor design method applied to global resistance. The choice of a relevant method for the calculation of the design resistance depends on

7.11.3 Safety formats for non-linear analysis

7.11.3.1 General

The design principles based on the global resistance design method described in clause 4.6 apply, whereas the character of numerical simulation and non-linear analysis must be considered appropriately.

The design condition is considered in the form of Eq. (4.6-2) and (4.6-4) with separated uncertainties for actions and resistance.

$$F_d \le R_d\,,\quad R_d = \frac{R_m}{\gamma_R^* \gamma_{Rd}} \tag{7.11-1}$$

where:

F_d is design value of actions;

R_d is design resistance;

R_m is the mean value of resistance;

γ_R^* , γ_{Rd} are global resistance safety factor and model uncertainty factor, respectively.

The design resistance R_d can be evaluated by various methods based on different levels of implementation of probabilistic theory:

- probabilistic method for required reliability index or failure probability;
- global resistance method;
- partial safety factor method.

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engineering judgement while taking into consideration the resistance model and possible uncertainties. In practical design a usage of at least two methods is recommended, which provides independent verifications of limit state.

The probabilistic analysis based on numerical simulation with random sampling can be briefly described as follows:

(1) Formulation of a numerical model based on non-linear finite element method. Such a model describes the resistance function r (**r**) and can perform deterministic analysis of resistance for a given set of input variables.

(2) Randomization of input variables (material properties, dimensions, boundary conditions, etc.). This can also include some effects, which are not in the action function s (**s**) (for example prestressing, dead load etc.). Random properties are defined by a random distribution type and its parameters (mean, standard deviation, etc.). They describe the uncertainties due to variation of resistance properties. The randomization can be done by two methods: (1) *Random variables*, where the parameter is constant within a sample (structure), but changes between samples. (2) *Random field*s, where the parameter is randomly variable within a sample. A correlation of random variables should be considered appropriately.

(3) Probabilistic analysis of resistance. This can be performed by the numerical method of Monte Carlo-type of sampling, such as the LHS (Latin Hypercube Sampling) method. Results of this analysis provide random parameters of resistance, such as mean, standard deviation, etc. and the type of distribution function for resistance.

(4) Evaluation of design resistance based on the reliability index β or probability of failure.

For evaluation of design resistance, i.e. only right side of design condition (4.6-1) is considered: the reliability index shall be considered in the form of the product $\alpha\beta$, where β is the overall reliability index and α is the reduction factor for resistance. In absence of more detail data, the value $\alpha\beta$ = 0.8x3.8=3.04 can be considered for the reliability index. This corresponds to the failure probability $P_f = 10^{-4}$ and to the average situation in design according to partial safety factor method.

7.11.3.2 Probabilistic method

The design value of resistance R_d can be evaluated by probabilistic analysis following the general safety format according to clause 4.4. In this approach the resistance function r (**r**) is represented by non-linear structural analysis. The safety can be evaluated with the help of the reliability index β, or alternatively by the failure probability P_f taking into account all uncertainties due to random variation of material properties, dimensions, and possibly other random effects.

Global design resistance R_d shall be obtained as

$$R_d = \frac{1}{\gamma_{Rd}} R(\alpha\beta) \tag{7.11-2}$$

where

$\gamma_{Rd} > 0$ is the model uncertainty factor;

$R(\alpha\beta)$ is the resistance corresponding to reliability index β, which is reduced by factor α in order to account for a separate safety assessment of resistance, i.e. only right side of design condition (4.6-1).

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In this method the mean material properties for the calculation of resistance are considered, whereas different uncertainties of steel and concrete are accounted for. The safety approach can be demonstrated by a simplified reasoning when considering isolated limit states of each material and current partial factors.

The safety factors for mean in case of steel failure, γ_{GL}^{steel} is defined as:

$$f_{sd}\,\gamma_{GL}^{steel} = f_{sd}\,1.1\,\gamma_s = f_{sm} \tag{7.11-7}$$

$$\gamma_{GL}^{steel} = 1.1 \times 1.15 = 1.27 \tag{7.11-8}$$

where 1.1 is the factor for the mean strength and γ_s =1.15 is the partial safety factor for the characteristic strength of the steel.

The safety factor for mean in case of concrete failure, $\gamma_{GL}^{concrete}$ is defined as:

$$f_{cd}\,\gamma_{GL}^{concrete} = f_{cd}\,1.1\,\gamma_c = f_{cm} \tag{7.11-9}$$

$$\gamma_{GL}^{concrete} = 1.1 \times 1.5 = 1.65 \tag{7.11-10}$$

where 1.1 is the factor for the mean strength and γ_c =1.5 is the partial safety factor for the characteristic strength of the concrete.

Since the partial factors are formulated for the same failure probability, they yield different safety factors for mean of these two materials. In order to prevent two different global factors the mean strength of concrete is modified to provide for the greater safety required for concrete.

A new reduced mean concrete strength can be derived according to:

$$f_{cm} = \frac{1.1\,\gamma_s}{\gamma_c} f_{ck} \cong 0.85 f_{ck} \tag{7.11-11}$$

It yields the new global safety factor of concrete:

7.11.3.3 Global resistance methods

7.11.3.3.1 Global resistance factor method

Design resistance is calculated from

$$R_d = \frac{r(f_m, ...)}{\gamma_R\,\gamma_{Rd}} \tag{7.11-3}$$

The function *r* represents the non-linear analysis with mean input material.

Reinforcing steel properties:

$$f_{ym} = 1.1 f_{yk} \tag{7.11-4}$$

where f_{ym} is the mean value for the yield stress of the steel

Concrete properties:

$$f_{cf} = 1.1\,\alpha_{cf}\,f_{cf\,k}\frac{\gamma_s}{\gamma_c} \tag{7.11-5}$$

where f_{cf} is the concrete strength parameter reduced to account for greater random variability of concrete. It can be: compressive strength, tensile strength, or fracture energy density. In analogy other parameters entering the numerical model may be considered.

Coefficient α_{cf} accounts for sustained load effect and unfavorable effects of load application and represents the same uncertainty as given by α_{cc} , α_{ct} in subclause 7.2.3.1.4. The range of this parameter is (0.85; 1). Using other then lower limit must be substantiated by nonexistence of such uncertainties.

f_{cfk} is a characteristic value of the relevant concrete property (compressive strength, tensile strength, fracture energy density).

This can be further simplified by inserting values of the partial factor for steel γ_s =1.15 and for concrete γ_c =1.5 resulting in:

$$f_{cmd} = 0.85 f_{ck} \tag{7.11-6}$$

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$$\gamma_{GL}^{concrete} = \frac{f_{cm}}{f_{cd}} = \frac{0.85 f_{ck}}{\frac{f_{ck}}{1.5}} = 1.27 \qquad (7.11\text{-}12)$$

The unique safety factor 1.27 for mean for steel and concrete is obtained.

Probabilistic studies indicate that the random distribution of resistance of reinforced concrete members can be described by a two-parameter lognormal distribution with the lower bound at origin. Therefore, the present method is based on the assumption of a lognormal distribution, which can be identified by two random parameters: R_m – mean resistance and V_R – coefficient of variation of resistance. Other parameters, such as a characteristic resistance, can be derived since they are dependent on the above definition.

Based on these assumptions it is proposed to identify the random distribution of the resistance by mean R_m and characteristic R_k values. The following expression for the characteristic value can be regarded as an acceptable approximation:

$$R_k = R_m \exp(-1.65 V_R) \qquad (7.11\text{-}17)$$

Assuming the probability for the characteristic value as 0.05 the coefficient of variation can be expressed as

$$V_R = \frac{1}{1.65} \ln\left(\frac{R_m}{R_k}\right) \qquad (7.11\text{-}18)$$

Finally, the global factor for the mean resistance γ_R can be calculated from

$$\gamma_R = \frac{R_m}{R_d} = \exp(\alpha_R \beta V_R) \qquad (7.11\text{-}19)$$

The values of parameters according to subclause 3.3.3.1 of this Model Code can be considered to be $\alpha = 0.8$, $\beta = 3.8$, corresponding to probability $P_f = 10^{-3}$, which leads to the final expression for the global factor as a function of coefficient of variation of resistance:

The reduction factor 0.85 shall be used to obtain the reduced mean values of other material properties of concrete like tensile strength, fracture energy, bond strength, etc.

The partial factor of resistance shall be $\gamma_R = 1.2$

The model uncertainty factor shall be considered by the factor $\gamma_{Rd} = 1.06$

7.11.3.3.2 Method of estimation of a coefficient of variation of resistance

An estimate of mean and characteristic values of resistance shall be calculated using corresponding values of material parameters:

$$R_m = r(f_m, ...), \quad R_k = r(f_k, ...) \qquad (7.11\text{-}13)$$

where r represents the non-linear analysis and f_m and f_k are mean and characteristic values of input material parameters, respectively.

The coefficient of variation V_R of resistance shall be determined from:

$$V_R = \frac{1}{1.65} \ln\left(\frac{R_m}{R_k}\right) \qquad (7.11\text{-}14)$$

The global resistance factor γ_R for mean shall be determined from:

$$\gamma_R = \exp(\alpha_R \beta V_R) \qquad (7.11\text{-}15)$$

where α_R is a sensitivity (weight) factor for the reliability of resistance and β is a reliability index.

The design resistance is then calculated from:

$$R_d = \frac{R_m}{\gamma_{Rd}\, \gamma_R} \qquad (7.11\text{-}16)$$

The global resistance factor γ_R is related to the random variation of resistance due to basic material parameters and failure modes. The model uncertainty factor for well validated numerical models is proposed as $\gamma_{Rd} = 1.06$. In case of low-level validation higher model uncertainy values > 1.06 should be used.

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$$\gamma_R^m = \exp(3.04\ V_R) \qquad (7.11\text{-}20)$$

It should be emphasized, that this method depends on reliable and justified values of mean and characteristic values of material parameters used for input of resistance analysis and relevant to the failure mode under consideration.

In this case the structural analysis is based on extremely low material parameters in all locations, which does not correspond to the probabilistic concept of simulation. This may cause deviations in structural response, e.g. in failure mode. However, the case studies indicate that the partial factor method is practically applicable. The method of partial factors may be used as a safe estimate in absence of a more refined solution.

The model uncertainty factor γ_{Rd}, as proposed in subclause 4.5.1.3 (b) should be used appropriately. In a standard situation the model uncertainty is included in the partial safety factor $\gamma_M = \gamma_m * \gamma_{Rd}$ and reflected in design values of basic material parameters $f_d = f_m / \gamma_m$. Therefore, in case f partial factor method the model uncertainty factor shall not be applied to the design resistance R_d.

The advanced identification methods are based on numerical methods of probabilistic methods and optimization techniques. They are utilizing stochastic simulation based on Latin Hypercube Sampling combined with artificial neural network. Such approach reflects the sensitivity of resistance to the input parameters and can offer a best-fit set of material parameters.

The weight of most important parameters, which effect the resistance and failure, is automatically recognized by the method. Such methods are more rational and superior to the heuristic trial-and-error fitting, especially in case of many input parameters. Simulation of resistance is performed by nonlinear finite element analysis. Virtual simulation is used to create a training set for the neural network model. Finally, the trained model can provide the best set of material parameters for a given experimental load-deflection curve.

7.11.3.4 Partial factor method

The method of partial factors can be applied, in which the design resistance R_d is calculated using the design values as input parameters f_d for the nonlinear analysis:

$$R_d = r(f_d, ...) \qquad (7.11\text{-}21)$$

The design parameters f_d are obtained by the method described in subclause 4.5.1.3 and $r(f_d, ...)$ represents the nonlinear analysis model.

7.11.4 Resistance parameter identification

Resistance parameter identification is an inverse method to the assessment of resistance. In a standard design situation the assessment of structural resistance is calculated for a given set of materials parameters using structural analysis. The numerical simulation, such as non-linear finite element analysis, is an advanced way of modelling structural resistance.

In an inverse task the structural resistance is known and material properties are sought using a structural model. This can be described as parameter identification for a certain structural model. The aim of identification is to identify the material parameters, which ensure the best fit of the calculated and given resistances for the considered structural model.

This method can be used for identification of material parameters, in basic material tests, or in tests of existing structures. Material parameters determined by identification can be used for simulation of other structures in similar conditions.

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In this context, 'response' may mean stiffness, strength or durability characteristics.

Examples of such structural members or structural systems may be

– assemblages of reinforced concrete prefabricated elements;
– units with special shape (polyhedric panels, bunkers).

Examples of special design problems studied experimentally may be

– prestress losses;
– special production processes (accelerated curing, etc.).

Examples of tests which generally cannot be statistically interpreted

– tests on frictional losses of prestress made by measurements of forces transmitted by tendons on the structure during construction;
– tests on uncertainties related to static equilibrium made by measurements on the smaller reaction of supports;
– single tests on large dimension elements or existing structures.

When dealing with cases in which a statistical interpretation of results is not possible, a combined procedure may be used to reduce the entity of experimental tests, as follows:

– experimental tests integrated by “a priori knowledge”, using a Bayesian approach on structures similar both in geometry and age. This kind of approach allows for the reduction of experimental tests and for a more reliable estimation of resistance.
– numerical simulations with non-linear analyses followed by a sensitivity analysis of the parameters showing a significant influence on the result. Such kind of analyses should be also validated by the comparison of different non linear programs. Either a semi-probabilistic or a completely probabilistic approach may be used, in agreement with clause 7.11.

7.12 Verification assisted by testing

7.12.1 Scope

This clause contains guidance for the experimental assessment of the response of structural members of structural systems not included in the Model Code's range of application. In very particular cases (see subclause 7.12.3) verification assisted by testing may also be used in order to modify specific provisions in this Model Code's range of application.

This clause covers only those cases in which a statistical interpretation of results is possible.

In cases in which a statistical interpretation of results is not possible, a procedure based on a combination of testing and numerical verifications, as discussed in clause 7.11, can be applied.

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– numerical simulation integrated by “later knowledge” derived from experimental test able to allow for the improvement of the relationship between actions and effect of actions. In practice the numerical models will be calibrated on the actual structural response derived from destructive and/or non-destructive test, by means of an optimization procedure involving the main mechanical parameters. Such procedure may be considered as an improvement of numerical models by “later” experimental information.

In a wide sense verification assisted by testing may cover all kinds of load tests on either scaled-down models, on special specimens or on the entire structure itself; these tests are expected to give additional information to the designer. In this respect, two particular cases may be distinguished in practice:

– *Small structural elements repeatedly produced under factory conditions*: An improvement of the existing design model is sought; a considerable number of full scale specimens are tested, taking into account all actions and influences during the lifetime of the structure where these elements will be incorporated.
– *Larger structural elements to be constructed under site conditions*: An analytical model may not be initially available or considerable modification of existing models is sought. A comparatively lower number of tests are carried out simulating the real life conditions. This case is meant to be the main case addressed by this clause.

However, the following cases are not covered by this clause:

– non-destructive test-loadings of single finished structures;
– wind tunnel or earthquake simulator tests;
– routine control tests on reinforced concrete industrial products;
– accelerated durability tests;
– proof loading.

7.12.2 Definition

Verification assisted by testing is considered here as a procedure where loading tests on limited series of representative specimens are used for the determination of the response of structural members or structural systems.

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Such cases may be deviations from the provisions of this Model Code with respect to, for example:

- geometric shapes;
- combinations of new materials;
- particular detailing and/or connections;
- severe actions.

Such cases may be considered where, for example:

- the economic importance of a project or of an element to be mass produced justifies the effort of verification assisted by testing;
- the codified model, in some particular conditions is judged to be clearly too rough.

Special care is necessary for several reasons:

- computational design versus ultimate limit states often covers indirectly serviceability as well and intermediate limit states (which for simplicity are not codified): a modification of the codified provisions for ULS might result in deficiencies in this respect;
- verification assisted by testing is not able to account for some uncertainties (e.g. on structural analysis or on parasitic phenomena due to the environment) which are covered by code provisions, nor to justify a reliability format more precise than the codified one.

7.12.3 Aims of verification assisted by testing

The aim of the verification assisted by testing is to obtain design values for the parameters governing the response of structural members and structural systems under specified load conditions with respect to a certain limit state.

Verification assisted by testing may be pursued instead of verification by calculation when:

- the calculation models are insufficient or out of this Model Code's range of application;
- the calculation models given in this Model Code are thought to lead to unnecessary uneconomic results or where greater precision is desirable.

Verification assisted by testing shall be done with special care and reasonable checks should be exercised to verify the results and conclusions.

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This is particularly important when verification assisted by testing is to be permitted for financial reasons (see subclause 7.12.3).

In this respect, see e.g.:

- the in-time variation of the basic variables (subclause 7.12.5.2);
- the condition affecting the conversion factor η (subclause 7.12.9.1.1).

If some of these variations and conditions are expected to have a systematic effect on the structural response under investigation, their complete reproduction in the laboratory shall be secured.

If such a model is not available prior to testing, trial preliminary tests should be carried out in order to facilitate parameter identification.

7.12.4 Requirements

The verification-assisted-by-test procedure should ensure that the design leads to the same level of reliability as this Model Code .

To this end all possible critical conditions, actions and possible influences expected during the lifetime of the structure shall be appropriately reproduced in the laboratory.

The whole procedure shall be developed by the designer and be approved by the relevant counterpart.

The experimental assessment shall be performed by qualified institutions with staff experienced in planning, executing and evaluating tests.

7.12.5 Planning

7.12.5.1 Calculation model-limit states

A plan should be drafted by the designer, which shall contain the objective of testing and all indications necessary for the sampling or manufacturing of the specimens, the execution of the tests and their evaluation.

The experimental procedure may cover either an ultimate limit state or a serviceability limit state. In either case, minimum knowledge of the relevant response mechanism and parameters is sought.

It is desirable that on the basis of this knowledge, an empirically or physically based calculation model is available prior to testing, in order to evaluate the response. This model is referred to as 'prior calculation model' g_R

$$R_t = g_R(\boldsymbol{X}, \boldsymbol{W}, \boldsymbol{D}) \qquad (7.12\text{-}1)$$

where:

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The response depends in general on a set of measurable quantities. The quantities which are random with respect to an elementary population are referred to as basic variables $\boldsymbol{X}$. The quantities which are considered as deterministic, with respect to an elementary population, are referred to as nominal variables $\boldsymbol{W}$. Nominal variables may be constant within a population or vary in a predetermined manner.

NOTE: The terminology used in this clause and the following ones, with regard to the variables, is not exactly the same as elsewhere in this Model Code (e.g. in clause 4.3 and subsequent) because the character of some variables (fundamental or not) is not yet established at this stage. Some variables indeed, although random, may have very little influence on the response.

In case of unidentified significant parameters, a larger scattering of results is expected.

Special care is necessary because statistical parameters of basic variables have not an intrinsic character. They generally depend on the considered statistical population, which may be very different at the level of the Code, at the level of the specimens and at the level of the expected application of the test result.

Special mention should be made of the quasi-deterministic concrete strength of the specimens which exhibit a very low variability of concrete strength. Thus, in the relevant model, the characteristic strength of in-structure concrete should be made equal to the characteristic strength of the specimen concrete.

Aspects to be considered are for example:

- the in-time development of the concrete strength positive or negative, depending on the concrete composition;
- the possible decrease of the tensile strength of concrete due to hydrothermal cyclic conditions;

R_t is the response of the available theoretical model;

$\boldsymbol{X}$ is the vector of basic variables with known statistical distribution;

$\boldsymbol{W}$ is the vector of known nominal variables;

$\boldsymbol{D}$ is the vector of unknown coefficients to be determined by the testing.

All quantities affecting the response should be present in the model either as basic variables or as nominal variables.

7.12.5.2 Information on basic variables

For the basic variables included in the calculation model, statistical parameters have to be known. If these values are not known, it is recommended to estimate them by preliminary tests. Where information is available only from a limited population, variances need to be increased accordingly.

The in-time variation of the basic variables should be taken into account in the model.

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– the possible decrease of steel strength, ductility or fatigue performance due to minor corrosion conditions.

The necessary number of tests is higher where not only mean values but also standard deviations are to be statistically assessed.

The dimensions of the specimens should, if possible, cover the entire range of the probable variation of the dimensions of the structural element.

When scaled-down specimens are used, special attention should be paid to the influence of aggregate size, size and number of re-bars, concrete workability, etc., on the response.

Other influences may be temperature, humidity, etc., conditioning the behaviour of the specimens and their response.

The actions process ('load path') should also include data regarding rate effects (e.g. load application velocity, the order of application of each influence).

Specimens taken from production may be used either for preliminary tests or in the case of small structural elements repeatedly produced under factory conditions defined in subclause 7.12.2.

Gross errors cannot be covered by this procedure.

7.12.5.3 Number of specimens

The number of tests carried out should be sufficiently large in order to lead to results with satisfactory small statistical uncertainty (see subclause 7.12.8)

7.12.5.4 Scale effects

The specimens should preferably be dimensioned as close to full scale as possible, so that the scale effects do not increase the model uncertainties.

Otherwise, similitude laws, e.g. on the basis of fracture mechanics, shall be appropriately taken into account.

7.12.5.5 Actions

The actions may be direct forces (loads in general), or imposed deformations or other influences varying in space and/or in time.

The scheduled actions process should be fully determined by a set of load (or occasionally of other influence) parameters. The actions process shall be selected so that they are representative for the anticipated scope of application of the structural member.

Each specimen may be subjected to the same or different action processes.

7.12.5.6 Origin of specimens

The specimens should be specifically manufactured for the testing.

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Quantitative and qualitative observations are highly recommended in order to check these assumptions, made implicitly or explicitly, or to allow alternative evaluations afterwards.

If, during testing, a given condition is found to influence considerably the results, this condition shall be more systematically studied as a new basic variable of the model g_R.

When a direct measurement is not possible, an indirect measurement is allowed. Then the respective conversions should be used, by introducing the necessary new variables of the conversion factors or by increasing the variance of the relevant variable.

In the case of indirect measurement of a variable, the difference of the statistical properties between the populations in the laboratory and on site shall be taken into account.

Since the response of the structural member is in general identified in terms of load intensity at which the limit state is reached, the actual limit value of the limit load should be measured with additional care.

It is recommended that measurements are redundant in order to make a mutual checking of the results possible.

7.12.6 Testing conditions and measurements

Special care should be taken in order to check that all assumptions made in the planning (see subclause 7.12.5) are satisfied.

7.12.6.1 Basic and nominal variables

The actual values of all basic and nominal variables included in the calculation model should be, as far as practicable, determined by direct measurements for each specimen and each experiment. If not, the resulting uncertainty should be accounted for.

7.12.6.2 Actions

The actual values of the imposed actions have to be recorded during the test and especially at the 'critical point' of the limit state considered.

7.12.6.3 Deformation - structural behaviour

During the tests, systematic measurements shall be carried out concerning the deformations (elongation, deflection, rotation).

7.12.7 Laboratory report

The laboratory report should contain at least:

- the name of the part asking for the verification assisted by testing;
- the names of the laboratory staff involved in the verification assisted by testing;

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- the scope of the verification assisted by testing;
- technical description of the specimens (dimensions, materials, fabrication technique, number of specimens, etc.);
- testing procedure;
- all measurements of basic and nominal variables;
- the actions process;
- the deformation of specimens;
- crack pattern;
- the failure mode and the critical material;
- photos and/or video recordings;
- the prior calculation model used;
- laboratory comments on the results obtained.

7.12.8 Statistical analysis of test results

For details, reference is made to:

- ISO 12491:1997, "Statistical methods for quality control of building materials and components";
- Kersken-Bradley M., W. Maier, R. Rackwitz and A. Vrouwenvelder with contributions by F. Bijlaard, H. Mathieu, G. Sedlacek and J. Stark, "*Estimation of Structural Properties by testing for use in limit state design, General principles for planning, sampling and evaluation for the elementary case*", JCSS Working document, IABSE, Zurich, 1990.

7.12.8.1 Estimation of the unknown coefficients *D*

The coefficients ***D*** may also be considered as random variables, but in order to check the validity of the Eq. (7.12-1), in a first step, the coefficient ***D*** should be given the best value $\boldsymbol{D_m}$.

The estimation of the coefficients ***D*** (see Eq. (7.12-1)) may be based on:

- least square methods;
- maximum likelihood methods;
- Bayesian regression analysis.

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For simple models of the type

$$R_t = d\, g_R(\boldsymbol{X}, \boldsymbol{W}) \qquad (7.12\text{-}2)$$

d is single parameter (i.e. model uncertainty) and the distribution of d is a central t-distribution. When elaborating the test a distinction should be made between variable x_i that can be measured in each test and variable x_i that cannot be measured.

Often it is known that coefficients can only take values larger or smaller than unity.

Generally, the characteristic value R_k (usually 5% fractile) is defined as the value for which:

$$P(R_t < R_k) = 0.05 \qquad (7.12\text{-}3)$$

Accordingly, appropriate statistical analysis shall be conducted (see references given in 7.12.8) or the following simplified approximation shall be applied:

$$R_k \approx d_k\, g(\boldsymbol{X_k}, \boldsymbol{W}) \qquad (7.12\text{-}4)$$

For linear models or models which can be transformed to linear, coefficients generally should be assumed as normally distributed.

Where no linear models are obtained, a Taylor expansion in the vicinity of the expected design point may be used to avoid non-linear regression analysis.

When coefficients of applied models are to be estimated, prior information on those coefficients should be derived from physical considerations.

7.12.8.2 Characteristic value

The characteristic value R_k may be calculated from test results taking into consideration the statistical uncertainty as a result of a limited number of tests.

The calculation can either be based on a predictive Bayesian estimate or on assuming a specific confidence level (normally the confidence level is taken equal to about 0.75).

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If relevant, the characteristic values $\boldsymbol{X_k}$ are substituted in R_k by other representative values depending on the combination and/or the limit state.

The material safety factor γ_m is to be adopted in accordance with this Model Code as:

$$\gamma_m = \gamma_s \quad (7.12\text{-}6)$$

or

$$\gamma_m = \gamma_c \quad (7.12\text{-}7)$$

For numerical values of the γ-factors according to the failure mode and to the material decisive for the failure see chapter 4.

The differences covered by the factor η may, for example, be:

- loading time (when concrete strength is critical, a factor equal to 0.85 should be applied at least to account for sustained loads);
- support conditions;
- humidity conditions and their alternations;
- differences in geometry;
- differences in workmanship and/or curing conditions.

Influences of systematic character should be covered by the model and not by η, e.g. in case of a brittleness factor such as $(1\text{-}f_{ck}/250)$.

Those conversion conditions included in γ_m should not be duplicated when assessing η.

7.12.9 Verification procedure

7.12.9.1 Design values

7.12.9.1.1 Design values for ULS

The design value of the response is given by the expression:

$$R_d = \eta\, R_k / \gamma_{Rd}\, \gamma_m \quad (7.12\text{-}5)$$

where:

R_k is the characteristic response defined statistically on the basis of the test results;

γ_m is the material partial safety factor adopted according to the failure mode of the material decisive for the bearing capacity;

η is a conversion factor, taking into account the differences between testing conditions and conditions in the actual structure. If a more detailed model is known, accounting separately for the role of both materials, separate conversion and safety factors may be used accordingly;

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Only influences of secondary importance may be accounted for by means of conversion factors estimated in a more or less empirical way.

γ_{Rd} should be increased according to experience in cases where the scatter of d is high (e.g. its coefficient of variation is greater than 0.15) or the expected variability of the basic variables is significantly higher on site than in specimens (see subclause 7.12.5.2), or the value of the conversion factor on site conditions seems to be less than the already adopted η value.

Experts' opinions may be used in evaluating η and γ_{Rd} provided that all systematic differences are taken into account in the tests.

For details, reference is made to ISO 12491:1997, "Statistical methods for quality control of building materials and components".

γ_{Rd} values for SLS are generally different than those for ULS. Some minor aspects of a conversion factor may as well be accounted for by this γ_{Rd} factor.

γ_{Rd} is a complementary model-uncertainty-factor intended to cover differences between the testing conditions and the actual ones in the structure, which cannot be accounted by d_k, by γ_m or by the conversion factor η.

It is also possible to make a direct estimation of the design value on the basis of the statistical results,

7.12.9.1.2 Design values for SLS

The design values for the serviceability limit states are given by the expression:

$$R_d = R_k / \gamma_{Rd} \qquad (7.12\text{-}8)$$

with R_k and γ_{Rd} as defined in subclause 7.12.9.1.1.

7.12.9.2 Verification

The most frequent verification inequality is:

$$E_d \leq R_d \qquad (7.12\text{-}9)$$

where:

E_d denotes the design value of the load effects;

R_d denotes the design value of the resistance.

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Generally Δc_{dev} can be taken equal to 10 mm. Where a quality assurance system is used, including measurement of the concrete cover, the allowance in design for deviation Δc_{dev} may be reduced to a value 10 mm $\geq \Delta c_{dev} \geq$ 5 mm. Where a very accurate measuring device is used for monitoring and non-conforming members are rejected (e.g. precast elements) the value Δc_{dev} can be further reduced.

7.13 Detailing

7.13.1 Basic principles

The dimensions of structural members shall be chosen in relation to the amount and position of the reinforcement and to the properties of the structural materials. The position and composition of the reinforcement shall allow a qualitatively perfect casting and compaction of the concrete. Special attention shall be paid to the placing of the reinforcement.

The detailing shall correspond to the structural model underlying the structural analysis. The influences of any deviations between the structural model and the actual properties of the structure shall be taken into account.

If the reinforcement is used for the earthing of construction works, the measures have to be discussed with the responsible electrical engineers, whereby particular attention shall be paid to corrosion protection.

7.13.2 Positioning of reinforcement

7.13.2.1 General

The provisions of this section apply to reinforcing bars with diameters $\leq$ 40 mm. Bar diameters above 32 mm shall only be used in exceptional cases; the principles of detailing shall be adapted if necessary.

7.13.2.2 Cover of reinforcement

The cover of reinforcement c_{nom} refers to minimum values which have to be complied with if no special protective measures are taken or no special circumstances exist. Permissible dimensional tolerances are given in subclause 4.5.1.4.4 (see Tables 4.5-1 to 4.5-3).

The concrete cover shall be specified on the drawings. It is defined as an absolute minimum cover, c_{min} plus an allowance in design for deviation Δc_{dev}:

$$c_{nom} = c_{\min} + \Delta c_{dev} \qquad (7.13\text{-}1)$$

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Table 7.13-1 gives indicative values for minimum cover with respect to durability demands, assuming moderate concrete strength (from C20/25). The values apply for a service life of 50 years. For a service life of 100 years the values of c_{min} should be increased. The necessary increase of the cover depends on the time dependant nature of the ingress of chemicals with a detrimental effect. In general the cover thickness should be increased with about 30%. The cover can be reduced if a concrete with smaller permeability is used. This reduction can be more substantial if the water cement ratio decreases and the degree of hydration increases. Also quality control plays an important part.

The cover is the largest of four values, depending on:

– technological requirements;

– bond and anchorage requirements;

– fire requirements;

– durability requirements.

The highest value applies. The cover chosen should be shown on the drawings.

In order to ensure good casting, the cover should be at least equal to the maximum aggregate particle diameter plus 5mm.

In order to ensure the transfer of the bond forces between concrete and reinforcing bars c_{min} shall be larger than:

– in case of single bars: half the diameter of the reinforcing bar;

– for bundles: the equivalent diameter $Ø_n$, see Eq. (7.6-7).

In case of posttensioned bars c_{min} shall be larger than:

– for circular ducts: the duct diameter;

– for rectangular ducts: the smaller dimension or half the largest dimension.

For pretensioned strands or wires c_{min} shall be larger than:

– 1.5 the diameter of the strand or plain wire;

– 2.5 the diameter of an indented wire

In order to fulfil the fire resistance requirements, reference is made to subclause 7.5.1.

For durability the minimum cover depends on the concrete resistance against the ingress of chemical substances in combination with the agressivity of the environment and the intended service life of the structure. For verification of durability requirements reference is made to clause 7.8.

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Table 7.13-1: Minimum cover for reasons of durability (indicative values)

Environmental class	X0	XC1	XC2 XC3	XC4	XD1 XS1	XD2 XS2	XD3 XS3
Reinforced concrete	10	15	25	30	35	40	45
Prestressed concrete	10	25	35	40	45	50	55

In the case of structural members in which concrete is cast directly against soil, the minimum value of the cover on the reinforcement is $c_{nom} \geq 75$ mm; in the case of soil preparation $c_{nom} \geq 50$ mm.

If the cover of reinforcement exceeds 50 mm, it may be appropriate (e.g. regarding fire action) to provide a skin reinforcement.

In the case of architectural concrete surfaces, in particular those with a treated surface, particular attention shall be paid to the cover of the reinforcement.

7.13.2.3 Minimum bar spacing

The spacing of reinforcing bars and prestressing tendons shall be sufficient to allow perfect casting and compaction of the concrete. Splices of reinforcement shall be staggered if necessary.

The clear distance between parallel reinforcing bars and prestressing tendons should be at least equal to the maximum aggregate particle diameter +5 mm and the diameter of adjacent bars. It shall be at least 20 mm.

In the case of dense reinforcement, the arrangement of bar bundles with a maximum of three contacting bars running in the same direction is permitted.

7.13.2.4 Forms and bends

Compliance with the minimum mandrel diameters prevents bending cracks in the reinforcing steel and failure of the concrete enclosed by the bend. The mandrel diameters shall be specified by the design engineers. The following minimum mandrel diameters have to be complied with:

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– for bends	$d_1 = 15Ø$	
– for end hooks, angle hooks, loops	$d_2 = 6Ø$	for bars ≤ 20 mm
	$d_2 = 8Ø$	for bars > 20 and ≤ 30 mm
	$d_2 = 10Ø$	for bars > 30 and ≤ 40 mm
– for stirrups	$d_3 = 4Ø$	for bars ≤ 16 mm

If transverse reinforcement is provided, the mandrel diameters may be reduced in special cases (e.g. frame corners or loop anchorages):

– for end hooks, angle hooks, loops	$d_2 = 4Ø$	for bars ≤ 16 mm
	$d_2 = 7Ø$	for bars > 16 mm ≤ 30 mm
	$d_2 = 10Ø$	for bars > 30 and ≤ 40 mm

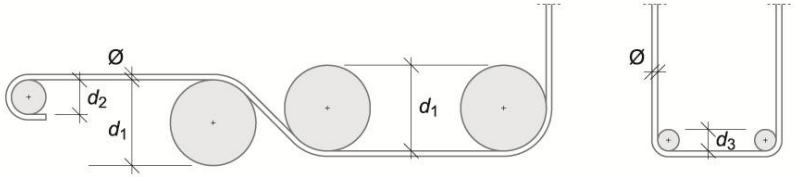


Figure 7.13-1: Mandrel diameters

No welds are permitted in the immediate vicinity of bends.

In the case of welded reinforcement, the minimum distance between the bend and the weld is 10Ø. The distance may be reduced if the weld is not fully loaded.

In the case of bent mesh reinforcement, the cross bars should be located outside the bending zone.

Recommended values for the minimum permissible mandrel diameter for welded reinforcement or welded fabric bent after welding are given in Fig. 7.13-2.

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This subclause gives rules which are simplified with regard to the relations given in subclause 6.1.3.

The more refined rules of subclause 6.1.3 may permit shorter anchorage lengths.

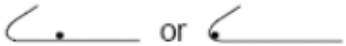
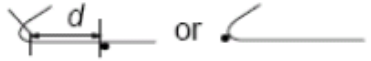
Recommended minimum mandrel	
	
5Ø	d ≥ 3Ø: 5Ø d > 3Ø or welding within the bent zone: 20Ø

Figure 7.13-2: Recommended minimum mandrel diameter for welded reinforcement or welded fabric after welding

Specifying the minimum radii of curvature of prestressing tendons the technical documentation of the prestressing system should be regarded. In addition, it may be necessary to check the local concrete stresses. Reference is made to subclause 5.4.9.2 which gives minimum radii of curvature for post-tensioning tendons.

7.13.2.5 Anchorage

This section covers anchorage of ribbed reinforcing bars and meshes of reinforcing steel which satisfy the requirements for classification as 'high bond reinforcement' in accordance with subclause 5.2.4.2.

This section does not cover bars forming part of a bundle.

The rules in this clause are limited to the following:

a) characteristic yield strength of reinforcement ≤ 500MPa;
b) minimum requirements for confining reinforcement and cover in 6.1.3.1 are satisfied; where the diameter Ø of the anchored bars is smaller than 20mm, transverse reinforcement or links provided for other reasons may be assumed sufficient to satisfy minimum requirements for confining reinforcement without further justification;
c) on flexural members, loading is predominantly uniformly distributed along the span;
d) no applied tensile stress acts transverse to the axis of the anchored bar.

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The contribution of a hook or bend to anchorage of tension reinforcement and the contribution of end bearing to anchorage of compression reinforcement is included in the bond length given by Eq. (7.13-2).

Values for β_b in Table 7.13-2 include a partial safety factor of 1.5 on bond strength

Figure 7.13-3 provides illustrations of representative situations for each type of bond zone.

If any of these conditions is not satisfied the detailed rules of subclause 6.1.3 shall be used.

The dimensioning value of the bond length is:

$$l_b = \alpha_1 \cdot \beta_b \cdot \left(\frac{25}{f_{ck}} \right)^{0.5} Ø / \eta_3 \geq l_{b,\min} \qquad (7.13\text{-}2)$$

where

l_b follows from Eq. (6.1-19);

$\alpha_1 = A_{s,cal} / A_{s,ef}$ for anchorages in zones AS, AB and AC;

where

$A_{s,cal}$ is the calculated area of reinforcement required by the design;

$A_{s,ef}$ is the area of reinforcement provided;

α_1 =1.0 in other circumstances;

η_3 represents the influence of bar diameter, as given in subclause 6.1.3.1;

$\eta_3 = 1.0$ for $Ø \leq 25$ mm;

$\eta_3 = (25/ Ø)^{0.3}$ for $Ø > 25$ mm (Ø in mm);

β_b is a factor given in Table 7.13-2 for anchorage length appropriate to bond zones classified as follows:

Type AS : straight bars in tension in zones near the ends of members and in which the support reaction or load from upper storeys is transferred through the anchorage zone in a direction perpendicular to the plane passing through the axes of the anchored bars.

Type AB : tension bars which terminate in a hook or bend in zones near the ends of members and in which the support reaction or load from upper storeys is transferred through the anchorage zone in a direction perpendicular to the plane(s) passing through the axes of the anchored bars.

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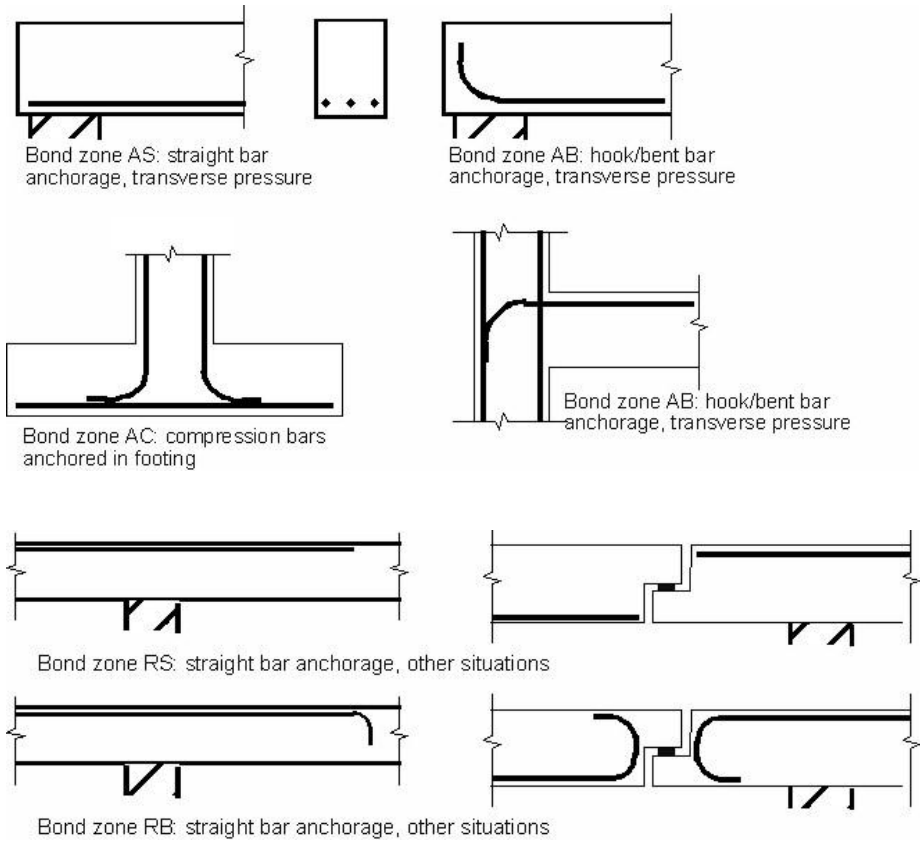


Figure 7.13-3: Classification of anchorage bond zones

Type AC : column and wall bars anchored near the middle of footings or pilecaps and acting in compression under all design loadings.

Type RS : straight bars in all other situations.

Type RB : anchorages of bars terminating in a hook or bend in all other situations.

Table 7.13-2: Bond length factors for anchorages

Bond zone	Bond length factor β_b			
	Casting position			
	'Good'		'Poor'	
	c_{min} / Ø ≥ 2.5,	c_{min} / Ø ≥ 1.0,	c_{min} / Ø ≥ 2.5,	c_{min} / Ø ≥ 1.0,
AS	42	42	59	59
AB	29	29	39	39
AC	29	–	39	–
RS	66	95	95	131
RB	41	74	74	95

Casting position is defined in subclause 6.1.3.2. Minimum cover c_{min} is defined in Figure 6.1-2 and Figure 6.1-15 for straight and bent/hooked bars respectively.

$\beta_{b,min}$ shall be taken as 15 for Type RS and RB bond zones, or 10 for Types AS, AB and AC.

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This clause gives rules which are simplified with regard to the relations given in subclause 6.1.3.

The more refined rules of subclause 6.1.3 may permit shorter lap lengths.

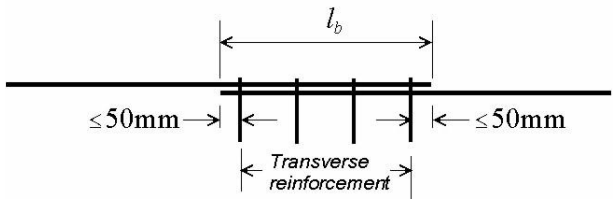


Figure 7.13-4: Location of links near ends of lap

The contribution of end bearing in compression laps and anchorages is included in the average bond strength given by Eq. (7.13-2)

Mesh reinforcement with welded cross bars may be considered to lie within Types AB or RB as appropriate. l_b may be reduced by 15% for each cross bar positioned within the anchorage area, but not by more than 30%.

7.13.2.6 Lapped joints

This section covers lapped joints or splices of ribbed reinforcing bars and meshes of reinforcing steel which satisfy the requirements for classification as 'high bond' in accordance with requirements of subclause 5.2.4.2.

Wherever possible, splices shall be arranged in zones of low stress.

For bar diameters > 12 mm, lap splices shall, if possible, be staggered so that, in a cross-section, not more than one third the reinforcement force needs to be transferred by a lap.

The rules in this clause are limited to the following:

a) characteristic yield strength of reinforcement ≤ 500 MPa;

b) minimum requirements for confining reinforcement and cover in 6.1.3.1 are satisfied; where the diameter ϕ of the anchored bars is smaller than 20 mm, transverse reinforcement or links provided for other reasons may be assumed sufficient to satisfy minimum requirements for confining reinforcement without further justification;

c) at least 1 item of transverse reinforcement should be positioned within a lap length no further than 50 mm from the end of both the lapped bars; other items of transverse reinforcement should be spaced evenly throughout the lap length (Figure 7.13-4);

d) no applied stress transverse to the axis of the lapped bars.

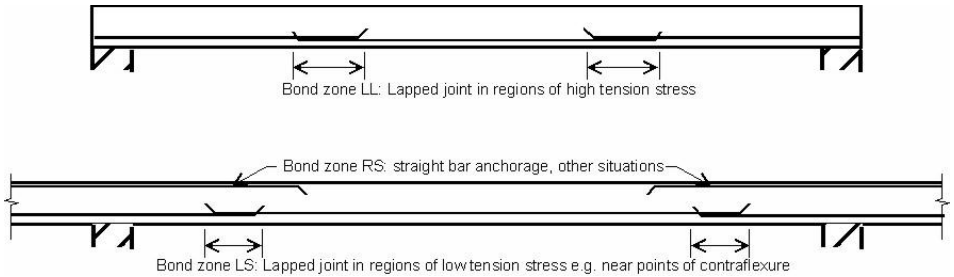
If any of these conditions are not satisfied the detailed rules of subclause 6.1.3 shall be used.

The dimensioning value for the lap length is given by Eq. (7.13-2), where the value of β_b is given in Table 7.13-3.

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Values for β_b in Table 7.13-3 include a partial safety factor of 1.5 on bond strength.

Figure 7.13-5 provides illustrations of representative situations for each type of bond zone.



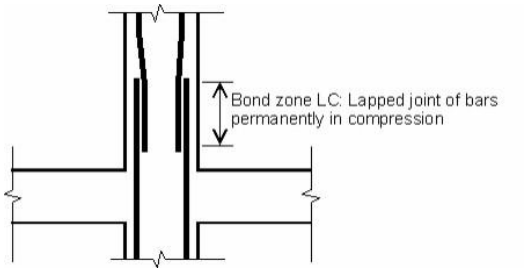


Figure 7.13-5: Classification of lapped splice zones

Bond zones for lapped splices are classified as follows:

Type LS : straight bars in tension where the splice will be subject to a stress not exceeding 50% of the characteristic strength of the reinforcement at the ultimate limit state;

Type LL: straight bars in tension not classified as Type LS;

Type LC: straight bars in compression under all load combinations.

Table 7.13-3: Bond length factors for lapped joints

Bond zone	Bond factor β_b			
	Casting position			
	'Good'		'Poor'	
	Confinement			
	c_{min} / Ø ≥ 2.5	c_{min} / Ø ≥ 1.0	c_{min} / Ø ≥ 2.5	c_{min} / Ø ≥ 1.0
LS	48	66	66	95
LL	66	95	95	131
LC	41	74	74	95

Mesh reinforcement with welded cross bars may be considered to lie within Types LS, LL or LC as appropriate. l_{bd} may be reduced by 15% for each cross bar positioned within the anchorage area, but not by more than 30%.

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7.13.2.7 Deviations and curvatures

In the case of curved or kinked tension or compression chords, the effects of the deviation forces shall be regarded.

Deviation forces acting toward the surface of the concrete, as shown in Figure 7.13-6 shall in general be resisted by means of additional stirrup reinforcement. If this is not provided it shall be verified that the deviation forces at yielding of the steel can be resisted by the cover concrete. In this specific case the concrete tensile strength may be taken into account with a maximum value according to:

$$f_{ctd} = \frac{1}{3}\frac{f_{ctk\,0.05}}{\gamma_c} \tag{7.13-3}$$

The strength according to Eq. (7.13-3) has to be reduced by 50% in case of plastic deformations in the reinforcement.

If the deviation forces have to be resisted by the concrete, the inaccuracies of execution (thinner cover concrete, smaller spacing between reinforcements, non-uniform curvature) shall be taken into account for the verification.

For the calculation of the concrete tensile strength the effective width b_u may be assumed to be equal to:

$$b_u = s - \varnothing \le 2\sqrt{3(c_{nom} + \frac{\varnothing}{2})} \tag{7.13-4}$$

where s = spacing between reinforcing bars and $\varnothing$ = bar diameter.

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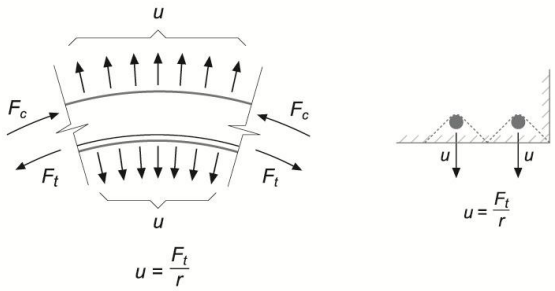


Figure 7.13-6: Equilibrium in curved tension and compression chords (r = radius of bend)

In the case of reinforcing bars which are curved or bent parallel to outer surfaces, the effects of the transverse tensile forces shall be examined.

Transverse tensile forces according to Figure 7.13-7 shall be resisted by means of transverse reinforcement (e.g. stirrup or U-shaped reinforcement). If no transverse reinforcement is provided, it shall be verified that the transverse tensile forces due to the yield tensile force can be resisted. The concrete tensile strength may be taken into account according to Eq. (7.13-3) at most. No verification is necessary if the lateral cover of reinforcement corresponds to at least three times the bar diameter.

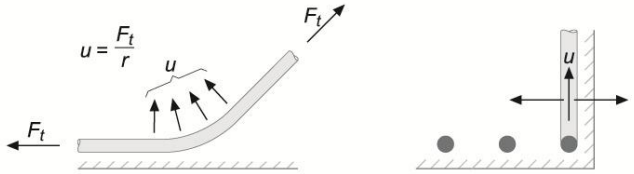


Figure 7.13-7: Transverse tensile forces in the case of bent-up reinforcing bars

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The equations (7.13-5a) and (7.13-5b) follow from Eq. (6.1-40), subclause 6.1.8.4 for a sudden release of the prestressing force, and a position of the tendon in the good bond area of the member.

7.13.3 Prestressed structures

With regard to the transmission of the prestressing force into the concrete for post-tensioned structures, two zones area distinguished:

(1) The area immediately around and behind the contact area, where the prestressing force is transmitted to the concrete: dimensions and reinforcement detailing are specified in Technical Approvals (e.g. ETA) of the prestressing system. The reliability for the corresponding detailing is the responsibility of the system holder.

(2) The zone where the prestressing force is spread over the full cross section. Design and detailing of the reinforcement in this zone is the responsibility of the designer.

Force spreading zones shall be detailed in such a way that the dimensioning value of the prestressing force according to clause 5.3 can be introduced into the structure.

The flow of forces shall be examined using suitable models, neglecting the tensile strength of the concrete. Splitting forces shall be resisted by means of reinforcement.

A minimum reinforcement shall in any case be placed in anchorage and coupling zones to avoid the occurrence of harmful cracks.

Anchorages and couplings shall if possible be positioned in areas in which only small stress variations occur.

7.13.3.1 Anchorage of prestressing wires and strands

A practical expression for the upper bound value of the transmission length l_{bpt} is:

$$l_{bpt,95\%} = \frac{0.10\phi_s\sigma_{pi}}{f_{ctd}} \qquad \text{for strands} \qquad (7.13\text{-}5a)$$

and

$$l_{bpt,95\%} = \frac{0.15\phi_s\sigma_{pi}}{f_{ctd}} \qquad \text{for indented wires} \qquad (7.13\text{-}5b)$$

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The design value of the anchorage length l_{bpd} is:

$$l_{bpd} = l_{bpt,95\%} + \frac{A_p(\sigma_{pd} - \sigma_{pcs})}{\pi \cdot \varnothing_s \cdot f_{bpd}} \quad (7.13\text{-}6)$$

where:

σ_{pd} is the tendon stress under the design load;

σ_{pcs} is the tendon stress after all losses;

σ_{pi} is the initial prestress;

f_{bpd} follows from Eq. (6.1-38).

7.13.4 Bearings and joints

In the case of unreinforced concrete structures, the arrangement of expansion joints may be practical in order to compensate for the effects of shrinkage and temperature changes.

In the case of reinforced concrete structures, expansion joints may have to be arranged if the effects of shrinkage, creep, temperature changes and non-uniform settlements cannot adequately be rendered harmless by the provision of crack distributing reinforcement.

In order to reduce the effects of shrinkage, it shall be examined whether shrinkage joints for the temporary partition of larger structural members or other measures shall be specified.

Bearings of important structural elements shall be permanently protected against moisture. They must be easily accessible and easy to replace.

Expansion joints shall, if possible, be designed in such a way that moisture cannot reach the joint.

7.13.5 Structural members

7.13.5.1 Unreinforced structural members

Unreinforced concrete is used for solid structural members that are primarily subjected to compression and are not subjected to any significant seismic or dynamic loads. Examples include:

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– foundations,
– arches and vaults,
– retaining walls,
– walls,
– compression members.

The dimensioning value of the compressive strength f_{cd} shall be reduced by 20% for verifying the structural safety of unreinforced structural members.

When verifying structural safety, a state of equilibrium between the internal and external forces shall be assumed, neglecting the concrete tensile strength. Kinematic compatibility has to be ensured. Any constrained deformations and displacements have to be taken into account. External forces (reactions) should be taken into account if these act even in case of small deformations (e.g. frictional forces, abutment forces).

For structural members of secondary importance, the structural safety shall be verified taking into account the concrete tensile strength according to

$$f_{ctd} = 0.5 \frac{f_{ctk,0.05}}{\gamma_c} \tag{7.13-7}$$

In addition, it shall be ensured that the formation of cracks does not result in failure of the structural member.

The structural safety of unreinforced compression members may be verified in analogy to subclause 7.3.7. The dimensioning value of the compressive strength shall be reduced as indicated above.

7.13.5.2 Beams and T-beams

A beam is defined as a bearing element, spanning in one direction, with a width smaller than 5 times the cross-sectional depth

Sections containing less reinforcement than $A_{s,min}$ should be considered as unreinforced.

The area of longitudinal reinforcement should not be taken as less than:

$$A_{s,\min} = 0.26 \frac{f_{ctm}}{f_{yk}} b_t d \tag{7.13-8}$$

where b_t is the width of the tension zone.

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– In the application to new structures, the rules of this section should be complied with in full.
– In the application for the assessment of existing structures, the detailing rules presented in this section may be relaxed provided that the assessment on strength in cases of non compliance is assessed fully.

The minimum amount of shear reinforcement (Eq. 7.13-9) is required to ensure that failure does not occur immediately upon shear cracking and truss action can develop.

In beams at least 50% of the shear reinforcement should consist of closed stirrups, because the inclined struts tend to exert a lateral force on the outer longitudinal reinforcing bars (Figure 7.13-8). In slabs those lateral forces can be accounted for by transverse bars.

Adequate reinforcement shall be provided to cope with restraining effects which have been neglected in the structural analysis.

Where shear reinforcement is required (see subclause 7.3.3), the minimum area of shear reinforcement shall be:

$$A_{sw,\min} = 0.08 \sqrt{f_{ck}} \frac{b_w s_w}{f_{yk}} \qquad (7.13\text{-}9)$$

In the case of wide webs, the web width may be taken into account up to a maximum of 400 mm. In the case of web widths > 500 mm, stirrups with more than two legs shall be used.

In beams, stirrups shall generally be provided. Their spacing s_w shall not exceed $0.75d$ or 500 mm.

Transverse reinforcement provided for shear and torsion may consist of a combination of:

– stirrups or ties perpendicular to the axis of the member, enclosing the longitudinal tension reinforcement and the compression zone;
– cages, ladders, welded wire fabric, etc. which are cast-in without enclosing the longitudinal reinforcement but are properly anchored in the compression and tension zones. Such reinforcement may only be applied in combination with stirrups enclosing the longitudinal reinforcement, carrying at least 50% of the total force in the shear reinforcement;
– longitudinal bars bent to provide an inclined portion having an angle of 30° or more with the longitudinal bars and crossing potential diagonal cracks. However, only the centre three-quarters of the inclined portion of these bars may be considered effective;
– headed shear reinforcement demonstrated to be able to achieve the yield strength of the bar;
– spirals.

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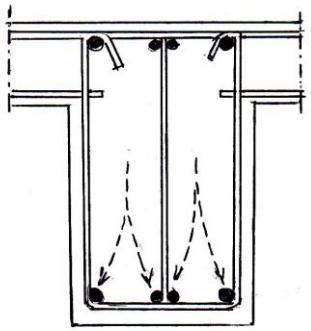


Figure 7.13-8: Lateral forces on outer bars due to strut action

A slab is a bearing element, spanning in one ore two directions. A one way bearing element is considered to be a slab if the width of the cross section is equal or larger than 5 times the overall cross-sectional depth.

- In the application to new structures, the rules of this section should be complied with in full.
- In the application for the assessment of existing structures, the detailing rules presented in this section may be relaxed provided that the bearing resistance in cases of non compliance is fully assessed.

The stirrups shall enclose the longitudinal tensile reinforcement. They shall be anchored in such a way that the necessary stirrup forces can act over the height of the lever arm of the internal forces.

In the support areas, at least 25% of the total chord reinforcement required in the span shall be fully anchored.

In the area of negative moments of T-beams, a significant portion of the tensile reinforcement shall be concentrated over the web. In order to limit crack widths, a certain proportion shall also be distributed over the adjacent slab. The effectiveness of the longitudinal reinforcement distributed outside of the web shall be ensured by means of transverse reinforcement.

Adequate transverse reinforcement, the cross-section of which should amount to at least 0.2% of the slab cross-section, shall be provided in the compression slab to ensure the shear connection.

7.13.5.3 Slabs

In the zones of largest moments, the bar spacing of the main reinforcement shall not exceed the lesser of 1.2 times the slab thickness and 300 mm.

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The minimum reinforcement of slabs shall be specified in accordance with the serviceability requirements. Transverse reinforcement shall not be less than 20% of the longitudinal reinforcement.

For the case of slabs without shear reinforcement, at least one-half of the bending reinforcement required at the points of maximum moment shall be fully anchored beyond the extent of the supports.

Free slab edges shall be reinforced with bent-up longitudinal reinforcement or with stirrup reinforcement of at least ∅10mm, in accordance with Figure 7.13-9.

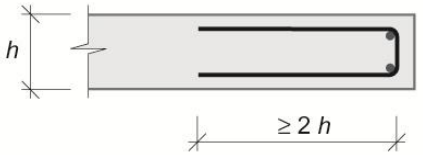


Figure 7.13-9: Free slab edges.

For punching shear, the development length and splices in flexural reinforcement shall not be located inside the perimeter defined by the minimum of

- a line at 2.5 d_v from the control perimeter of the supported region, and
- the position of the line of contraflexure of radial bending moments.

For members with punching shear reinforcement, the design equations of subclause 7.3.5.3 are applicable provided that:

- a minimum of two shear reinforcing elements are provided in the radial direction;
- the geometry and type of shear reinforcement guarantees anchorage at both ends;
- the distance between the first shear reinforcing element and the face of the support is larger than or equal to 0.35d_v and smaller than or equal to 0.75d_v (Figure 7.13-10); any shear reinforcement closer to the support than 0.35d_v shall not be considered for resistance;

The reinforcement details should aim at avoiding localization of cracks (anchorage, splices, etc.) as well as ensure optimum efficiency of transverse reinforcement.

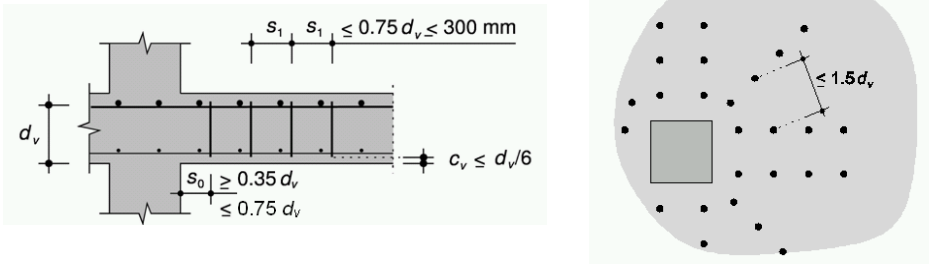


Figure 7.13-10: Detailing rules for punching shear reinforcement

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- the maximum spacing between shear reinforcing elements in the radial direction is not larger than the smaller value of $0.75d_v$ and 300 mm;
- the maximum distance between concentric shear reinforcing elements at the level of the second radial reinforcement does not exceed $1.5d_v$;
- the maximum concrete cover at the compression side of the slab does not exceed $d_v/6$;
- the maximum diameter of the shear reinforcement $Ø_{max}$ does not exceed the values given in Table 7.13-4.

Table 7.13-4: Maximum diameter of punching shear reinforcement as a function of slab effective depth d_v

d_v [mm]	$Ø_{max}$ [mm]
< 160	-
160-180	14
181-220	16
221-260	18
261-340	20
341-600	25
> 600	30

7.13.5.4 Compression members

In the case of storey-high compression members, the minimum dimensions according to Table 7.13-5 shall generally be complied with.

Table 7.13-5: Minimum dimensions of compression members

	Cast in place [mm]	Precast element [mm]
Compression members (side ratio up to 4:1)	200	150
Reinforced walls	150	100
Unreinforced walls	120	–

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In compression members, the minimum longitudinal reinforcement ratio is 0.2%. In the case of large column cross-sections, a wall thickness of at least 200 mm according to Figure 7.13-11 may be taken into account in determining the minimum reinforcement ratio.

The minimum reinforcement shall be distributed proportionally around the circumference of the column cross-section and shall have a diameter of at least 8 mm.

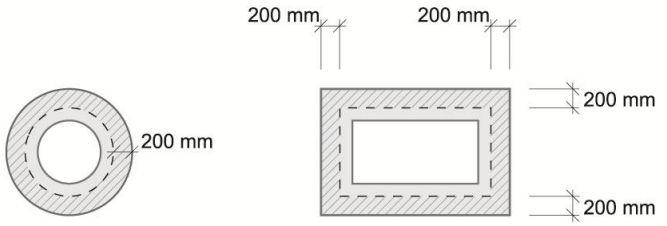


Figure 7.13-11:Minimum wall thickness for determining the minimum reinforcement of compression members

In walls, the minimum required cross-section of the vertical reinforcement is 0.2% of the concrete cross-section required for structural safety. The vertical bar spacing shall neither exceed twice the wall thickness nor 300 mm.

Compression members with a reinforcement ratio below that stipulated above shall be dimensioned like unreinforced structural members according to subclause 7.13.5.1.

The maximum longitudinal reinforcement ratio in compression members shall in general not exceed 0.04 outside lap regions, unless it is shown that the integrity of the concrete is not affected, and the full strength is achieved at the ULS. This limit should be increased to a maximum of 0.08 at laps. In the case of large reinforcement ratios, special detailing and execution measures have to be taken and the stirrup reinforcement shall be strengthened.

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The longitudinal reinforcing bars of compression members shall be prevented from local buckling by means of stirrups. If the reinforcement of the compression zone reaches the yield strength in the ultimate limit state, the corner bars as well as every second longitudinal bar shall be enclosed with hooks or additional stirrups.

The spacing s of the stirrups and hooks shall fulfil the following requirements:

$$\begin{aligned} & s \le 15\phi_{sl,min} \\ & s \le a_{min} \\ & s \le 300\,\text{mm} \end{aligned} \qquad (7.13\text{-}10)$$

where $a_{\min}$ is the minimum cross-sectional dimension and $\phi_{sl,min}$ is the minimum diameter of the longitudinal reinforcing bars.

In compression members with a polygonal cross-section, a longitudinal bar shall be provided at least in every corner.

In the zones where forces are applied, in the area of splices and where the cross-section of compression members changes, additional stirrups shall be provided in order to resist transverse tensile forces. The bar diameter of the stirrups shall be at least 1/3 of the diameter of the thickest longitudinal bar.

The provisions for compression members apply analogously to the stirrup reinforcement in walls.

The horizontal reinforcement of walls shall be dimensioned according to the requirements of clause 7.6 but shall not be less than 25% of the vertical reinforcement.

In the hinge regions of columns constructed with concrete of strength classes of C60 or higher, and without steel fibres, stirrups shall be detailed such that an equivalent minimum core confining pressure of $0.01f_{ck}$ is achieved where the confining pressure is determined in accordance with subclause 7.2.3.1.6. Confining reinforcement shall be continued beyond 1.2 h to each side of sections with maximum moments. Alternatively, 2% by weight of end-hooked fibres may be used to provide sufficient sectional ductility to compression members of high strength concrete.

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7.13.6 Special aspects of precast concrete elements and composite structural members

7.13.6.1 General

In designing precast concrete elements and structures assembled out of these, the final state as well as the construction states during transport and assembly shall be taken into consideration. The development of the concrete strength over time shall also be taken into account.

In the case of precast concrete elements and their connections, the technical requirements relating to production and assembly as well as the static requirements shall be taken into account when specifying the dimensional tolerances. In particular, the effects of deformations due to creep and shrinkage of the concrete as well as temperature changes shall be adequately taken into account.

Panels of facade plates with a multi-layered structure shall, as far as possible, be able to move freely within their plane. The panels shall be connected by means of fatigue-proof and corrosion-resistant fastenings. The connecting elements shall, as far as possible, be:

- chemically and physically compatible;
- protected against chemical and physical influences;
- fireproof in accordance with the structure.

7.13.6.2 Bearings

The nominal length of a simple bearing as shown in Figure 7.13-12 may be calculated as:

$$a = a_1 + a_2 + a_3 + \sqrt{(\Delta a_2^2 + \Delta a_3^2} \qquad (7.13\text{-}11)$$

where:

a_1 is net bearing length with regard to the bearing stress, being:

$a_1 = F_{Ed} /(b_1 f_{Rd})$;

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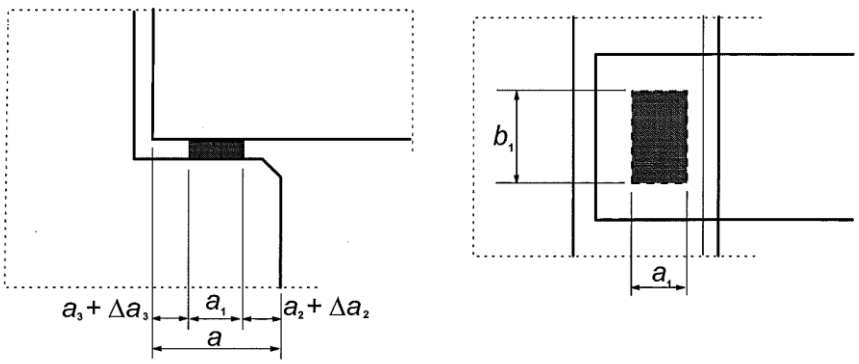


Figure 7.13-12: Definitions of bearing geometry

F_{Ed} is design value of support reaction;

b_1 is net bearing width;

f_{Rd} is design value of bearing strength, with:

$f_{Rd} = 0.4 f_{cd}$ for dry connections;

$f_{Rd} = f_{bed} \leq 0.85 f_{cd}$;

where:

f_{cd} is lowest design strength between supporting and supported member;

f_{bed} is design strength of bedding material;

a_2 is distance between edge of bearing and end of supporting member (Figure 7.13-12);

a_3 is distance between most remote edge of bearing and end of supported member (Figure 7.13-12);

Δa_2 is allowance for tolerances for distance between supporting members (Fig. 7.13-12);

Δa_3 is allowance for tolerances for the length of the supporter member, with: $\Delta a_3 = l_n / 2500$, where l_n = length of supported member (Fig. 7.13-12).

Values for a_1 a_2 a_3 and Δa_2 are given in Tables 7.13-6 to 7.13-9.

Table 7.13-6: Minimum value of a_1 (Figure 7.13-12), in mm

Relative bearing stress σ_{Ed} / f_{cd}	≤ 0.15	0.15 – 0.4	> 0.4
Line support (floors, roofs)	25	30	40
Ribbed floors and purlins	55	70	80
Concentrated supports	90	110	140

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Table 7.13-7: Minimum distance a_2 (Figure 7.13-12), in mm

Support material	σ_{Ed} / f_{cd}	≤ 0.15	0.15–0.4	> 0.4
Steel	line	0	0	10
	concentrated	5	10	15
Reinforced	line	5	10	15
Concrete ≥C30	conc.	10	15	20
Plain concrete	line	10	15	15
Reinf. Concrete <C30	conc.	20	25	25
Brickwork	line	10	15	–
	conc.	20	25	–

Table 7.13-8: Minimum distance a_3 (Figure 7.13-11), in mm

Detailing of reinforcement	**Support type**	
	Line	Concentrated
Continuous bars over support (restrained or not)	0	0
Straight bars, horizontal, close to end of member	5	15, but not less than end cover
Tendons or straight bars exposed at end of member		15
Vertical loop reinforcement	15	End cover + inner radius of bend

Table 7.13-9: Allowance Δa_2 (Figure 7.13-11), in mm

Support material	Δa_2
Steel or precast concrete	$10 \le l/1200 \le 30$ mm
Brickwork or cast in situ	$15 \le l/1200 + 5 \le 40$ mm

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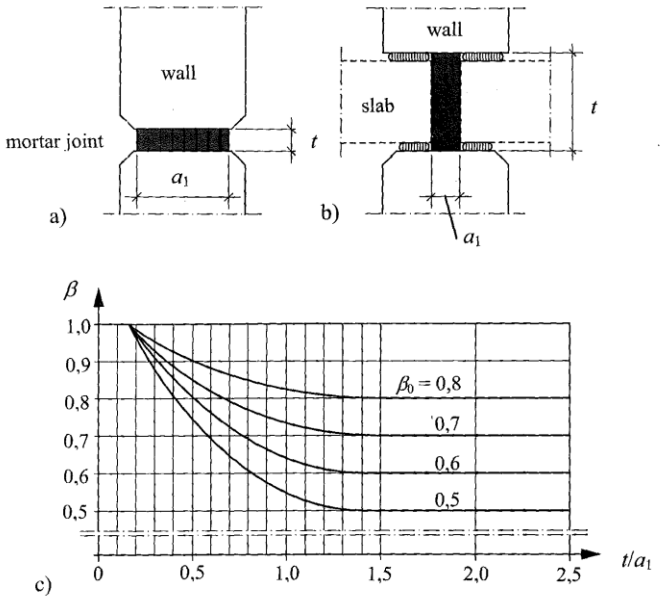


Figure 7.13-13: Capacity of mortar joint (fib Bulletin 43)

7.13.6.3 Mortar joints

Joints filled with mortar are used as a connection between precast members transmitting compression forces. The joint bearing capacity is:

$$N_{Rd,j} = f_{cd,j} \cdot A_j = \beta \cdot f_{cd,w} \cdot a_1 . l \qquad (7.13\text{-}12)$$

where:

$f_{cd,w}$ is design compressive strength of wall concrete;

$f_{cd,m}$ is design compressive strength of joint mortar;

$f_{cd,j}$ is design compressive strength of joint;

$\beta_0 = f_{cd,m} / f_{cd,w}$;

$\beta = f_{cd,j} / f_{cd,w}$.

The values β can be read in Figure 7.13-13.

The strength of the mortar should be at least 50% of the strength of the adjacent members.

7.13.6.4 Loop connections

Loop connections as exemplified in Figure 7.3-14 can be used to transfer tensile forces, bending moments and shear forces. Loop connections are used between solid slabs where continuity is demanded.

The radius of the loop should satisfy the following demands:

$$r \geq \frac{\pi \cdot \phi}{4} \frac{f_{yd}}{\sigma_{c,rad}} \qquad (7.13\text{-}13)$$

$R \geq 8\phi$

where:

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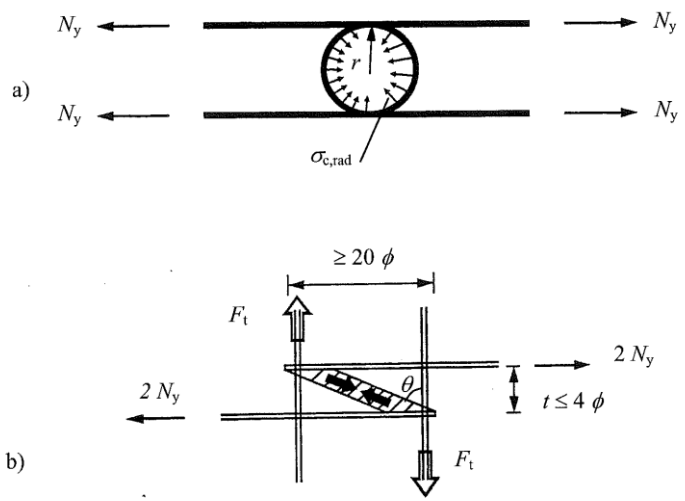


Figure 7.13-14: Loop connection (N_y = yield force in longitudinal bar, F_t = force in transverse bar)

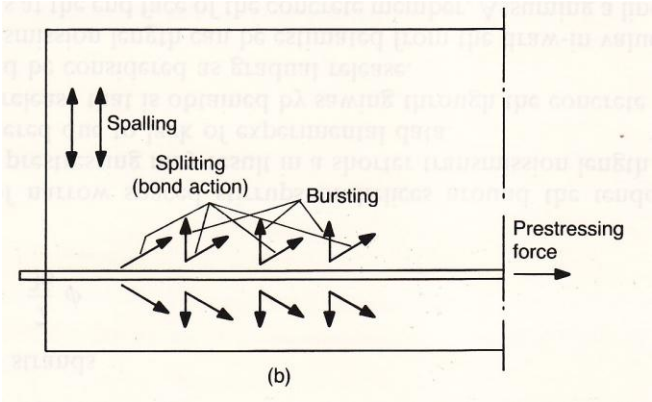


Figure 7.13-15: Bursting, spalling and splitting in the anchorage zone of a member, prestressed with steel with direct bond.

$$\sigma_{c,rad} \leq f_{cd}\sqrt{b_i / \phi} \qquad (7.13\text{-}14a)$$

$$\sigma_{c,rad} \leq 3 f_{cd} \qquad (7.13\text{-}14b)$$

in which:

$b_i = 2 \cdot (c_e + \frac{\phi}{2})$ but not smaller than t (see Figure 7.13-14);

$\sigma_{c,rad}$ = radial compressive strength at inside bend of loop (Fig. 7.13-14);

c_e = concrete cover between U-bar and edge of element;

ϕ = diameter of reinforcing bar.

7.13.6.5 Transverse stresses in the anchorage zone of prestressed tendons

7.13.6.5.1 General

In case of prestressing with pretensioned tendons or wires, three potential ways of cracking can be distinguished:

- bursting can occur due to tensile stresses which are generated as a result of spreading of the prestressing forces over the cross section;
- spalling can occur at the end of the member, especially in the case of thin webs;
- splitting can occur along the transmission length as a result of the effect of internal pressure exerted by the prestressing steel during shortening (wedging effect).

Figure 7.13-15 shows the various mechanisms in the anchorage zone of a prestressed member.

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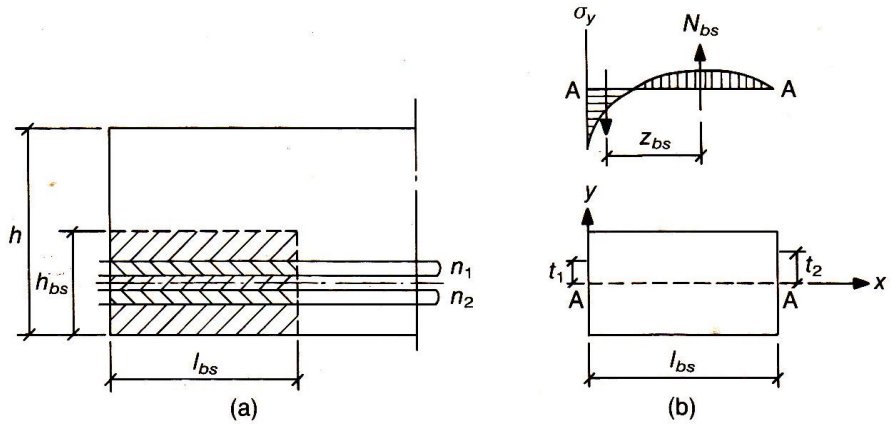


Figure 7.13-16: Calculation of the bursting force: (a) dimensions of the symmetrical prism; (b) moment equilibrium along section A-A

7.13.6.5.2 Bursting

For the calculation of the bursting force the symmetric prism analogy may be used (Figure 7.13-16). The calculation is based on a virtual prismatic element, defined in order to describe the bursting forces. The prism is shown in Figure 7.13-16(a) (shaded area). The length of the prism is:

$$l_{bs} = \sqrt{h_{bs}^2 + (0.6 l_{bpt})^2} < l_{bpt} \qquad (7.13\text{-}15)$$

where l_{bpt} is the transmission length, where now the 5% value is used: this is equal to 50% of the values given in Eq. (7.13-5a) or (7.13-5b).

The internal lever arm for the bursting force is:

$$z_{bs} = 0.5 l_{bs}$$

The bursting force N_{bs} follows from the moment equilibrium along section A-A (Figure 7.13-16):

$$N_{bs} = \frac{1/2(n_1 + n_2)t_2 - n_1 t_1}{z_{bs}} \gamma_1 F_{sd} \qquad (7.13\text{-}16)$$

where:

t_1 is distance between the centroid of tendons above section A-A to the centroid of the prism;

t_2 is distance between the centroid of the concrete stress block above section A-A to the centroid of the prism;

n_1, n_2 are numbers of the tendons above and below section A-A, respectively;

F_{sd} is design prestressing force per tendon;

γ_1 = 1.1, being the supplementary safety factor against overstressing.

The maximum bursting stress follows from:

$$\sigma_{bs} = N_{bs} / (b_{bs} l_{bs}) \qquad (7.13\text{-}17)$$

where b_{bs} = width of the prism.

If $\sigma_{bs} < f_{ctd}$ no bursting reinforcement is required.

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7.13.6.5.3 Spalling

The spalling stresses can be read from Figure 7.13-10, based on elastic analysis. If $\sigma_{spl} < f_{ctd}$ no spalling reinforcement is necessary.

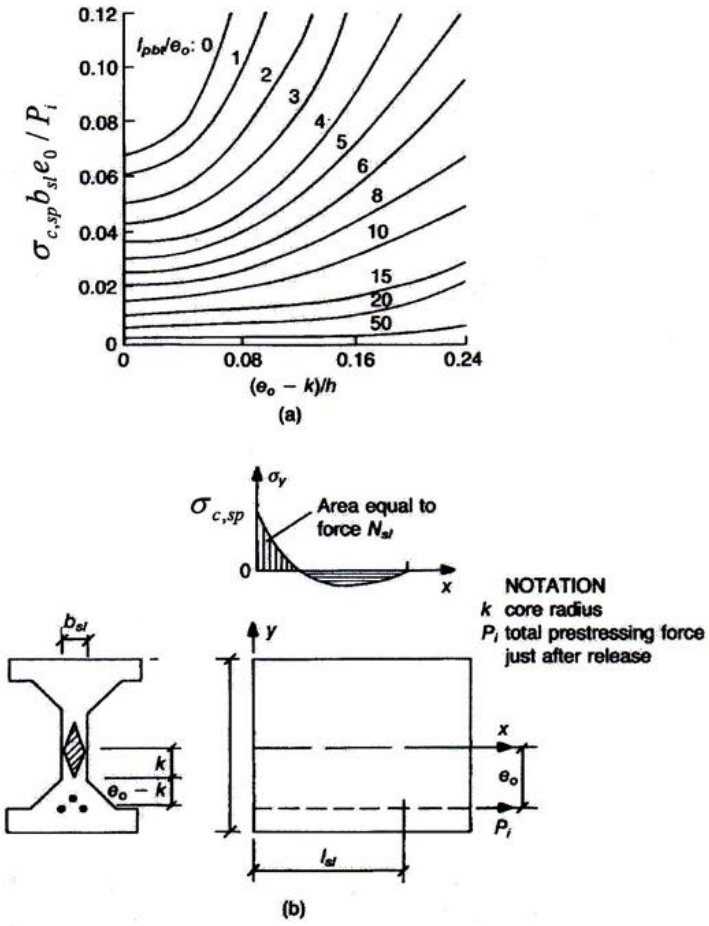


Figure 7.13-17: Maximum spalling stress as a function of eccentricity and transmission length (based on linear elastic analysis) for members with h < 400 mm

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7.13.6.5.4 Splitting

No reinforcement against splitting forces is necessary if the distance between the strands/wires (with diameter ∅) and the cover satisfies the minimum values given in Table 7.13-10:

Table 7.13-10: Minimum cover as a function of the clear spacing to resist the splitting stresses around pretensioned strands

Concrete strength	**Clear spacing**	**Cover**
C20/25 to C50/60	≥ 3 ∅ = 2.5 ∅	≥ 3 ∅ ≥ 4 ∅
≥ C55/67	≥ 2.5 ∅ = 2 ∅	≥ 2.5 ∅ ≥ 3 ∅

For creating an envelope of allowable covers, interpolation between the clear spacings 2.5∅-3.0∅ and 2.0∅-2.5∅ is possible

7.14 Verification of anchorages in concrete

The local and structural effects of anchorages should be considered in design. The design of anchorages in concrete should be performed according to the *fib* Bulletin 58 "Design of anchorages in concrete. Guide to good practice" (*fib*, 2011). Only anchors that have been prequalified for the intended use should be used.

The *fib* Bulletin 58 "Design of anchorages in concrete. Guide to good practice" (*fib*, 2011) covers the design of anchorages using the following types of anchors:

- cast-in anchors, such as headed anchors and anchor channels;
- post-installed anchors, such as torque-controlled and deformation controlled expansion anchors, undercut anchors, screw anchors, bonded anchors and torque-controlled bonded expansion anchors;
- post-installed deformed reinforcing bars.

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The suitability of an anchor for the intended use should be demonstrated by a standard or a technical approval or by the results of suitable prequalification tests.

The following design situations are covered:

- verification of ultimate limit state for predominantly static loading;
- verification of ultimate limit state for non-static loading (fatigue, seismic);
- verification of ultimate limit state for extreme thermal conditions (fire);
- verification of serviceability;
- verification of limit states associated with durability.

The *fib* Bulletin 58 “Design of anchorages in concrete. Guide to good practice” (*fib*, 2011) includes the local transmission of loads into the structure by anchorages without or with special anchor reinforcement. In the design of the structure the anchor loads and additional design requirements given in Section 8 of the *fib* Bulletin 58 “Design of anchorages in concrete. Guide to good practice” (*fib*, 2011) should be taken into account.

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8 Construction

8.1 General

The execution of concrete works at the construction site shall comply with the requirements given in ISO 22966 or a technical specification that fulfils the minimum requirements of this standard.

In the following subclauses, key-elements from ISO 22966 are given together with some amendments.

The "execution specification" is the total sum of documents required for the execution of the works as provided by the designer to the constructor.

It includes ISO 22966 and the "project specification" prepared to supplement and qualify the requirements of ISO 22966 for a particular project, as well as referring to the national provisions relevant in the place of use.

Annex A of ISO 22966 lists issues in this standard where information for inclusion in the project specification, where relevant, is needed.

8.2 Execution management

8.2.1 Assumptions

These assumptions correspond with the assumptions of ISO 22966 .

The following assumptions apply for this chapter:

This Model Code assumes that there is a site management which will take charge of the organization of the works and enable the correct and safe use of the equipment and machinery, the required quality of materials, the execution of a conforming structure and its safe use up to the delivery of the works.

This Model Code presupposes that the work is carried out with the necessary skill and adequate equipment and resources to perform the work in accordance with ISO 22966 and the requirements of the execution specification.

Health, safety and environmental aspects of construction are not within the scope of this Model Code.

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"As-built" documentation of the direct input parameters to the service life design models might confirm the design assumptions and possibly give the basis for corrective measures. It might also serve as a basis for the condition assessments of the structure during its service life. Such extracts of the "as-built" documentation is sometimes named the structure's "Birth Certificate".

Supervision and inspection are parts of the quality management.

The three execution classes give the option to specify the required level of quality management based on the importance of the component/structure and the criticality of the execution for its ability to fulfil its function.

Execution class 1 should only be used for structures where consequences in case of failure are small or negligible.

8.2.2 Documentation

Before commencement of execution of any part of the works, the execution specification relevant to that part of the works shall be complete and available.

The following items shall be included in the execution specification:

- a reference to ISO 22966 and, if published, its National Annex;
- a reference to other relevant international standards and national technical approvals;
- a reference to relevant national regulations and standards;
- a project specification giving information and requirements for the particular project prepared to supplement and qualify the requirements of the above listed documents;
- drawings and other technical documents needed for the execution.

In execution class 2 and 3 (see subclause 8.2.3), an inspection report for the materials and products brought to the construction site as well of the completed works is required.

8.2.3 Quality management

Quality management forms an integrated part of the reliability differentiation for the project, see subclause 3.3.3.

Quality management aims to eliminate failures due to gross errors, and to ensure the resistance assumed in the design.

Requirements for quality management are specified using one of the following 3 classes, for which the required strictness increases from class 1 to class 3:

- execution class 1;
- execution class 2;
- execution class 3.

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The execution classes comprise requirements for inspection and, dependent on the relevant national annex or the execution specification, requirements for quality planning focusing on organizational measures and allocation of resources and personnel.

The three execution classes are connected to the three levels of reliability differentiation given in ISO 2394:1998, subclause 4.2.3.

The extent of inspection to be applied shall be according to national regulations and shall be stated in the execution specification by the selection of the appropriate execution class.

The types of inspection are normally differentiated in three categories:

- self-inspection by the operator;
- inspection in accordance with the procedures of the constructor;
- possible inspection by another company, i.e. an independent inspection.

If the inspection of the completed works reveals that the original design assumptions are not met, a partial or full redesign may be needed to get an overview of the possible consequences for the future performance of the structure. The new calculation shall be supplemented with data from the inspection.

An example for such a situation may be if the inspection document mentions a too small concrete cover of the reinforcement. The actual cover should then be implemented in a recalculation of the service life and form the basis for conclusions regarding possible rectifications.

Updates from the inspection should also result in updates of the "maintenance plan" and the "inspection plan" for the structure, if relevant.

Prefabrications include cut reinforcement, cut and bent reinforcement and assemblies of these elements using ties, mechanical splices or welds manufactured off-site by a reinforcement fabricator and delivered to site or manufactured on the construction site.

The execution class may refer to the complete structure, to components of the structure or to certain materials/technologies used for the execution.

The execution class being used shall be stated in the execution specification.

The needed inspection to perform a conformity evaluation of the completed work shall be carried out and the results documented.

Where inspection reveals a non-conformity, appropriate action shall be taken to ensure that the structure remains able to perform as designed.

The following aspects shall be investigated in the listed order:

- the implications of the non-conformity on further execution and fitness for intended design purpose;
- the measures necessary to make the component acceptable;
- the necessity of rejection and replacement of the non-repairable component.

8.3 Reinforcing steel works

The following subclauses apply to both prefabrications and site fabricated reinforcement.

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Light surface rust, of a normal atmospheric nature, should be acceptable. However, if reinforcing steel is stored in an environment in which salt is present in the atmosphere, e.g. in a coastal zone, then rust formed may be considered harmful.

Passivation of the zinc coating can be achieved by storing the products outdoors for a sufficient period of time (4 weeks is thought to be a sufficient time) or by application of an appropriate passivation treatment.

Mechanical cutting is an appropriate method.

8.3.1 Transportation and storage

Reinforcing steel bars, coils, welded fabric and prefabrications shall not be damaged (i.e. by mechanical damage and/or corrosion) during transportation and storage.

The surface condition of the reinforcing steel shall be examined prior to use on site to ensure that it is free from mechanical damage (e.g. notches) and from loose rust and deleterious substances which may adversely affect the steel, concrete or the bond between them.

When galvanized reinforcement is used, either the zinc coating shall be sufficiently passive to avoid chemical reactions with the cement or the concrete shall be made with cement that has no detrimental effect on the bond to the galvanized reinforcement.

8.3.2 Identification

The reinforcing steel delivered on site shall be identified to steel grade and manufacturer using either the rolling marks on the steel when available or the accompanying documentation in order to check that it conforms to the appropriate standard referred to in the order.

Anchorage devices and couplers shall be identified using the marks on the devices or on its packaging/documentation in order to check that they conform to the appropriate standard or approval referred to in the order.

If means of identification has been lost, acceptance tests on samples may be required.

8.3.3 Cutting and bending

Cutting and bending of reinforcing steel shall be made in accordance with accepted standard practice and shall conform to the execution specification.

Bending shall be carried out by mechanical methods using equipment designed for the purpose and in one operation at a uniform rate with the aid of mandrels. The diameter of the mandrel used shall be suitable for the actual type of reinforcement and shall not be less than values that could induce bending cracks in the reinforcement or failure of the concrete inside the bend of the reinforcement.

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Recommended values for the minimum permissible mandrel diameter for bending reinforcing steel are given in Table 8.3-1.

Table 8.3-1: Recommended minimum mandrel diameter for bending bars

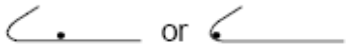
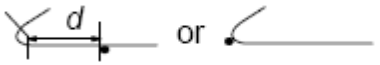
Bar diameter (mm)	**Recommended minimum mandrel diameter for bends, hooks and loops**
$\varnothing \leq$ 16 mm	4$\varnothing$
16 mm < $\varnothing \leq$ 32 mm	7$\varnothing$
32 mm < $\varnothing \leq$ 40 mm	10$\varnothing$

The recommended minimum mandrel diameters for bends, hooks and loops given in Table 8.3-1 are minimum values for reinforcing bars in accordance with product standards, which specify mandrels for bending tests which are 3$\varnothing$, 6$\varnothing$ and 7$\varnothing$, respectively. Larger minimum mandrel diameters in the order of 10$\varnothing$ to 15$\varnothing$ should be used if re-bending or straightening of bars is intended.

For the sake of good detailing, larger minimum mandrel diameters may be required, see also subclause 7.13.2.4

Recommended values for the minimum permissible mandrel diameter for welded reinforcement or welded fabric bent after welding are given in Table 8.3-2.

Table 8.3-2: Recommended minimum mandrel diameter for welded reinforcement or welded fabric bent after welding

Recommended minimum mandrel	
	
5$\varnothing$	5$\varnothing$ if $d \geq 3\varnothing$ 20$\varnothing$ if $d < 3\varnothing$ or welding within the bent zone

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Bending of steel reinforcement at temperatures lower than –5°C should only be permitted, if allowed by the execution specification and provided that the bending procedure conforms to accepted additional precautionary procedures.

Unless otherwise agreed, bending by heating shall not be permitted.

When permitted by the execution specification, straightening of bent reinforcement shall be allowed only if:

- special equipment to limit local stresses is used;
- appropriate straightening procedures are available;
- visual inspection of the straightened reinforcement is carried out for any cracks and other damage.

Some countries require that the decoiled and straightened reinforcement shall have the same properties as specified in the product standard for straight reinforcement.

Reinforcement produced from coils shall only be acceptable if an appropriate decoiling and straightening equipment is used and appropriate procedures in accordance with the equipment manufacturer's instructions are applied.

The decoiled and straightened reinforcement shall meet the requirements of the relevant specification.

8.3.4 Welding

Suitability for welding is defined in the appropriate product standards and is normally defined by limiting the carbon equivalent value of the steel (CEV).

Table 8.3-3 gives examples of appropriate welding processes and applications.

Welding shall only be carried out on reinforcing steel that is suitable for welding and by the use of approved equipment, personnel and approved welding procedures.

Welding of reinforcing steel and welding of reinforcing steel to structural steel in load bearing joints shall be performed according to appropriate procedures and as specified in the execution specification.

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Table 8.3-3: Examples of appropriate welding processes and applications

Loading case	**Welding method**	**Bars in tension[1]**	**Bars in compression[1]**
Predominantly static	flash-welding	butt joint	
	manual metal arc welding and metal arc welding with filling electrode	butt joint with ∅ ≥ 20 mm, splice, lap, cruciform joints [3], joint with other steel members	
	metal arc active welding	splice, lap, cruciform joints [3], joint with other steel members	
		–	butt joint with ∅ ≥ 20 mm
	friction welding	butt joint, joint with other steels	
	resistance spot welding	lap joint[4], cruciform joint [2, 4]	
Not predominantly static	flash-welding	butt joint	
	manual metal arc welding		butt joint with ∅ ≥ 14 mm
	metal arc active welding		butt joint with ∅ ≥ 14 mm
	resistance spot welding	lap joint[4], cruciform joint [2, 4]	

Notes
1. Only bars with approximately the same nominal diameter may be welded together.
2. Permitted ratio of mixed diameter bars ≥ 0.57.
3. For load bearing joints ∅ ≤ 16 mm.
4. For load bearing joints ∅ ≤ 28 mm.

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ISO 17660-1, “Welding – Welding of reinforcement steel – Part 1: Load-bearing welded joints” and “Part 2: Non-load-bearing welded joints” give requirements for welding.

It is assumed that the execution specification gives detailed information on the layout and spacing of reinforcement as well as any precautions to be taken in areas of congested reinforcement.

Acceptable installation tolerances are given in ISO 22966, clause 10. See also subclause 8.6.5 of this Model Code.

In general, reinforcing steel should not be welded at or near bends of the bars.

Spot welding of non-load-bearing welds performed according to an appropriate procedure should be permitted unless otherwise specified.

Spot welding of non-load-bearing joints for assembly of reinforcement using reinforcing steel with established suitability may be permitted without production weld testing unless otherwise specified.

Where a risk of fatigue exists for the concrete structure, welding of reinforcement should conform to special requirements.

The production and control of the welded connections should comply with the relevant requirements.

8.3.5 Joints

Bars shall be jointed by laps, couplers or welding in accordance with the execution specification.

Overlapping bars should be placed in contact; in beams and columns the laps should be tied. If points of overlapping are not shown on the drawings, they should be evenly distributed and the longitudinal distance between two adjacent laps should not be less than 0.3 times the lap length.

Joints made with mechanical connecting devices shall be carried out according to the manufacturer's instructions.

8.3.6 Assembly and placing of the reinforcement

The reinforcement shall be placed according to the execution specification.

The reinforcement shall be fixed and secured so that its final position after placing and concreting lies within the tolerances given in the execution standard or specification.

The assembly of reinforcement may be done with tie wire or spot welding (see subclause 8.3.4).

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Concrete and cementitious spacers should have at least the same strength and should give at least the same corrosion protection as the concrete in the structure. Steel spacers should ensure that the correct concrete cover is achieved.

The specified cover to the reinforcement shall be maintained by the use of suitable chairs and spacers.

8.3.7 Construction documents – reinforcement

Reinforcing steel works shall be defined with details and required quality of works in a execution specification containing as a minimum: (i) construction drawings (shop drawings) giving all necessary information such as geometry of the structure and amount and position of reinforcing steel; (ii) description of all products to be used with any requirement for the application of the products; (iii) works description (method statements) including inspection requirements.

Actual construction works shall be documented with sufficient detail and in sufficient number of construction records/certificates as specified in the construction specification.

8.4 Prestressing works

8.4.1 General

It is recommended that the prestressing works should be done by a specialized prestressing company.

To ensure safety, reliability and durability of tendons it is necessary to provide:

- skilled supervisors and personnel, trained for the purpose and the prestressing system to be used;
- robust and safe equipment (regular maintenance and calibration) compatible with the prestressing system to be used.

High amounts of energy are stored in stressed tendons. Sudden unexpected release of the tendons therefore, is dangerous.

Suitable safety precautions shall be taken on site during all prestressing works.

The engineer shall provide detailed execution specifications to sufficiently specify the requirements for prestressing works to be done on site including at least the following:

- specifications for the tensile elements and the prestressing system to be used;
- drawings to specify the tendon geometry;

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- minimum concrete strength for stage or final stressing of tendons;
- prestressing forces for each tendon and each tendon end, and sequence of stressing of tendons;
- permissible tolerances for prestressing force and tendon elongations;
- requirement for any hold point during prestressing works needing approval of the engineer before continuation of works;
- requirements for any special testing to be performed (e.g. measurement of transmission, i.e. friction losses in tendon, or grouting tests);
- requirements for traceability of materials.

The company responsible for the installation of the prestressing system shall provide information to site to define the specific methods for the prestressing works to be done including at least the following:

- site requirements for storage and handling of materials and equipment;
- shop drawings showing tendon layout (support heights), anchorage assembly, and grouting details (vent locations);
- method statements for tendon installation, tensioning, and grouting, adapted to the complexity of the site;
- theoretical tendon elongations and permissible tolerance for each tendon at specified stressing force;
- required quality control and reporting with optional hold points as specified by the engineer.

8.4.2 Packaging, transportation, storage and handling of materials and components

Materials and components shall be protected from contact with any harmful substances.

Materials and components shall be packed, transported, stored and handled such that they are not damaged and do not deteriorate (e.g. rust). Any damage or deterioration which is considered unacceptable shall be repaired or the materials or components shall be rejected.

Materials and components shall be packed such that they are protected from excessive moisture and/or dirt. Temporary corrosion protection may need to be provided on materials and components where required for the duration of storage and/or environmental conditions (e.g. humidity).

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Certificates of materials received on site shall be compared against the execution specifications. Materials not complying shall be rejected or placed in quarantine.

Materials shall be used before their shelf life expires.

8.4.3 Prestressing works for post-tensioning tendons

8.4.3.1 Installation of tendons

Installation of the tendons and components shall be in accordance with the shop drawings and method statements relevant for the project.

The placing of anchorages, coupling devices and ducts shall be according to the specified geometry and tolerances. Care shall be taken to avoid kinks in the tendon profile and damage to the components. Anchorages, coupling devices and ducts shall be sealed against ingress of water and other foreign substances into the tendons during construction.

Tensile elements shall be installed in the ducts as specified in the relevant procedures in the workshop (prefabricated tendons) or on site either before or after placing of the concrete.

Tensile elements shall be protected from welding sparks.

The tensile elements shall obtain temporary corrosion protection before installation if the expected time between installation/stressing and grouting of the duct exceeds acceptable time limits. Alternatively, the installed tensile elements may be protected by blowing of dry air or other equivalent methods. Temporary corrosion protection materials applied to the tendon shall allow sufficient bond after grouting and shall have no detrimental effect on the tensile elements or on the grouting material. Only products approved by the supplier of the prestressing system shall be used.

Tendons, ducts and vents shall be suitably marked for identification and recording during stressing and grouting operations.

Tendon components shall be inspected before placing of the concrete for compliance with the relevant project and system documents.

Tendon installation tolerances are provided in national or international standards; see e.g. EN 13670, ISO 22966 or ACI 318.

No particular temporary corrosion protection is required if tendons made of prestressing steel are provided with permanent corrosion protection within 2 and 6 weeks after stressing for aggressive and benign environments, respectively, or if the prestressing steel is supplied with a factory-applied corrosion protection.

If the above periods cannot be satisfied, temporary corrosion protection with a suitable water soluble oil may be provided. Only oils shall be used which do not need to be removed before providing permanent corrosion protection (Reference: *fib* Bulletin 20, FIP Recommendations for Corrosion Protection of Prestressing Steels). Tendons should not be flushed with water.

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Refer to the FIP State of the Art Report: 'Tensioning of tendons: force-elongation relationship', Thomas Telford, London, 1986.

In general, the tendon force is determined with the pressure in the stressing equipment. In exceptional cases, direct force measurement with load cells or similar may be done.

The calibration certificates shall not be older than 6 months, in general.

Typically post-tensioning tendons are stressed in the following stages:

- stressing from 0 to 100% of the specified stressing force in one stage for tendons with little tendon deviation and little tendon slack, and recording of the tendon elongations only at the 100% mark. Such tendons are typically found in building slabs as single strand tendons or tendons in flat ducts;
- stressing in stages of 25, 50, 75 and 100% of the specified stressing force for tendons with considerable tendon deviation and tendon slack, and recording of the tendon elongations at each stage. With this procedure, the effect of tendon slack may be deleted from the full force-elongation graph. Such tendons are typically found in bridge construction as multistrand/wire tendons in round ducts.

Traceability of tendon components shall comply with the execution specification.

8.4.3.2 Tensioning operations

General

Stressing of tendons shall be in accordance with the project specific method statements.

Activities before stressing

- check concrete strength to have reached or exceeded the specified minimum strength for stressing;
- check that structure is free to move as may be needed during stressing;
- inspect exposed tendon ends and surfaces on which anchorages and stressing equipment will bear;
- check calibration certificates of stressing jacks to be recent.

Activities during stressing

- increase tendon force gradually and steadily in the stages indicated in the method statement;
- record tendon force (pressure) and tendon elongation at the stages specified in the method statement.

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If the actual tendon force and/or elongations are below acceptable values, the following measures may be considered:

- if the available prestressing steels and prestressing system allow, to apply a higher stress at the stressing end(s) of the tendons. This stress should never exceed the value of 0.95 $f_{p0.1k}$;
- lubricate prestressing steel of tendons to reduce the friction losses;
- install and stress additional internal tendons into provided spare anchorages and ducts or additional external tendons into provided spare anchorages and deviators as may have been specified in the execution specification.

Elongation measurements are only a rough indication for the friction losses during stressing. If these friction losses are considered critical, the measurement of the force transmission with lift-off tests is recommended.

Suggested values for permissible deviations from expected elongations are:

- for tendons of 15 m length or less: 15% for a particular tendon, but not more than 7% for the sum of all values of tendons in the same section;
- for tendons of more than 15 m length: 10% for a particular tendon, but not more than 5% for the sum of all values of tendons in the same section.

Tendon elongations should be calculated with the properties of the prestressing steel as given in the material certificates.

Where tendon tails are burned off, the anchorage devices should be protected from excessive heat development.

- compare specified force and elongation with measured and calculated values;
- take corrective actions if deviation between specified and measured values is greater than permitted tolerances;
- maintain tendon tails to allow re-stressing; cut-off of tendon tails and grouting only after final approval.

Activities after stressing

- visual inspection of concrete and anchorages;
- supervision of temporary protection of tendons;
- cutting of tendon tails once the stressing records have been approved.

Records

- pressure in stressing jack;
- elongation of tendons with adjustment for seating in tendon anchorages, as applicable, and for effects of measuring procedure (e.g. elongation inside stressing jack, if applicable);

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- any particular observations during tensioning;
- site stressing records to be transmitted to and approved by the design engineer in view of any necessary adaptation of above instructions.

8.4.3.3 Grouting of prestressing ducts

General

Grouting of prestressing ducts shall be in accordance with the project specific method statements. Grouting materials shall be in accordance with subclause 5.4.2.3.

For additional information reference is made to the *fib* Bulletin 20, Guide to Good Practice 'Grouting of tendons in prestressed concrete', 2000, and EN standards 445-447(2007).

Activities before grouting

- equipment shall be confirmed to be capable of adequately mixing and pumping the specified grouting material and shall be operational;
- permanent supplies of water under pressure and of compressed air;
- grouting material to be confirmed for use under conditions expected on site (temperature, time required for grouting operations, tendon profile) with suitability tests or similar;

In hot or cold climates, only grouting materials shall be used which have been confirmed to provide the specified performance and to remain sufficiently fluid over the expected period of time at the expected range of temperatures.

If measuring equipment (temperature gauges) is available, the 5°C value may be referred to the temperature of the structure in the vicinity of the tendon.

- instructions for actions in case of incidents (e.g. fault during injection) and harmful climatic conditions (e.g. after and during periods with temperature lower than 5°C);
- materials batched (excess to allow for overflow);
- blowing of air through the ducts, or use other approved methods, to ensure that ducts are free of harmful material (e.g. water, ice);
- vents prepared and marked with identification;
- preparation of moulds and other tools for quality control tests for grouting material.

Activities during grouting

Grouting of a duct should be done without interruption. The speed at which the grouting material is pumped through the duct can vary from 5 to 15

- grouting to be carried out in a continuous operation and at the rate and in the sequence specified in the method statement;

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m/min.

– grouting shall continue until the density of the grout flowing from the tendon ends and the vent openings is about the same as that of the injected grout; after closing the last vent, pressure may be held for about one minute to verify that there are no unacceptable leaks in the system.

Activities after grouting

– immediately after grouting check all accessible tendon areas for complete grouting e.g. by light tapping. If voids are suspected, pump grouting material again until voids are filled;
– all equipment to be thoroughly washed shortly after grouting, followed by thorough draining of pump, mixer and hoses;
– record all operations, measurements, tests and particular events and submit to the engineer;
– after setting of grouting material check all accessible tendon areas (caps, vents, inspection ports) for complete grouting. If unacceptable voids are found, top up with fresh grouting material, or with vacuum grouting as may be suitable.

Records

– records to be established on all operations, measures, tests and particular events; to be transmitted to the design engineer.

Provisional inspection ports in the anchorages are recommended to check the grouting of the anchorages behind the anchor head.

Small voids which do not leave the prestressing steel unprotected do not need to be grouted.

Sealing of anchorages

To obtain a good seal of anchorage recesses, a preparatory treatment to the surface around the anchorages may be applied; in large recesses, connecting reinforcement should be provided.

Once the grouting material has set, all openings, grouting tubes and vents shall be hermetically sealed and the anchorage recesses filled as specified in the execution specification to prevent penetration of water and harmful products (e.g. de-icing agents).

8.4.4 Prestressing works for pretensioning tendons

8.4.4.1 Installation of tendons

Installation of the tendons and components such as hold-down devices at deviators shall be in accordance with the shop drawings and method statements relevant for the project.

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Tendon installation tolerances are provided in national or international standards; see e.g. ISO 22966, EN 13670 or ACI 318. See also subclause 8.6.5 of this Model Code.

The placing of anchorages and tensile elements shall be to the specified geometry and tolerances. Care shall be taken to avoid kinks in the tendon profile. Installation shall be in accordance with the relevant procedures.

Details of local de-bonding of tendons shall be provided as specified in the relevant shop drawings.

Tendons shall be protected from welding sparks.

The tensile elements shall be suitably protected from adverse environmental conditions. The time between installation and stressing of the tendons and pouring of the concrete providing permanent corrosion protection shall be limited in accordance with subclause 8.4.3.1.

Tendons shall be suitably marked for identification and recording during stressing and release operations.

Tendon components shall be inspected before placing of the concrete for compliance with the relevant project and system documents.

Traceability of tendon components shall comply with the execution specification.

8.4.4.2 Tensioning operations

General

Stressing of tendons shall be in accordance with the project specific method statements.

Refer to the FIP State of the Art Report: 'Tensioning of tendons: force-elongation relationship', Thomas Telford, London, 1986.

In general, the tendon force is determined with the pressure in the stressing equipment. In exceptional cases, direct force measurement with load cells or similar may be done.

Activities before stressing

- inspect tendon ends and abutments on which anchorages and stressing equipment will bear;
- check calibration certificates of stressing jacks to be recent.

The calibration certificates shall not be older than 6 months, in general.

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If the actual tendon forces and/or elongations are below acceptable values, the following measures may be considered:

- if the available prestressing steels and prestressing system allow, to apply a higher stress at the stressing end(s) of the tendons. This stress should never exceed the value of 0.95 $f_{p0.1k}$;
- install and stress additional internal tendons as may have been specified in the execution specification.

Suggested values for permissible deviations from expected elongations for deviated tendons are:

- for tendons of 15 m length or less: 15% for a particular tendon, but not more than 7% for the sum of all values of tendons in the same section;
- for tendons of more than 15 m length: 10% for a particular tendon, but not more than 5% for the sum of all values of tendons in the same section.

Suggested values for permissible deviations from expected elongations for straight tendons are:

- 10% for a particular tendon, but not more than 5% for the sum of all values of tendons in the same section.

Tendon elongations should be calculated with the properties of the prestressing steel as given in the material certificates.

Sudden release of the tendons may cause significant increase of the transmission length.

Activities during stressing

- stress tendons in the specified sequence gradually to the specified force and/or elongation;
- record pressure at jack or load cell and elongations of prestressing steel;
- compare measured and calculated values;
- take corrective actions if deviation between specified and measured values is greater than permitted tolerances.

Release of tendons

- check that concrete strengths have reached or exceeded the specified minimum strength for release;
- release the tendons with the approved method and in the specified sequence;
- record the tendon draw-in as specified for the project;
- compare measured and expected draw-in;
- observe instructions if deviation is greater than expected.

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Records

- pressure in stressing jack;
- elongations of tendons with adjustment for seating in tendon anchorages, as applicable, and for effects of measuring procedure (e.g. elongation inside stressing jack, if applicable);
- records of draw-in of tensile elements;
- records to be established on all operations, measures, tests and particular events; to be transmitted to the design engineer;
- any particular observations to be recorded during tensioning;
- site stressing records to be transmitted to and approved by the design engineer in view of any necessary adaptation of above instructions.

8.4.4.3 Sealing

All tendon ends, de-bonded tendon areas and any other penetration (e.g. hold-down devices at deviators) shall be hermetically sealed as specified for the project to prevent penetration of water and harmful products (e.g. de-icing agents).

8.4.5 Replacement of tendons

Any potential future replacement of internal unbonded or external tendons shall be carefully planned and carried out by an experienced specialist contractor.

Particular procedures have to be specified and applied. When replacing the tendons by cutting individual elements, special procedures have to be applied to avoid the potential transfer of load from already cut individual prestressing elements to the remaining elements due to bond or clamping effects at deviation points which could overload the tendon and lead to an unintended sudden failure of the tendon.

Care shall be taken to avoid unacceptably high unbalanced loads on deviators or diaphragms with tendon anchorages.

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ISO 22966 provides details of required construction documents.

8.4.6 Construction documents – prestressing

Prestressing works shall be defined with details and required quality of works in a execution specification containing as a minimum: (i) construction drawings (shop drawings) giving all necessary information such as geometry of the structure, number and position of prestressing tendons and anchorages; (ii) description of all prestressing systems to be used with any requirement for the application of the systems; (iii) works description (method statements) including inspection requirements.

Actual construction works shall be documented with sufficient detail and in sufficient number of construction records/certificates as specified in the construction specification. Actual prestressing systems hardware shall be documented with assembly drawings.

8.5 Falsework and formwork

fib Bulletin 48, "Formwork and falsework for heavy construction", gives guidance for the selection of falsework and formwork.

Falsework and formwork including their supports and foundations shall be designed and constructed so that they are:

- capable of resisting any foreseeable action to which they are submitted during the construction process;
- stiff enough to ensure that the tolerances specified for the structure are satisfied and the integrity of the structural member is not affected.

8.6 Concreting

8.6.1 Specification of concrete

"*D*" represents a sieve size *D* of the aggregate to be used in the concrete shall not be less than that given in the execution specification.

"*D*" represents a sieve size *D* of the aggregate which at least 80% to 85%, but not more than 99% of the sample pass.

ISO 22965-1 defines concrete as " … mixing cement, coarse and fine aggregate and water …".

Concrete and its specification shall comply with ISO 22965.

The concrete specification shall include requirements given in the execution specification and requirements related to the actual method of execution.

The actual upper sieve size *D* of the aggregate to be used in the concrete shall not be less than that given in the execution specification

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Standards on aggregates may define coarse aggregates as aggregate with $D \geq 4$ mm where D is the actual upper sieve size of the aggregate. Included in this definition are both a maximum and a minimum percentage passing on sieve size D.

The term d_{max} normally defines the "maximum nominal upper aggregate size" which is selected to ensure a proper casting taking into account the cover and free spacing between the reinforcement bars.

In principle, any value of d less than d_{max} satisfies a requirement related to d_{max}.

Concrete with aggregates of upper size d in the range from 4 mm to 12 mm might in some cases not support the design assumptions, e.g. aggregate interlock, shear capacity, stiffness, fracture energy.

ISO 22965-1 therefore states in a note that in concrete for general purpose the coarse aggregate should normally have a minimum size of 16 mm.

To ensure compatibility with the design assumptions, the actual maximum size of the aggregate being used should be specified, in particular if d smaller than 16 mm is allowed.

SCC mix design should comply with specific requirements in the fresh state depending on the type of application, and especially on:

- confinement conditions related to the concrete element geometry and the quantity, type and location of reinforcement, inserts and recesses;
- placing equipment (pump, truck-mixer, skip, etc);
- placing methods (number of delivery points);
- finishing method.

Those requirements might be expressed and justified in terms of:

- flowability and grouting ability;
- viscosity (measure of the speed of flow);
- passing ability (flow without blocking);
- segregation stability.

8.6.2 Placing and compaction

The concrete shall be placed and compacted in order to ensure that all reinforcement and cast-in items are properly embedded and that the concrete achieves its intended strength and durability.

By the use of concrete described as Self Compacting Concrete (SCC), the compaction of the fluid concrete is achieved due to the effect of gravity. Working procedures for the actual cast shall be established, based on the constructor's experience and/or pretesting, to enable the required compaction to be obtained.

Additional requirements to those given in ISO 22965 to the fresh concrete properties and its conformity criteria, if any, shall be agreed with the producer.

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The following methods are suitable for curing used separately or in sequence:

- keeping the formwork in place;
- covering the concrete surface with vapour-proof sheets which are secured at the edges and joints to prevent draughts;
- placing of wet coverings on the surface and protection of these coverings against drying out;
- keeping the concrete surface visibly wet with suitable water;
- application of a curing compound of established suitability.

Other curing methods of equal effectiveness may be used.

At the time of publishing this Model Code, standardized test methods characterizing the properties of curing compound are not available.

The quality of the curing will influence the properties of the concrete cover. Good curing will realize the potential of the mixed concrete, while less good curing will not. Deemed-to-satisfy requirements for service life design normally ends up with tabulated requirements to cover thickness and cover quality. The tradition differs from nation to nation on how the quality of curing is handled. In some national provisions a relatively high w/c ratio (proxy characteristic for the potential quality) is specified together with strict requirements for the curing regime. In other provisions lower w/c ratios are given with a more relaxed demand for the curing.

ISO 22966 gives requirements for the construction operations involving structural precast elements from their reception at the site or in the case of site manufactured elements from removal from the forms, until the completion of their installation and final acceptance.

The placing of any additional reinforcement and in-situ concreting for the completion of the structure shall conform to the relevant clauses of ISO 22966.

8.6.3 Curing

Concrete in its early life shall be cured and protected:

- to minimize plastic shrinkage;
- to ensure adequate surface strength;
- to ensure adequate surface zone durability;
- from harmful weather conditions;
- from freezing;
- from harmful vibration, impact or damage.

ISO 22966 defines 4 "curing classes" for the duration of applied curing ranging from 12 hours to 70% of specified characteristic 28 days compressive strength.

The curing class to be applied shall be stated in the execution specification.

8.6.4 Execution with precast concrete elements

Precast elements shall be used as specified in the execution specification and the design coordination between them and the structural performance of the overall structure shall be verified.

Handling, storage and protection of the precast elements shall be carried out in accordance with the execution specification.

Requirements for the placing and adjustment of the precast elements shall be given in the erection specification.

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ISO 22966 gives 2 sets of permitted geometrical deviations.

The requirements given in clause 10 of ISO 22966 are considered as achieving the design assumptions for the normally required level of safety, while those given in annex G of ISO 22966 are assumed to have relevance for service performance.

The tolerances given in subclause 4.5.2 are slightly different to those given in ISO 22966, but may be considered as equivalent.

Any requirements for special tolerances shall be identified in the execution specification and the following information shall then be given:

- any amendments to the permitted deviations given in ISO 22966;
- any further type of deviation to be controlled, together with defined parameters and permitted values;
- whether these special tolerances apply to all relevant components or to particular components which are identified;
- if the "box principle" shall be applied, and what deviation is permitted.

The "box principle" will require that all points of the structure are within the specified theoretical position with a margin in any direction corresponding to the permitted deviation.

Structural connections including other materials, for instance steel, shall conform to the relevant standards for execution like international welding standards.

8.6.5 Geometrical tolerances

The completed structure shall be within the maximum allowable deviations to avoid detrimental effects in terms of:

- mechanical resistance and stability in transient and in service stages;
- service performance during the use of the construction works;
- placing compatibility for the erection of the structure and its non-structural components.

Deviations from the specified tolerance range shall be handled in accordance with subclause 8.2.3. Small deviations which have no significant consequence on the performance of the finished structure may be ignored.

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Special performance requirements may be defined for special structures, such as historical structures. Special performance requirements, such as preservation of structural and architectural features in the structures, may restrict the type of intervention.

Information obtained via conservation activities can provide feedback to the design employed (in particular to the verification of limit states associated to durability, see clause 7.8), facilitating assessment of compliance with the original performance requirements or revised performance requirements, if appropriate.

Activities to achieve the intended service life of a structure.

Activities to achieve an extended service life of a structure or revised performance requirements.

9 Conservation

9.1 General

In this Model Code, conservation is defined to mean all activities aimed at maintaining or returning a structure to a state which satisfies the defined performance requirements.

This chapter provides a general basis for the conservation of concrete structures and the components thereof, during their planned and/or extended service life, so as to ensure that the performance level of the structure/its components remain above that required for structural safety and serviceability.

Typically, two different conservation objectives are distinguished:

- conservation activities concerned with enabling a structure to meet its intended service life, as envisaged at the time of design;
- conservation activities concerned with extending the planned service life of a structure or enabling it to meet revised performance requirements (e.g. revised loading or functionality needs).

Condition control activities required for the conservation of a concrete structure shall be planned when designing the structure. These plans shall be developed on the basis of the design assumptions and a prognosis for the behaviour under the envisaged environmental and loading conditions over its intended service life, considering the performance requirements that need to be met.

Condition control activities required for the conservation of a concrete structure to extend its intended service life or accommodate revised performance requirements shall be developed when re-designing the structure on the basis of knowledge of the current condition of the structure, a prognosis for future behaviour under the envisaged environmental and loading conditions and recognizing the implications of the revised performance requirements.

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Whilst one conservation strategy might be chosen for the management of an entire structure, in other circumstances different conservation strategies could rationally be adopted for the management of different groups of component parts of a structure.

Current engineering practice for conservation management for new structures is typically based on minimizing of costs to achieve and maintain a minimum performance/quality requirements (e.g. minimizing the costs of design, construction and conservation whilst satisfying appropriate performance/quality requirements).

For existing structures, optimization of conservation schemes is often similarly based on minimizing of the costs of conservation, whilst satisfying

9.2 Conservation strategies and tactics

9.2.1 General

The necessity for conservation works varies widely and is influenced by many factors including the design and detailing of the structure, the environmental and loading conditions, along with the performance requirements.

Appropriate conservation strategies (one or possibly more) shall be chosen for the management of the structure, with suitable conservation tactics being chosen for implementation.

For new structures, the conservation strategy and tactics shall be defined during design stage. Both the chosen strategy and tactics shall be confirmed/revised on the basis of the outcome of the condition evaluation, plus any associated intervention works, performed after construction of the structure.

For existing structures without a Condition Control Plan in place, the preliminary conservation strategy and tactics shall be set out during the re-design stage. Both the chosen strategy and tactic shall be confirmed/revised on the basis of the outcome of the current condition evaluation and current performance requirements.

In the case of revised performance requirements, the conservation strategy and tactics shall be revised to meet the new performance requirements. Both the chosen strategy and tactics shall be set out/confirmed/revised during the re-design stage on the basis of the outcome of the current condition evaluation.

The goal will usually be to minimize life cycle costs for the structure, as discussed in clause 3.5.

The choice of conservation strategy and tactic to be used depends on various factors including the:

- consequences of a potential failure; which are related to the social and economic importance of the structure, the impact on third parties, the function of the structure or components thereof, the design, etc.

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minimum performance/quality requirements.

Rational optimization requires an approach which weights the direct and associated indirect through-life cost of conservation against the benefits such as a longer service life, fewer faults or greater safety. Increasingly, attempts are being made to deal with conservation management on the basis of rational risk analysis. For more details refer to "Probability in Civil Engineering" CUR Report 190, 1997.

Conservation activities are categorized on the basis of whether they:

- adopt proactive or reactive activities;
- adopt planned or unplanned activities;
- amend the reference performance level of the structure or component part; whether the activities are to maintain the intended performance level (i.e. a maintenance or repair activity) or are to enhance the current performance level (i.e. strengthening activity).

The reasons for adopting a proactive approach might include consideration of factors relating to technical, functional, aesthetic or economic issues. For example, this might be where interventions will be expensive and/or technically difficult if undertaken once appreciable deterioration has occurred. A proactive approach is likely to be particularly appropriate for structures having a long design service life (e.g. monumental, important or sensitive structures).

- feasibility of evaluating the condition of the structure; that is how easy it will be to undertake a condition survey and assessment (which is related to the ease of inspections, measurements, the nature of the design, etc.).
- predictability of the service life of the structure or its components; which is related to factors such as the design service life, environmental conditions, etc.
- recording and quantification of the actions which occur during the service life of the structure (e.g. mechanical loadings, physical, chemical and biological actions on the structure).
- feasibility of preventative or remedial interventions (e.g. repairs, replacements, etc.).
- cost of conservation activities; which are related to matters such as the direct cost of the works; plus the frequency of inspections, repairs, replacements, indirect costs etc. when a through-life perspective is being taken.

The available conservation strategies and respective tactics are classified on the basis of their proactive versus reactive characteristics as follows:

- proactive conservation activities;
- reactive conservation activities;
- situations where conservation activities are not feasible.

9.2.2 Strategy using proactive conservation measures

A proactive conservation strategy is based on preventative (or protective) measures and interventions aimed at avoiding and minimizing future deterioration or loss by applying some form of treatment or taking action prior to damage becoming visible.

A proactive approach to conservation of concrete structures is desirable as it should enable early identification of problems and possible risk issues affecting the condition of the structure, potentially enabling early

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A proactive conservation strategy relates to an assumed Condition Control Level/Inspection Regimes CCL3. For a new structure these requirements would be defined during the Service Life Design (refer clause 7.8).

A proactive conservation strategy should be applied to structures or parts thereof where deterioration needs to be kept within acceptable and pre-determined bounds.

When appropriate limit states are defined beforehand that lead to the decision to increase the frequency of condition control (warning limits) and those that lead to the decision to carry out other conservation activities (activities limits). Condition based conservation may also be referred to as state dependent conservation.

Condition based conservation may be applied when it is difficult to make a prognosis of the future condition of a structure in the absence of information about the actions on the structure. Alternatively, condition based conservation may be adopted when regular determination of the condition of the structure by direct inspection is simple and cheap.

An example of this approach is the monitoring of the number of load or action cycles applied to a structure. Conservation works are undertaken after the registration of a certain number of load units (i.e. after registering an extreme action or after registering a pre-determined measure of cumulative actions). Thus the application of a load dependent conservation tactic involves a load unit norm. The criteria and threshold values (limit states) are established beforehand during the Service Life Design stage. Similar criteria might be applied for other types of actions.

Registration of the actions which cause deterioration is often difficult. In such situations a conservation tactic that involves a time based norm may be more appropriate and practical to implement.

preventative action to be taken to minimize the overall cost of ownership. A proactive conservation strategy should be applied to structures or parts thereof where it is necessary or desirable that the performance be kept above a specified minimum performance requirement.

Tactics for delivering a proactive conservation strategy may involve various approaches including the following, which are illustrated below:

- condition based conservation;
- time based conservation.

9.2.2.1 Condition based conservation

With this approach the condition of the structure or its components is determined regularly and the evaluated condition shall be registered, together with changes over time. The frequency at which the data is gathered and the associated evaluations are made may vary with time, potentially being related to the perceived criticality of the condition of the structure.

This approach requires that an appropriate parameter (or parameters) exists which can be evaluated and used to reflect the condition of the structure or its components. The condition based tactic involves norms related to limit states. Exceeding the pre-determined limit states invokes a preventative intervention to extend the service life of the structure.

Condition based conservation might involve consideration of factors such as:

- the number of load or action cycles applied;
- the development of cracking and/or spalling;
- loss of structural strength or stiffness;
- aesthetic effects (i.e. unacceptable deterioration in appearance).

9.2.2.2 Time dependent conservation

This approach involves the use of a time based norm to define when a preventative intervention should be undertaken to extend the service life of a structure.

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Time dependent conservation may be adopted when regular determination of the condition of the structure is not feasible or is expensive compared with the anticipated intervention costs. Different preventative interventions could be undertaken at different times/frequencies.

Reactive conservation strategy may be applied to structures or components thereof when apparent deterioration does not compromise meeting critical performance requirements. A reactive approach may be appropriate where direct condition control by regular inspections and interventions is not practical (e.g. because it is not feasible, too expensive and/or technically too difficult).

A reactive conservation strategy relates to the following assumed Condition Control Level/Inspection Regimes. For a new structure these requirements would be defined during the Service Life Design (refer clause 7.8):

– CCL2 is a reactive approach which utilizes a planned inspection but this is only visual in nature;

– CCL1 is a reactive approach utilizing an ad-hoc inspection and testing regime.

The failure dependent conservation tactic involves a pre-determined failure norm/non-compliant performance condition. Thus on this basis a remedial intervention, perhaps involving a repair or replacement, to a structural component is not undertaken until it no longer meets the specified performance requirements. The failure dependent conservation tactic might potentially be applied to those parts of the structure that do not contribute to the probability of failure of the structural system as a whole. However proper judgment of the consequences of failure is required because use of a failure dependent conservation tactic would often not be acceptable because of the potentially large consequences of failure.

This involves estimating an appropriate timing of preventative interventions or the time period between such interventions. The goal is to estimate a time period (life span) corresponding to a sufficiently low probability of failure/non-compliance for the actions being considered, typically taking them as a random variable over the period of time in question.

9.2.3 Strategy using reactive conservation measures

A reactive conservation strategy is based on remedial (or corrective) measures and interventions, aimed at arresting currently active processes which are causing deterioration or damage. Typically remedial interventions involve some form of treatment or the taking of measures after damage has become apparent, presumably involving visual indications (e.g. cracking or spalling of concrete).

Tactics for delivering a reactive conservation strategy may include waiting for a specified failure/non-compliant condition to develop.

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There are parts of structures, such as foundations, where it may not be feasible to apply direct condition control & conservation measures.

The situation where there is no conservation strategy relates to the Condition Control Level/Inspection Regime CCL0, which assumes that no direct physical inspection or monitoring is feasible.

For a new structure these requirements would be defined during the Service Life Design (refer clause 7.8).

In such circumstances it may be possible to gather indirect indications of structural performance through ancillary behaviours. For example, the performance of foundations might be inferred from related indirect parameters, such as differential displacements and/or relative movements within the various parts of structure.

These activities will require proper organization of the through-life conservation process, including development of the Condition Control Plan (CCP) for chosen conservation strategy and tactics (see subclause 9.3.2).

Relationships to the activities described in this section and related activities within the Model Code for Service Life Design are drawn out.

Condition survey, condition assessment and condition evaluation are condition control processes, which are performed as a part of the overall conservation process. For a chosen conservation strategy and conservation tactic, an appropriate Condition Control Plan (CCP) shall be developed that facilitates conservation management; see subclause 9.3.2

9.2.4 Situations where conservation measures are not feasible

There are parts of structures where direct condition control, inspection and the application of conservation measures is very difficult or not practical, or where it would be difficult economically and/or technically for preventative or remedial measures to be undertaken.

However, it may be feasible to obtain indirect indications of condition or performance of these elements via activities such as land surveying, structure or ground movement measurement, or other indirect but related activities such as leakage detection. Before this approach is adopted a proper judgment should be made of the consequences of failure and the feasibility of condition control.

Should significant damage or deterioration occur and be detected, a potentially difficult and expensive reactive or remedial intervention could be required.

9.3 Conservation management

9.3.1 Through-life conservation process

To allow effective and efficient life cycle management, the conservation process shall be adequately managed and documented. In particular, the conservation planning shall be prepared in accordance with the selected conservation classification for the structure or its component parts, as appropriate.

In general, the through-life conservation process for a structure involves the following types and sequence of activities:

- condition survey i.e. gathering information on the current condition and the previous development to that condition – see clause 9.4;
- condition assessment i.e. assessing available information to obtain an indication of current performance and to make a prognosis of future performance, including identification of deterioration mechanisms and prediction of damage – see clause 9.5;

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Figure 9.3-1 shows an idealized representation of the through-life performance of a structure requiring a remedial intervention to meet its intended design service life.

After completing the construction, the first condition control is performed and the Birth Certificate Document is recorded. During the post-construction lifetime, knowledge of the performance of the structure is accumulated by means of through-life condition survey and/or monitoring.

Evaluation of the performance records against the performance criteria supports decision-making as to whether there is a need for an intervention. After carrying out the (required) intervention, the condition control is performed and a Re-birth Certificate for a structure is recorded. During the post-intervention lifetime the knowledge of the performance of the (possibly revised) structure is accumulated by means of through-life condition survey and/or monitoring.

- condition evaluation and decision making activities are concerned with establishing the implications for the conservation of an asset (see clause 9.6), evaluating potential conservation options, the selection of an appropriate intervention option(s) and the timing of such works– see clause 9.7;
- execution of preventative or remedial works – see subclauses 9.7.2 and 9.7.3;
- undertaking through-life condition survey and/or monitoring including recording of the information required for life cycle management – see clause 9.8.

The general flow of conservation process procedures for new structures and for existing structures is shown in Figure 9.3-2. In parallel, it is indicated which information shall be recorded to meet the requirements of Life Cycle Quality Management, as discussed in clause 3.6 with regard to development of the Life Cycle File.

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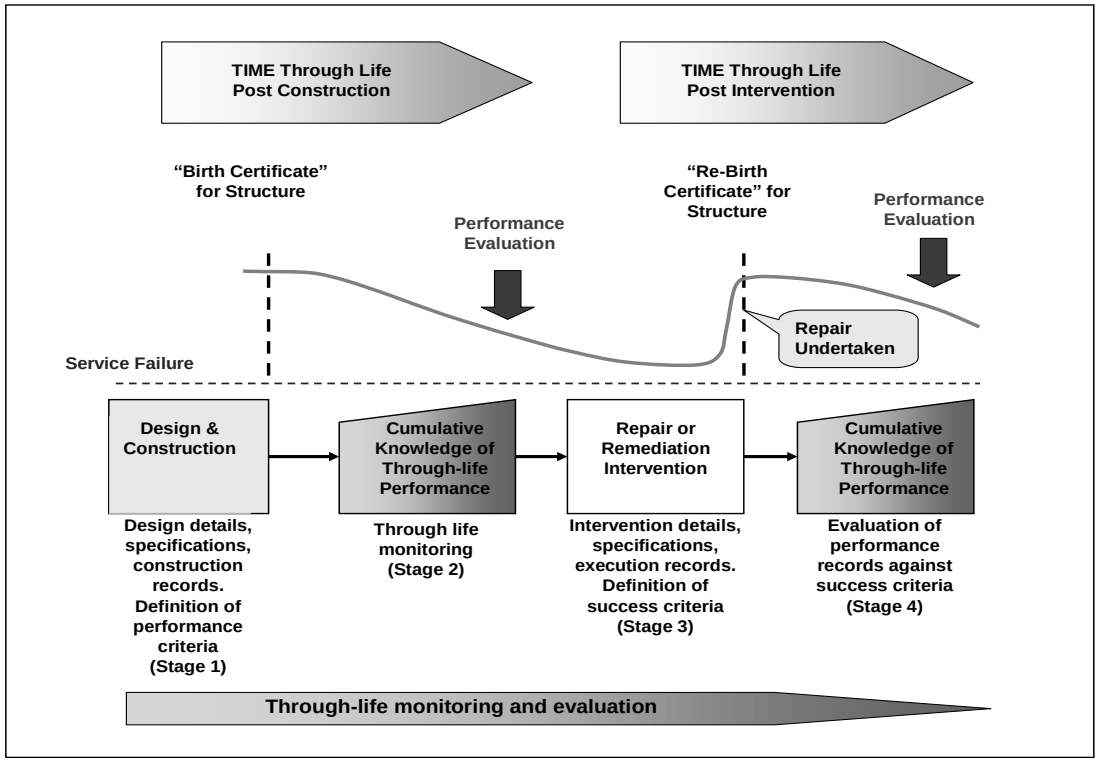


Figure 9.3-1: Idealized representation of the through-life performance of a structure requiring a remedial intervention to meet its intended design service life

Notes to Figure 9.3-1:

- Stage 1. Details about the design concepts and execution of the original structure along with the expectations for its performance, including material specifications and information upon the quality on execution, to allow the Birth Certificate Document to be prepared.
- Stage 2. Gathers information upon/monitors through-life environmental and loading influences, as well as performance post construction and prior to a preventative or remedial intervention.

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– Stage 3. As Stage1 but for preventative or remedial intervention(s), plus the definition of appropriate success criteria upon which to evaluate the subsequent performance of the intervention to allow the Re-Birth Certificate Document to be prepared.

– Stage 4. Monitoring of through-life performance post-intervention, using defined performance indicators and environmental parameters and related factors establishing the context within which this data should be evaluated against the success criteria defined in Stage 3 (and recorded in Re-Birth Certificate Document).

Condition evaluation: Use of an agreed methodology to assess the overall condition of the structure (i.e. condition control is performed); see clause 9.6.

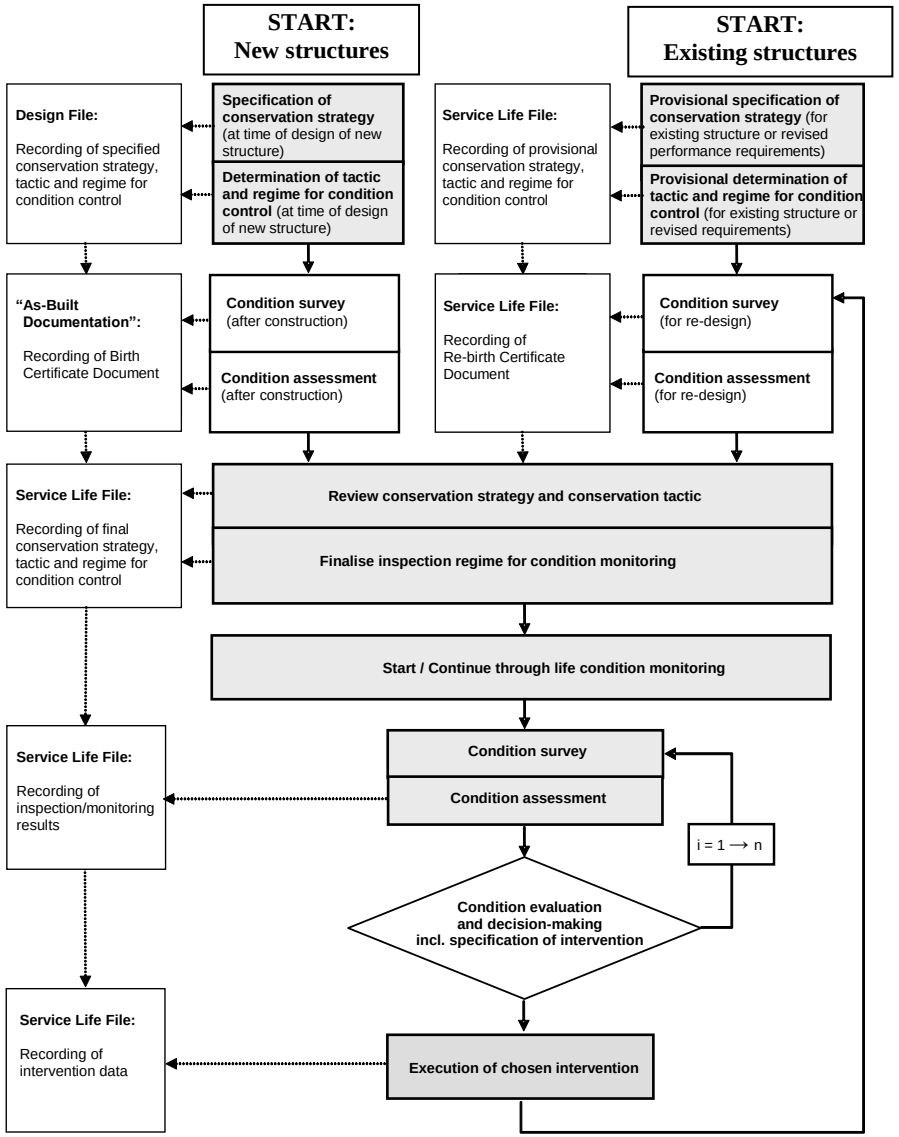


Figure 9.3-2: Conservation management: through-life conservation process and recording of information

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The necessity for condition control, conservation and maintenance works varies widely depending on factors such as the social and economic importance of the structure, the nature of its design and that of its components, potential threats posed to third parties and the intended (planned) service life of the structure. Account needs to be taken of the potential difficulties of undertaking inspections and associated condition control, conservation and maintenance works, along with prediction of deterioration and execution of intervention works.

Examples of structure within the three categories of condition control & conservation are as follows:

Category A:

– structures whose deterioration must not be apparent or would be technically unacceptable;
– monumental, important or sensitive buildings and structures.

Category B:

– structures for which remedial measures can be taken after deterioration becomes apparent (presumably visually);
– buildings and other common structures.

Category C:

– structures where it would be difficult economically and/or technically for preventative or remedial measures to be taken, such as foundations, for which direct inspection is so difficult and/or costly that assessment and judgment may need to be made based on information gathered on indirect performance indicators.

The Conservation Plan shall state the types of inspection, testing and condition monitoring that shall take place, what components of the structure are to be inspected/monitored, what the frequency of the inspections shall be, etc.

The provision for planned activities in Conservation Plan shall address the deterioration processes that are relevant for a structure during its service life.

The condition control and conservation planning adopted in Chapter 9 shall accord with the selected conservation classification for the structure or its component parts, as defined during the service life design process (as defined below).

Different categories are defined depending on factors such as the importance of the structure, its function, design service life, impact on third parties, environmental conditions, ease of maintenance and cost. Requirements for condition control should be defined at the time of design. The categories are as follows:

Category A: structures or parts thereof which are to be managed by planned proactive condition control and conservation.

Category B: structures or parts thereof which are managed by reactive condition control.

Category C: structures or parts thereof for which condition control & conservation measures are extremely difficult or are not practical.

9.3.2 Conservation Plan

The Conservation Plan shall state the type and frequency of the foreseen conservation activities and may include:

– specification of regime for inspection, testing and condition monitoring activities; see clause 9.4;
– methods of estimation the degree and rate of deterioration; see 9.5;
– methods of evaluating structural performance against the performance requirements (criteria to be met); see 9.6;

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This process leads to the final Conservation Plan to be adopted for the through-life management of the structure.

This process leads to a Conservation Plan which is to be adopted for the future through-life management of the structure.

The results are usually compared to specified requirements and standards for determining whether the item or activity is in line with these targets.

Surveys may involve visual inspections, measurements, tests or some form of gauging applied to certain characteristics in regards to an object or activity. Testing undertaken is usually non-invasive or non-destructive in nature. Monitoring often involves the automatic recording of performance related data for the structure and possibly some degree of associated data processing. Strictly this does not need to be so, as there are a variety of

– interventions which may be required as a result of non-conformity with the performance requirements; see 9.7;
– methods of documentation of condition control; see 9.8.

For new structures, the conservation strategy and conservation tactics shall be defined and the Conservation Plan shall be developed during design stage. Both strategy and tactics shall be reviewed on the basis of the outcome of condition evaluation performed after construction of the structure.

For existing structures without a Conservation Plan in place, a Conservation Plan shall be developed during the re-design stage on the basis of the current condition evaluation and consideration of the current performance requirements.

In case of the revised performance requirements, the conservation strategy and tactics, along with the Conservation Plan, shall be revised to meet new performance requirements/criteria to be met. These shall be reviewed during the re-design stage, on the basis of the outcome of the current condition evaluation and consideration of the revised performance requirements.

9.4 Condition survey

9.4.1 Condition survey and monitoring activities

In this Model Code, condition survey is understood to mean activities performed to gather information regarding the form and nature of the structure, current or potential deterioration mechanisms and/or change in performance of the structure or service conditions which are needed to evaluate conformity with the design (data) for actions and/or material and/or product properties. Condition survey is a part of condition control, which also involves condition assessment (see clause 9.5) and condition evaluation (see clause 9.6).

Accordingly, condition surveying is understood to mean an organized examination by a range of appropriate techniques and procedures performed on a periodic basis during the service life of the structure; while condition monitoring is understood to mean the process of observing such parameters on a quasi-continuous basis.

Significant changes in the data relating to the condition of the structure

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means of gathering appropriate data.

Engineering evaluation and interpretation of the data obtained is central to the process of converting data into useful context related information.

It may be possible to gather indirect indications of performance through ancillary behaviours. For example, the performance of foundations might be inferred from the measurement of differential structural movements.

Proactive conservation requires more than just periodic visual inspections because environmental factors slowly and subtly change the internal conditions within the outer zone of the concrete (e.g. typically within the concrete providing the cover to the embedded reinforcing bars). As a result concrete structures tend to experience changes in their condition which initially do not provide visible indications. Thus these changes do not manifest themselves as deterioration. Accordingly these changes cannot be recognized on the basis of visual observations alone.

It is only perhaps many years later that physical deterioration becomes sufficiently developed to be visibly evident (e.g. cracking and spalling of concrete arising from the corrosion of embedded reinforcing bars). As these changes and deterioration processes generally develop progressively from the surface of the concrete, and often over considerable periods of time, information on the progress of these actions is generally gathered using appropriate forms of non-destructive testing or sampling of the concrete. Appropriate data can also be obtained by suitable monitoring techniques.

may be indicative of a developing failure. However, such deductions can only be established through an adequate process of data assessment and evaluation, which enables an appropriate context-based engineering interpretation to be made of the indications obtained.

Surveys, inspections, testing and condition monitoring activities shall be carried out from an early stage in the service life of the structure.

Regimes for survey and/or monitoring the condition shall be devised on the basis of the requirements of the conservation strategies selected for the structures or their components.

Potential approaches are shown in Table 9.4-1.

Table 9.4-1: Inspection, testing and monitoring regimes for classes of condition control/conservation strategy

Conservation strategy/ Class of condition control	**Inspection, testing and monitoring regime (Associated Condition Control Level)**
Proactive conservation measures (Category A structures or structural elements)	Planned periodic inspection and systematic monitoring of parameters relevant to the design, in particular to the deterioration processes that are critical for the verification of the limit states associated to durability. (Condition Control Level: CCL3)
Reactive conservation measures (Category B structures or structural elements)	Planned periodic inspection (i.e. visual inspection by qualified personnel). No systematic testing or monitoring. (Condition Control Level: CCL2)
	Ad-hoc inspection and testing/investigation. No systematic inspection, testing or monitoring. (Condition Control Level: CCL1)
No conservation measures (Category C structures or structural elements)	No direct inspection, testing or monitoring. (Condition Control Level: CCL0)

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Whilst similar inspection, testing and monitoring activities may be employed, criteria for evaluating the results may differ where a structure is to meet:

- its intended service life as envisaged at the time of design; or
- revised performance requirements, which might include an extension of service life, revised loading and/or other actions or functionality needs.

Information acquired by inspection, testing and monitoring the condition of the structure can greatly improve the accuracy of performance prediction by more accurately assessing the variability of the input parameters, which are typically assumed to be random variables. This can help reduce the uncertainty associated with the input parameters and aid characterization of their variability, as illustrated in Figure 9.4-1.

9.4.2 Locations for surveys and monitoring activities

Locations where inspection, testing and condition monitoring activities are to be undertaken shall be carefully selected so that the desired information about the deterioration of materials and/or structural performance can be obtained, keeping in mind factors such as:

- the likely mechanism(s) and rate of deterioration;
- the environmental conditions;
- the conservation strategy and tactics, and the inspection testing and monitoring regimes defined at the time of design or re-design.

9.4.3 Tools and techniques for surveys and monitoring

Tools and techniques for inspection, testing and monitoring could involve a wide range of procedures. Typically they are likely to include a combination of visual observations, material sampling and possibly selected non-destructive and non-invasive testing methods.

Gathering the data necessary to establish the form and current condition of a structure may involve numerous techniques including inspections, measurements, sampling, as well as various forms of local condition and global response testing.

The approaches chosen shall be appropriate to:

- the required reliability of the information to be obtained;
- the conservation strategy and tactics, and the inspection testing and monitoring regimes defined at the time of design or re-design.

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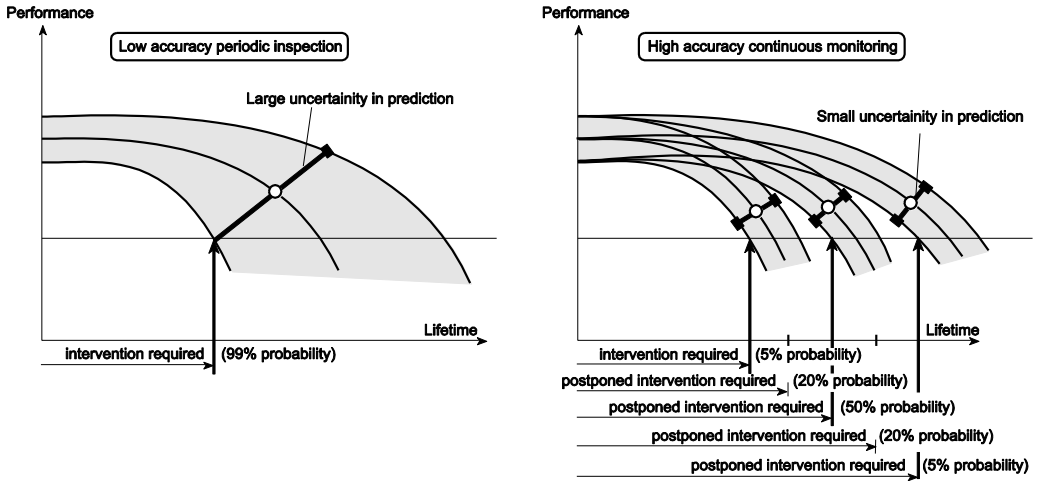


Figure 9.4-1: Estimating the timing of an intervention – illustrating the influence of the uncertainty of the input data

Figure 9.4-1 illustrates how the reliability of the data gathered and the assessment made using it can influence the estimated timing of required interventions, which can change significantly when the level of uncertainty in the data and assessed outcome are taken into account.

For more information regarding specific tools and techniques for inspection, testing and monitoring reference may be made *fib* Bulletin 17: "Management, maintenance and strengthening of concrete structures – maintenance, operation and use" (*fib*, 2000), and *fib* Bulletin 22: "Monitoring and safety evaluation of existing concrete structures" (*fib*, 2003).

An inspection is 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.

A survey is a process, often involving visual examination but which may utilize various forms of sampling and localized condition testing (e.g. measurement of depth of cover to reinforcement), whereby information is

9.4.4 Gathering data for Condition Control purposes

Data may be gathered from the structure for Condition Control purposes by a variety of techniques which are used for undertaking inspections, measurements, testing and monitoring activities.

Inspection, surveys and investigations can be classified on the basis of their nature and the methods to be used, their frequency, the planning and timing of inspections/surveys, as shown in Table 9.4-2.

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gathered about the form and current condition of a structure or its components. The testing undertaken is usually non-invasive or non-destructive in nature.

An investigation is a systematic process of inquiry undertaken to establish the specific details of a situation without restriction upon the methods used. For example, it may involve inspections, measurements, sampling, as well as various forms of local condition and global response testing to establish the form and current condition of a structure. The process may also provide data for a prognosis of future condition and behaviour of the structure.

For more information regarding regimes for inspection, testing and monitoring reference may be made to the *fib* Bulletin 44: "Concrete structure management – Guide to ownership and good practice", (*fib*, 2008).

Initial inspection/survey should include not only the obvious inspection of external appearance of the structure, but potentially should address other 'out of sight' parameters (e.g. cover to reinforcing bars) and workmanship issues. Data from the initial inspection/survey can be of use for subsequent routine/regular, detailed and extraordinary inspections/surveys.

An initial inspection/survey related to the development of a Birth Certificate for the structure should be conducted as the first step in collecting information on the structure.

In the case of an existing structure requiring a remedial intervention, such a process would lead to the development of a Re-Birth Certificate for the structure. This would be expected to lead to the adoption of a pro-active conservation strategy for condition control in the structure. Thus such an

Table 9.4-2: Gathering data for Condition Control purposes for various conservation strategies

Gathering procedure (Inspection/survey type)	**Proactive conservation measures**	**Reactive conservation measures**	**No conservation measures**
Initial inspection or survey	Planned	Planned	None
Routine (or regular) inspection or survey	Planned	Planned or ad-hoc	None
Detailed investigation	Planned	Planned or ad-hoc	None
Extraordinary inspection or survey	Ad-hoc	ad-hoc	None

Initial inspection or survey

An initial inspection/survey is carried out using appropriate tools and techniques in order to examine whether or not the structure is adequately constructed or whether previously specified preventative/remedial actions have been implemented satisfactorily.

It may also provide the means of collecting basic data about the as-built or repaired structure in order to initiate appropriate maintenance or preventative intervention works, or as part of a previously unplanned inspection/survey regime.

The following three types of inspection/survey are assumed as the initial inspection:

- inspection/survey carried out prior to the use of newly constructed structure upon completion of construction (Birth Certificate),
- the first inspection/survey of an existing structure which has been subjected to a major preventative or remedial intervention, or substantially renewed (Re-Birth Certificate),

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initial inspection/survey should be conducted as the first step in collecting information on the structure.

The specific tools and techniques to be used and the frequency of such inspections/surveys shall be decided on the basis of factors such as the likely mechanism(s) of deterioration, environmental conditions, importance of the structure etc. It is anticipated that these issues would have been considered at the time of design or re-design, when an appropriate conservation strategy and condition control level would be selected.

It is usually desirable that the structure or the components concerned be inspected at close range (arms length). Some form of non-destructive testing, such as estimating the depth of carbonation or the penetration of chlorides, is likely to be required in order to understand the progress of initiation of deterioration. Not all inspections/survey need have the same objectives and coverage. Thus the focus of particular inspections/survey may differ and the scope and methods used would be expected to vary depending upon the nature of the structure or the component being inspected. Not all of these will need to be inspected/surveyed at the same frequency or in the same degree of detail.

This type of activity is intended to obtain detailed data regarding the deterioration and performance degradation of the structure or the component(s) concerned. The approach will often be hypothesis driven on the basis of the deterioration mechanism(s) acting or suspected. These activities are likely to be guided by records of past inspections/surveys, as well as those of previous maintenance, preventative and remedial works interventions. This information would typically be reviewed in conjunction with the results of deterioration prediction modelling.

The detailed quantitative data gathered might include characterization of the concrete surface, the quality of concrete, the depth of carbonation and chloride concentration in the concrete, the degree of corrosion of the reinforcing bars and/or similar parameters. Furthermore, the detailed

– the first inspection/survey of an existing structure carried out for the purpose of an intervention activity in the absence of previously collected data.

Routine (or regular) inspections or surveys

Routine (or regular) inspections/surveys shall be carried out periodically at intervals defined by the service life design to identify changing internal conditions within the concrete, indications of deterioration and the time of their first appearance.

These inspections/surveys shall be carried out using appropriate tools and techniques to meet the objectives of the particular inspection.

Detailed investigation

A detailed investigation shall be carried out to obtain sufficient specific information concerning the deterioration of materials and/or structural performance to facilitate detailed evaluation or re-design of the structure or components concerned. There are various circumstances when a detailed investigation may be required including when:

– signs of significant deterioration or a change in the performance level are observed during a routine/regular inspection/survey;
– a routine/regular inspection/survey is unable to provide the information required;
– it is suspected that the structural integrity of the structure could have been adversely affected by the extent of deterioration.

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inspection will need to include the data on the environmental conditions, such as temperature, humidity, airborne chlorides, loads and actions acting on the structure, and so on.

The investigation will require use of appropriate tools and techniques, such as non-invasive and non-destructive testing and core drilling for material sampling, with the aim of locating and defining deterioration, damage and defects. Provision for close access to parts of the structure will be required.

Extraordinary inspections/surveys are usually carried out when the structure is subjected to some form of extreme event such as a natural disaster perhaps involving an earthquake or typhoon, fire, and collision involving vehicles or ships. Such an inspection is intended to accurately ascertain the state of the structure and judge if a remedial intervention is necessary. Extraordinary inspections/surveys may also be carried out after an accident associated with the deterioration of a structure, perhaps caused by a piece of spalled concrete falling causing injury to a passer-by.

An extraordinary inspection/survey would usually be carried adopting a similar methodology as a routine or regular inspection. If further information upon the form or condition of a structure is judged to be necessary, a detailed investigation might then follow. Extraordinary inspections/surveys may also be carried out for a particular class of structure after an accident due to the deterioration of such a structure, as noted above. Such inspections/surveys may be carried out for similar structures, where the same accident may occur.

Judgment criteria are further discussed in clause 9.6 on condition evaluation and decision-making.

More information is required to decide whether a major remedial intervention is necessary and upon its potential scope. In a detailed investigation the items for inspection and/or the locations where testing and/or measurements are to be made, shall be appropriate both in number and characteristic to the mechanism(s) of deterioration acting or suspected.

Extraordinary inspection or survey

An extraordinary inspection/survey will usually be carried out after the structure has been subjected to an accidental load or some form of extreme event, such as that caused by an earthquake, storm, flood, fire, impact by a vehicle or ship, to assess the extent of the damage and the need for a remedial intervention.

9.4.5 General flow of condition survey process

The general flow of the through-life conservation management process is shown in Figure 9.3-2. This is complemented by that in Figure 9.4-2, which presents a flow diagram illustrating the decision making process for inspections/surveys and interventions; with particular attention being given to circumstances where judgments need to be made.

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When making inspections/surveys or investigations of a deteriorated structure, consideration should be given to whether the structure poses an immediate threat to the environment, its users, any third party, or the inspection/investigation team.

In such a situation, suitable emergency measures and temporary works may be required to be taken immediately in order to minimize risks and to attempt to avoid potential collapse etc.

Depending upon the circumstances, it may be appropriate to take a number of immediate actions to minimize the risk of injury or worse. These measures might include closing the structure to further use, evacuating the structure and the surrounding area, setting up an exclusion zone to control access to the structure and the area which might be affected by any collapse and/or the release of stored/contained materials, shutting-off of services such as piped gas which might be damaged and create secondary hazards, etc.

Detail investigations may be required once the structure has been "stabilized" and put into a "safe" condition.

Figure 9.3-1 illustrates the time dimension associated with the inspection/surveys processes portrayed in Figure 9.4-2. Figure 9.3-1 shows an idealized representation of the through-life performance of a structure from the time of its construction up to the time when the first remedial intervention was undertaken so that the structure would meet its intended service life. The various inspections/surveys portrayed in Figure 9.4-2 would correspond to the through-life monitoring activities (Stages 2 and 4) shown in Figure 9.3-1. The intervention activities portrayed in Figure 9.4-2 would correspond to Stage 3 in Figure 9.3-1.

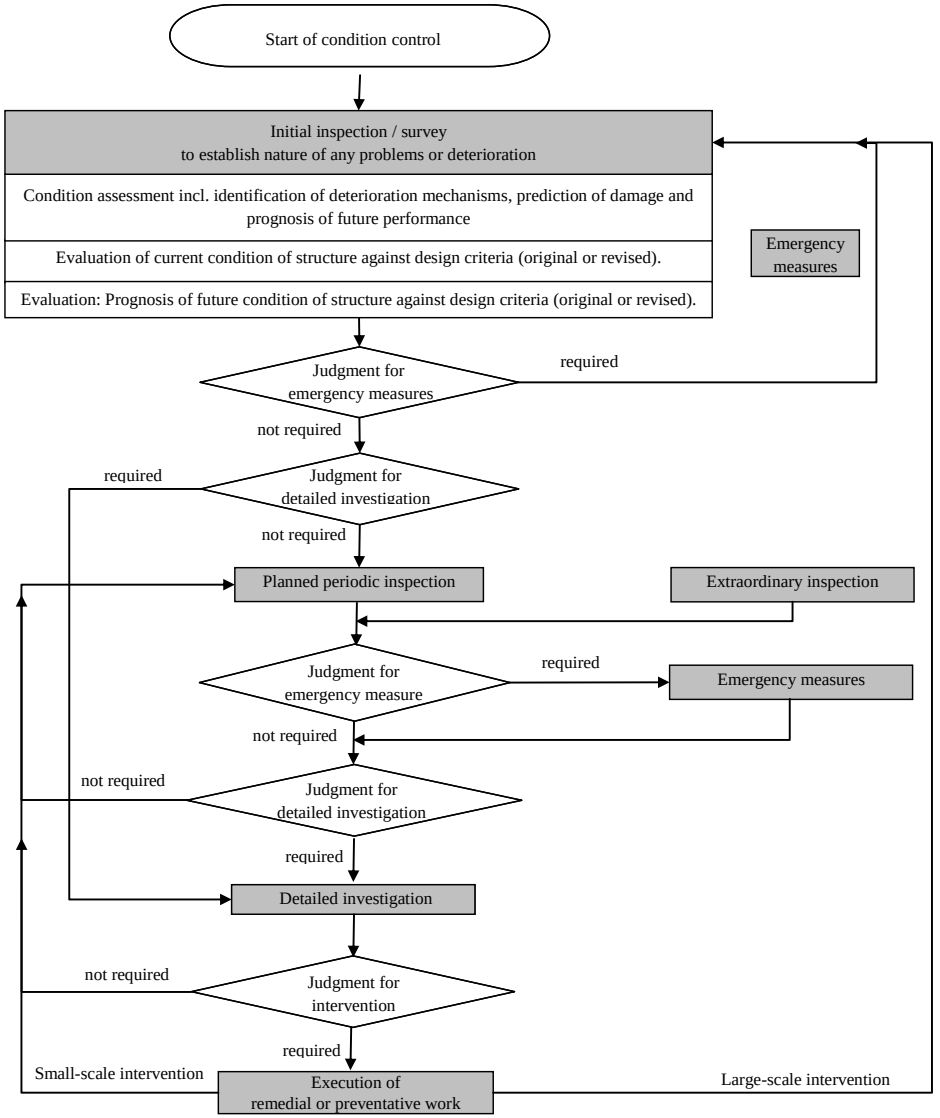


Figure 9.4-2: General flow of condition survey for structures taken into condition control following construction

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This process would be expected to draw upon information obtained for and detailed in the Birth Certificate, and potentially also in the Re-Birth Certificate assuming that these were available.

As activities required for conservation of a concrete structure to meet its intended service life shall be planned when designing the structure, this process would be expected to lead to the identification of the deterioration mechanism which might potentially affect the structure or its components.

Similarly the process of evaluation and re-design required to extend the intended service life of a concrete structure or to accommodate revised performance requirements would be expected to lead to the identification of the deterioration mechanism(s) which might potentially affect the structure or its components.

The following list illustrates some potential deterioration mechanisms, but should not be considered to include all possible mechanisms:

- carbonation induced corrosion of reinforcement;
- chloride induced corrosion of reinforcement;
- corrosion induced cracking and spalling of concrete;
- scaling, surface spalling or internal damage (disintegration) due to freezing-thawing with or without de-icing-agents;
- ASR (alkali silica reaction) induced cracking and deformation (expansion);

9.5 Condition assessment

9.5.1 Identification of deterioration mechanisms and prediction of damage

The deterioration mechanism, present deterioration level and deterioration rate of materials and/or structural performance shall be determined using appropriate models on the basis of the information obtained during from inspection, testing and monitoring activities, from the design and construction records, information upon previous interventions and the environmental conditions.

9.5.2 Identification of deterioration mechanism

It is necessary to identify the deterioration mechanism which are or might potentially affect the structure or its components.

These shall be identified from consideration of factors such as the environment to which the structure is exposed, variations in the exposure conditions, previous condition and performance data, if available, and any sign of deterioration that may be observed.

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– cracking, deformation (expansion, swelling) or dissolution due to chemical attack (sulphate attack, magnesium attack, acid attack, salt attack etc.);
– fatigue induced cracking or internal damage;
– sustained load induced cracking or internal damage.

Amongst the above mentioned deterioration mechanisms, corrosion and corrosion induced effects are the most common in many countries. The deterioration mechanisms under the combined effects of above mechanisms are also commonly observed.

This information would be used to make a prognosis for the behaviour of the structure under the envisaged environmental and loading conditions over the remainder of its intended or extended service life, considering the performance requirements that need to be met.

9.5.3 Factors influencing deterioration

Factors influencing deterioration may be classified into the following :

– actions upon the structure (external factors) such as temperature, humidity and any other characteristics of the environment, including physical loading;
– factors related to the nature of the structure (internal factors) such as designed parameters (e.g. material characteristics, concrete cover for the reinforcing bars, geometry of sections, etc.) and quality management during construction.

More than one factor may influence the development and progress of deterioration arising in a structure/member. For this case the combined (synergetic) effects of the deterioration factors shall be considered.

9.5.4 Determination of deterioration level and rate

The deterioration level and the rate of change of material properties and/or structural performance shall be determined from the results of the inspections/surveys and/or monitoring carried out and by using appropriate models for the mechanism(s) of deterioration.

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This process would be expected to draw upon information obtained for and detailed in the Birth Certificate, and potentially also in the Re-Birth Certificate assuming that these were available.

It is assumed that the inspection and monitoring process outlined above will be able to furnish sufficient information to permit decisions to be made in respect of options for the management of the structure. If there is not sufficient information it may be necessary to initiate a detailed inspection.

Decision making can be made either on a proactive (i.e. preventative intervention) or reactive (i.e. remedial intervention) basis, depending upon the underlying philosophy adopted for the management of the structure. It may be sufficient to re-design/evaluate the structure to address the performance non-conformity regarding limit states associated to durability. Consideration may also need to be given to changed performance requirements.

Account would need to be taken of the requirements for condition control and the conservation strategy and tactics defined at the time of design.

9.6 Condition evaluation and decision-making

9.6.1 General

An overall evaluation of the extent of material and/or structural performance deterioration shall be based on:

– records in the Design File and "As-Built-Documentation", see chapter 4;

– results of inspection and monitoring outlined above;

– assessment of the nature, severity, significance and rate of deterioration.

Besides, condition evaluation shall take account of the conservation strategy and tactics defined at the time of design or re-design of the structure.

Depending on how this evaluation compares with a minimum required performance level, a decision shall be taken on how these findings and prognoses impact upon the current and any proposed manner of managing the structure or its components. It may be necessary to undertake an appropriate preventative (proactive) or remedial (reactive) intervention.

Suitable preventative or remedial activities, including repair or strengthening, shall be initiated as necessary.

9.6.2 Threshold levels for deterioration of material and/or structural performance

The minimum level of performance for the (deteriorated) structure shall be determined at the time of design and/or re-design.

The level set will take account of the required performance, such as the load carrying capacity to achieve the required functional performance, as well as issues such as serviceability, and possibly other factors such as the appearance of the structure.

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The process of selecting the most appropriate intervention option for the given circumstances might involve:

- confirmation of the conservation strategy and tactics to be applied to the asset concerned, and whether there is any intentional differentiation of the performance standards across the asset (e.g. a higher standard of aesthetic appearance may be required on facades which are in public view, or a higher performance standard may be required on some façades to minimize the risk of spalled concrete falling into public zones);
- confirmation of appropriate indicators of through-life performance;
- establishing the threshold levels for the performance indicators to be used in the design and evaluation of the intervention, together with the subsequent post-intervention monitoring;
- identification of intervention methods appropriate to the desired result

9.6.3 Judgment criteria

When the results of the inspections undertaken (initial inspections, planned periodic inspections etc) show defects/damage/changes in condition (expected or otherwise), a detailed investigation should be undertaken to evaluate the effects of those defects/damage/changes in condition upon structural performance.

If the defects/damage/changes are obviously fatal to loss of structural safety and/or asset value, the emergency measures should be undertaken immediately before the detailed investigation is undertaken.

If the detailed investigation indicates that the structural performance does not satisfy the performance requirement or may not do so at some future time during the intended service life, it will be necessary to undertake an appropriate and timely intervention to rectify this situation.

The structural performance is evaluated and verified for the performance requirement by following the structural analysis and verification provided in Chapters 6 and 7 with the information related to materials and structural configurations, which have been obtained through the condition survey.

9.6.4 Selection of interventions

Selection of an intervention option, including the material to be applied, shall be made with consideration of the reasons for undertaking the intervention, the conditions under which the execution will have to be carried out and the conservation strategy/condition control level required after the intervention is made, so that the purpose of the intervention can be achieved in the most reliable way. Ease of conservation after intervention shall also be taken into account.

The selection of the optimum intervention option can operate at a number of different levels, from making decisions upon the treatment of individual members up to that of the overall structure. Thus the selection perspective may be on the basis of a particular element, a zone or the overall structure. Thus different preventative or remedial intervention methods could potentially be used in conjunction to meet the various objectives and performance goals. The steps involved might include:

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(e.g. the proposed improvement in the condition of the structure), which may or may not involve restoration back to the initial condition of the structure or fulfilment of the design requirements.

The process of choosing an intervention option may involve:

- defining the nature of the acceptance testing and verifications required;
- confirming the means for monitoring through-life performance post-intervention;
- benchmarking or comparison of the available intervention methods to establish the most appropriate (best) method for the particular circumstances and requirements.

The final published report for the CONREPNET project (EC Growth project GTC1-2001-43067) discusses the issues associated with the selection of appropriate interventions options when taking a through-life perspective.

Figure 9.7-1 shows schematically the performance of alternative intervention options. The evaluations made in this process need to consider a range of factors such as the importance of structure, the significance of non-compliance with the structure's performance requirement, and an extension of service life in future.

- performance requirements review, work analysis & work statement development;
- intervention design: goals & requirements/acceptance criteria;
- intervention design: specification development;
- execution of the selected intervention option(s);
- monitoring of the effectiveness of the intervention work.

9.7 Interventions

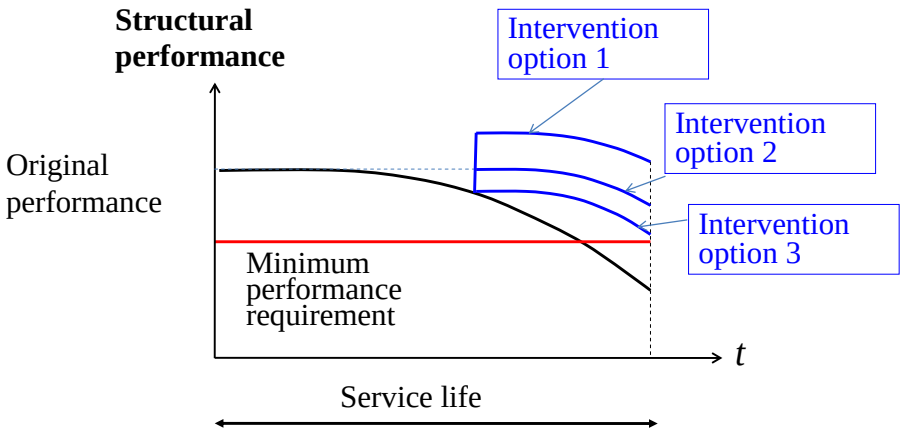
Undertaking interventions is one facet of the overall strategy for the through-life management of a structure, which necessarily involves time as well as cost in the consideration and evaluation of possible options.

In future cost may need to include issues such as the carbon burden/the environmental impacts associated with the structure and the proposed works, as well as their monetary cost (i.e. their first-cost and their whole-life cost).

Sufficient diagnostic tests shall be carried out before execution to ensure that the intervention option selected is appropriate to the circumstances and future performance requirements.

Once the most appropriate preventative or remedial intervention option(s) has been selected it is still necessary to get the work carried out. To do this the owner will usually need the support of a team of technical advisers, depending upon the expertise held by the owner. Together they should be able to address the various technical and process matters that may arise. Intervention activities need particular knowledge and experience. This group should be concerned with the performance of the repaired concrete structure. In essence what is required is an integrated team approach, wherever the necessary expertise actually resides.

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Note:
Option 1 is to upgrade the performance and to lessen the deterioration rate.
Option 2 is to restore the performance and to lessen the deterioration rate.
Option 3 is to lessen the deterioration rate only.

Figure 9.7-1: Schematic for intervention option selection

Planned interventions are typical for a proactive conservation strategy. A reactive conservation strategy may require both planned and unplanned interventions.

In the case of situations where conservation measures are not feasible, should significant damage or deterioration occur to the structure and be detected, a potentially difficult and expensive reactive or remedial intervention could be required.

For more information regarding interventions, reference may be made to the *fib* Bulletin 17: "Management, maintenance and strengthening of concrete structures – maintenance, operation and use" (*fib*, 2000) and *fib* Bulletin 44: "Concrete structure management – Guide to ownership and good practice", (*fib*, 2008).

Potential interventions can be classified based on planning as:

– planned interventions: i.e. maintenance, preventative and remedial interventions which are planned at the time of design for construction of new structures and design for repair of existing structure;
– unplanned interventions: i.e. remedial or strengthening interventions which are not planned at the time of design but become necessary due to unexpected damage or revision of performance requirements.

Appropriate steps shall also be taken to ensure the use of proper materials, methods and quality management. A plan for the execution of the intervention including details of the materials to be used, the method of undertaking the proposed works and quality control tests to ensure quality of the work, shall be drawn up before the work commences; see subclause 9.7.7.

A detailed record of the intervention work shall be maintained for future reference; see clause 9.8.

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This term is commonly applied in the context of building fabric components with a limited life (e.g. coatings, surface finishes), components associated with water management and rainwater run-off, items where regular intervention is required to maintain their effective operation by removing any build up of detritus (e.g. gutters, gulleys, drains, etc), to overcome the effects of ageing or fatigue upon materials (paints, coatings, sealants, movement joints, etc), to maintain an acceptable appearance by cleaning of surfaces etc.

There are various methods and materials used for making preventative interventions including the following:

Methods:

- cathodic prevention;
- chloride extraction;
- re-alkalization;
- electro-chemical dehumidification;
- surface coating, surface barriers, sealants, etc.;
- overlays, wearing courses, etc.

Materials:

- various materials available.

There are various methods and materials for undertaking repairs including the following:

Methods:

- restoration of affected concrete section (patch & sprayed repairs);
- restoration of reinforcement provision (replacement of reinforcing bars);
- surface coating, surface barriers, overlays, sealants, etc.

9.7.1 Maintenance interventions

Maintenance interventions involve various recurrent activities performed 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 performance at the level anticipated by the designer.

9.7.2 Preventative interventions

Preventative interventions are 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.

In particular, proactive maintenance interventions seek to utilize actions which will result in an extension of the period of initiation, prior to the onset of the propagation of deterioration, or slow down the further development of deterioration.

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.

9.7.3 Remedial interventions

Remedial interventions are undertaken to reinstate to an acceptable level the current and future performance of a structure or its components which are 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.

Remedial interventions are typically required 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,

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Materials:

– various materials available including cast and sprayed concretes and mortars, reinforced concrete, fibre reinforced concrete, polymer concrete, proprietary repair mortars, etc.

There are various methods and materials for undertaking these types of works which could include all the methods and techniques used in original construction. There is a wide range of various materials available including cast and sprayed concretes, reinforced concrete, fibre reinforced concrete, polymer concrete etc.

There are various methods and materials used for making strengthening or upgrading interventions including the following:

Methods:

– external bonding (plate or sheet bonding, over or under-laying, jacketing, etc.);

to meet the required performance levels because of deterioration (e.g. corrosion of embedded reinforcing bars). The situation may include circumstances where the performance requirements have changed over time or where the planned service life has been extended.

Remedial interventions might be instigated for the purposes of repair, rehabilitation, remediation, restoration or retrofitting of the structure concerned, or of elements thereof.

9.7.4 Rebuild, reconstruction and replacement

Rebuild, reconstruction and replacement activities are intended to provide a new structure or structural components to replace the original damaged, defective or deteriorated entity; generally without restriction upon the materials or methods employed.

Commonly this type of intervention would be undertaken to restore the performance level of the structure or specific structural components back to the level anticipated by the designer. In this case these activities would be deemed to be a remedial intervention. However, in other circumstances these works might provide the opportunity to revise the performance requirements to a higher level (i.e. above the level anticipated by the designer). In this case these activities might be deemed to be a strengthening or upgrading intervention.

Thus in some instance this approach may be considered to be an extreme form of remedial intervention, which involves the creation of a new structure or structural component to replace the original damaged, defective or deteriorated entity after its destruction or demolition. These activities require careful planning and management in order not to compromise the safety and/or performance of other parts of the structure whilst the works are carried out.

9.7.5 Strengthening or upgrading interventions

Strengthening or upgrading interventions are undertaken to improve the strength or some other performance property of a structure or structural elements (e.g. stiffness) above the level anticipated by the designer.

The revised performance requirements would be defined during the “re-design” process. Thus after these works the structure or structural elements would be expected to meet the revised performance requirements.

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– introduction of external prestressing and addition of tension cables;
– addition of member (girder, brace, support, etc.);
– increase in member cross-sectional size;
– replacement of member.

Materials:

– concrete (reinforced concrete, prestressed concrete, fibre reinforced concrete, polymer concrete, etc.);
– steel (plate, cable, bars, etc.);
– non-metallic materials (plate, sheet, cable, etc.).

This approach is only valid on the basis that the additional information gathered is able to contribute directly to the judgments to be made on the performance of the structure, including safety considerations, and the remaining design service life.

For example, where shear failure of a beam was the main concern it would not be appropriate to rely on the monitoring of flexural deflections because it is unlikely that this parameter would give adequate indication of distress or developing failure prior to a catastrophe collapse of the beam occurring.

In some situations the aesthetic appearance of a structure or building may be paramount and it may be necessary to assign this parameter a particularly high importance and to ensure that this aspect is always kept in order.

9.7.6 Other activities and measures

Intensified inspection, survey or monitoring

Intensified inspection/survey refers to inspections/surveys which may be carried out by suitably increasing one or more of the following: frequency of inspection/survey; number of inspected/surveyed items and the locations to be inspected/surveyed.

It is not appropriate to rely on an intensified inspection, survey or monitoring if the data or information gathered does not contribute directly to the decision making procedure associated with the safety management of the structure.

Usage restriction

Usage restriction is a structure management strategy which imposes some restriction on usage (e.g. restriction on the maximum traffic load or speed, maximum imposed floor load in a building, etc), so that the maximum effects of specified actions on the structure would be reduced.

The extent of restriction shall be determined depending on the level of deterioration observed or suspected and a prognosis for the (near) future.

Monitoring and/or other measures, such as detailed investigations, may also be necessary in addition to imposing the restriction.

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Plans for further action shall be drawn up on the basis of the known extent of deterioration, what further information is required in order to make a satisfactory assessment of its current condition and safety. Additional investigations may be required. Once the immediate threat and concerns have been addressed, consideration should turn to aspects of the remaining design service life, actual and desired structural performance and functionality, along with conservation strategy, tactics and any previous remedial actions taken.

See also observations made in subclause 9.4.5.

Dismantling and removal of a severely deteriorated structure is generally the last option. It shall be carried out only after taking into account of all relevant factors including hazards to the environment, public safety and disposal of the debris. The details are provided in Chapter 10. Special considerations may be required for pre-stressed concrete structures.

See also observations made in subclause 9.4.5.

The term execution is used to refer to all the physical activities undertaken for the physical completion of the intervention works. These activities include as procurement, scaffolding, falsework, formwork, reinforcement, concreting, curing, etc.; as well as the related inspection and documentation of those activities.

The certainty that the desired outcomes will be achieved can be improved by "thinking intervention" long before the intervention process actually starts on site. In this way those involved develop a process-focused approach which potentially leads not only to a reduction in problems on site and associated costs, but also to related improvements in health and safety for those involved.

Emergency measures

When a deteriorated structure poses an immediate threat to the environment, its user or any third party, suitable emergency action and temporary works shall be taken immediately. Detailed investigations may be required once the structure has been ‘stabilized’ and put into a ‘safe’ condition.

Dismantling and removal

Dismantling and removal of a severely deteriorated structure requires careful planning and evaluation before embarking upon the process as there are a wide range of potential complications which may arise.

9.7.7 Execution of interventions

Execution of interventions shall be carried out with minimum disturbance to the surrounding environment and the service condition of the structure.

A detailed project specification for the execution of the intervention works shall be developed clarifying and extending the general requirements of the ISO Execution Standard as necessary with project specific needs to meet the performance and durability requirements for the structure, together with the necessary supporting documents.

As part of the activities associated with selecting an appropriate intervention option; all intervention processes should be effectively analysed with a view to identifying significant potential problems and risks of non-conformance with the project requirements, including assessing the level of risk involved in each case.

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Quality Assurance Activities: Ensure that effective preventive actions are incorporated in relevant process control procedures at all stages of the definition and execution of the intervention so as to minimize the identified problems and/or reduce the level of risk to an acceptable value.

The process control procedures, and the preventive actions incorporated in them, must be documented so that the responsible operators and supervisors are clear not only what is to be done during the course of each process, but also how the process is to be undertaken.

Quality Control Activities: Ensure that verification procedures are prepared and implemented which define how, when, where and by whom each intervention related process will be monitored. The observations need to be recorded by appropriately qualified, experienced and independent staff dedicated to surveillance activities, with adequate time and resources to conduct the required surveillance activities.

Appendix G of *fib* Bulletin 44: "Concrete structure management - Guide to ownership and good practice", (*fib*, 2008) sets out steps for the development of supplementary documents to support the preparation of a project specification for the execution works for concrete structures, linking with the requirements of the ISO Execution Standard (ISO 22966, *Execution of concrete structures*, ISO, 2009).

Further information on quality management during execution is also given in the Model Code chapter on Life Cycle Management – see clause 3.5.

This process would be expected to draw upon information contained in the *Birth Certificate* document. There is the expectation that these activities would draw upon/contribute to the preparation of a *Re-Birth Certificate*.

The conservation records should be kept for the life of the structure. It is also important that an appropriate way is established to transfer the knowledge and understanding of the structure from the present generation of engineers/persons with responsibility for the management of structure to the following generation who will take over responsibility for its care and management.

The performance of structure after intervention is strongly affected by the quality of execution. Therefore, the execution of intervention shall be properly planned and carried out under an approved quality management plan based on the selected intervention option(s) and the requirements set down in the project specification. These activities shall be supported by appropriate Quality Assurance and Quality Control activities.

Appropriate verification tests shall be carried out as part of the execution process for quality control purposes. A detailed record of the execution shall be maintained for future reference.

After execution, an initial inspection/survey shall be carried out to examine the structural performance after intervention and to collect necessary data for condition control in future.

9.8 Recording

To allow effective and efficient Life Cycle Management, the conservation records shall be kept and preserved for future reference i.e. in the Service-Life File. Conservation records shall be included in the Life Cycle File. The conservation records for a structure shall be preserved while it remains in service.

Details of recording during conservation are given in clause 3.5, along with other information regarding Quality Management.

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10 Dismantlement

10.1 General

Structures can reach the end of their service life for different reasons:

- the structure no longer meets the demands of safety;
- the structure is no longer suitable to fulfil its function;
- the structure should be upgraded, but this is more expensive than demolishing and building another one.

In design for service life, due consideration should be given to a potential future need for refurbishment, adaptation to changing needs and dismantlement, considering re-use of parts of the structure involved. It should be noted that already in the design phase of buildings, the possibility of using the building for another function than the original one should be regarded. As an example, buildings with large span floors with residual bearing capacity, and facades allowing the entrance of ample daylight are often more suitable for a change of function than buildings with small rooms and heavy separation walls. A good conceptual design of structures can therefore prevent later dismantlement, since the structure can be re-used for another function.

In the past, structures have rarely been designed taking into account the possibility of adaptation or re-use in future for other aims. The increasing need to demolish structures has therefore led to an accelerated development of all types of demolition techniques. However, those new technologies often go along with other non-satisfactory phenomena resulting from insufficient analysis of their influence on the stability of the structure and with other negative effects for environment.

The dismantlement of large structures requires careful preparation. The aim of dismantlement should be defined with regard to its extent (partial of full), requirements for eventual re-use of the material or components, and the time available for demolition. The structure to be dismantled should be investigated with regard to its bearing structure and capacity, the materials used, its stability in the various stages of dismantlement and the effects of eventual contamination.

An appropriate method of dismantlement should be chosen, taking into account economy and boundary conditions of its eventual continued use during the (partial) dismantlement process. Measures should be taken to minimize the hinder for the environment with regard to noise, dust, water, contamination, vibration and shocks. Any stage of the dismantlement should be carefully planned with regard to the stability of the remaining part of the structure including the eventual effect of dynamic loads. The release of eventual dangerous substances and corresponding measures should get due attention. Finally a plan should be made for eventual re-use of the materials, their separation and handling, or their removal or storage.

For an eventual re-use it should be also considered that re-used materials might have a shorter service life than new materials. Maintenance might also differ between re-used and new materials.

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Dismantlement can be subdivided into two types:

- conventional dismantlement: demolition of a structure without previous demounting of structural members or structural materials;
- controlled dismantlement: systematic selective deconstruction with planned recycling. In this case separation of material fractions is strived after.

The consequence classes are connected to the levels of reliability differentiation. 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".

Especially the structural system (framework, stability by stiffening walls or cores, mixed systems) should have a significant influence on the choice of the dismantlement procedure.

Moreover the materials used have to be subject of inventory. This especially applies to materials which can be health-damaging or disturb the environment (types, masses, integration in the structure).

The history of the building has to be regarded (previous damage, for instance by fire or earthquake, and the way of repair).

10.2 Preparing dismantlement

10.2.1 General

Aspects to be considered in the planning stage of dismantlement are:

- consequence class of the structure,
- technical data,
- site conditions,
- time constraints,
- waste disposal,
- environmental protection,
- labour safety.

10.2.2 Consequence class of the structure

Structures can be subdivided into consequence classes;

- consequence class 1: low consequence for loss of human life; economic, social or environmental consequences small or negligible;
- consequence class 2: moderate consequence for loss of human life; economic, social or environmental consequences considerable;
- consequence class 3: serious consequences for loss of human life, or for economic, social or environmental concerns.

At least for the consequence classes 2 and 3 a detailed plan for dismantlement is required. In this respect the following aspects should be regarded: function of the structure, size, structural system including stability provisions, age and possible deterioration, previous repair or structural alteration, structural materials used.

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Particularly the use of parts of the structure (eventually already in the partially dismantled stage) for supporting demolition tools or equipment for dismantlement should be regarded. Especially the use of rigorous demolition techniques like break-down by explosion deserves thorough analysis and experience.

The inspection of a building in the preparation stage of dismantlement can be carried out according to Figure 10.2-1.

10.2.3 Structural analysis for dismantlement

Dismantlement is a transient situation. For the phase of dismantlement basically the same safety requirements apply as for the construction stage. By means of a structural analysis instable situations during dismantlement should be prevented. Intermediate stages have to be regarded and may require temporary measures for stabilization.

10.2.4 Investigation of potential contamination

The structure should be investigated with regard to the eventual presence of harmful substances. These substances, which have a negative effect on human health or environment, can be of organic or inorganic origin. Moreover the presence of disturbing substances should be traced. These are components of demolished material which can negatively influence the properties in case of re-use (gypsum, isolation material, foam, wood).

10.2.5 Waste disposal concept

The plan for waste disposal should regard the following aspects:

- volume of waste and quality,
- local spatial conditions,
- possibilities of regional re-use,
- possibilities for regional disposal.

10.2.6 Preparation report

The preparation phase should end with a number of conclusions:

- choice of dismantlement technique;
- measures for coping with harmful substances and disturbing material components before and during dismantlement;
- definition of necessary safety provisions;
- choice for waste disposal;
- definition of necessary capacities for transport, intermediate storage and further processing.

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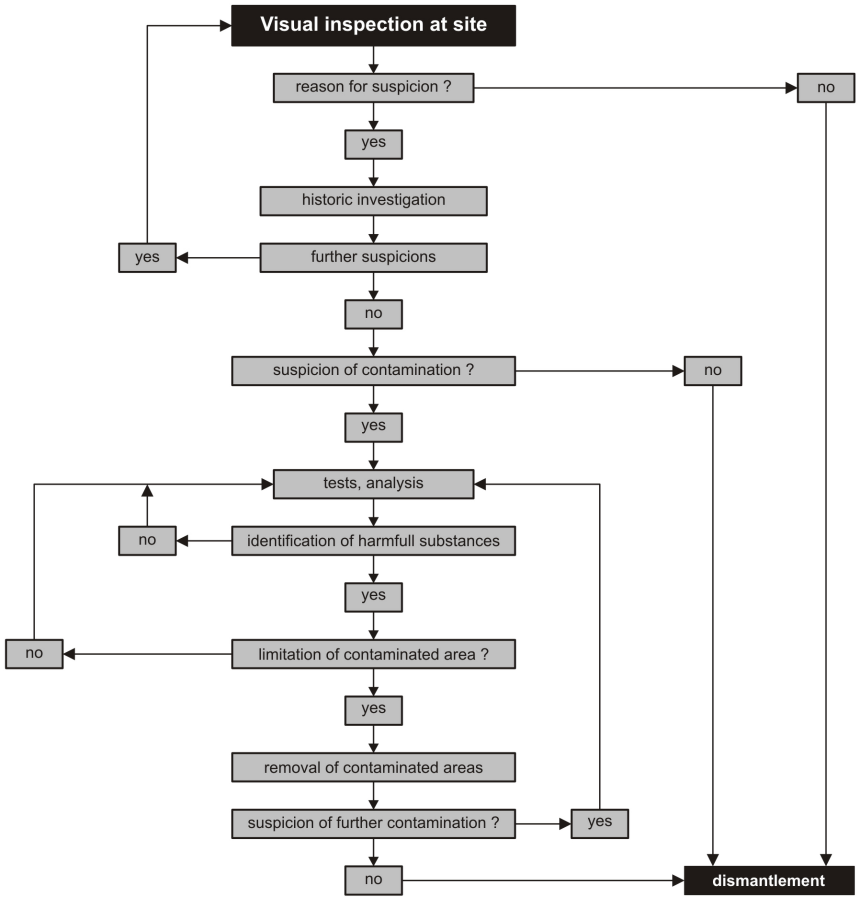


Figure 10.2-1: The inspection of a building in the preparation stage of dismantlement (C.O. Toppel, "Technische und ökonomische Bewertung verschiedener Abbruch-verfahren im Industriebau", Selbstverlag, 2003)

10.3 Safety and health provisions

A plan should be made to deal with the following safety and health aspects:

- Threats for the environment: this aspect regards protection against dynamic effects, flying material fractions, dust. This can be reached by providing various types of barriers and moistening.
- Threats of injury to persons due to: falling parts, splitters and larger scale collapse. This can be avoided by the right sequence of dismantlement and a reliable structural analysis of all intermediate stages of dismantlement. Moreover reliable scaffolding has to be used and information on the presence and position of electric ducts should be supplied.

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