INTERNATIONAL STANDARD

ISO 21725-1

First edition
2021-11

Simplified design of prestressed concrete bridges —

Part 1: I-girder bridges

Conception simplifiée des ponts en béton précontraint —

Partie 1: Ponts à poutres en I

Reference number
ISO 21725-1:2021(E)

Contents

Page

Foreword

ISO (the International Organization for Standardization) is a worldwide federation of national standards bodies (ISO member bodies). The work of preparing International Standards is normally carried out through ISO technical committees. Each member body interested in a subject for which a technical committee has been established has the right to be represented on that committee. International organizations, governmental and non-governmental, in liaison with ISO, also take part in the work. ISO collaborates closely with the International Electrotechnical Commission (IEC) on all matters of electrotechnical standardization.

The procedures used to develop this document and those intended for its further maintenance are described in the ISO/IEC Directives, Part 1. In particular, the different approval criteria needed for the different types of ISO documents should be noted. This document was drafted in accordance with the editorial rules of the ISO/IEC Directives, Part 2 (see www.iso.org/directives).

Attention is drawn to the possibility that some of the elements of this document may be the subject of patent rights. ISO shall not be held responsible for identifying any or all such patent rights. Details of any patent rights identified during the development of the document will be in the Introduction and/or on the ISO list of patent declarations received (see www.iso.org/patents).

Any trade name used in this document is information given for the convenience of users and does not constitute an endorsement.

For an explanation of the voluntary nature of standards, the meaning of ISO specific terms and expressions related to conformity assessment, as well as information about ISO's adherence to the World Trade Organization (WTO) principles in the Technical Barriers to Trade (TBT), see www.iso.org/iso/foreword.html.

This document was prepared by Technical Committee ISO/TC 71, *Concrete, reinforced concrete and pre-stressed concrete*, Subcommittee SC 5, *Simplified design standard for concrete structures*.

Any feedback or questions on this document should be directed to the user's national standards body. A complete listing of these bodies can be found at www.iso.org/members.html.

Introduction

The aim of this document is to provide rules for the design and construction of relatively short span prestressed concrete I-girder bridges. This document is developed for countries that do not have existing national standards on this subject and to offer to local regulatory authorities an alternative for the design of relatively small bridges that abound in urban overpasses and over creeks and rivers everywhere. This document may not be used in place of a national standard unless specifically considered and accepted by the national standards body or other appropriate regulatory organization. The design rules are based on simplified worldwide-accepted strength design models. This document is self-contained; therefore, loads, simplified analysis procedures and design specifications are included, as well as minimum acceptable construction practice guidelines.

The minimum dimensional guidelines contained in this document are intended to account for undesirable side effects that require more sophisticated analysis and design procedures. Material and construction guidelines are aimed at site-mixed concrete as well as ready-mixed concrete, and steel of the minimum available strength grades.

The earthquake resistance guidelines are included to account for the numerous regions of the world which lie in earthquake prone areas. The earthquake resistance for zones with high seismic hazard is based on the employment of structural concrete walls (shear walls) that limit the lateral deformations of the structure and provide for its lateral strength, in place of piers or frames that can be used in zones with intermediate, low or no significant earthquake hazard.

This document contains provisions that can be modified by the national standards body due to local design and construction requirements and practices. The specifications that can be modified are included using ["boxed values"]. The national standards body is expected to review the "boxed values" and may substitute alternative definitive values for these elements for use in the national application of this document.

Simplified design of prestressed concrete bridges —

Part 1: I-girder bridges

1 Scope

This document provides information to perform the design of the prestressed concrete I-girder bridge for road that complies with the limitations established in 6.1. The rules of design set forth in this document are simplifications of more elaborate requirements.

Designs and details for new road bridges address structural integrity by considering the following:

— the use of continuity and redundancy to provide one or more alternate paths;

— structural members and bearing seat widths that are resistant to damage or instability; and

— external protection systems to minimize the effects of reasonably conceived severe loads.

2 Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 28842, *Guidelines for simplified design of reinforced concrete bridges*

3 Terms and definitions

For the purposes of this document, the following terms and definitions apply.

ISO and IEC maintain terminological databases for use in standardization at the following addresses:

— ISO Online browsing platform: available at https://www.iso.org/obp

— IEC Electropedia: available at https://www.electropedia.org/

3.1
admixture
material other than water, aggregate, or hydraulic cement, used as an ingredient of concrete and added to concrete before or during its mixing to modify its properties

3.2
anchorage
device used to anchor a non-structural element to the structural framing

3.3
beam
horizontal, or nearly horizontal, structural member supported at one (such as a cantilever) or more points, but not throughout its length, transversely supporting a *load* (3.31), and subjected primarily to flexure

3.4
clearance
distance by which one thing clears another; the space between them

3.5
compression reinforcement
reinforcement provided to resist compression stresses induced by flexural moments acting on the member section

3.6
specified compressive strength
compressive cylinder strength of concrete used in design and evaluated in accordance with the appropriate ISO standard, expressed in megapascals (MPa)

Note 1 to entry: Whenever the quantity f_c' in under a radical sign ($\sqrt{f_c'}$), the positive square root of numerical value only is intended, and the result has units of megapascals (MPa).

3.7
confinement hook
hook (3.22) on a *stirrup* (3.46), hoop, or *crosstie* (3.11) having a bend not less than 135° with a six-diameter (but not less than 75 mm) extension that engages the *longitudinal reinforcement* (3.32) and projects into the interior of the stirrup or hoop

3.8
confinement stirrup
closed *stirrup* (3.46), *tie* (3.49) or continuously wound spiral

Note 1 to entry: A closed stirrup or tie can be made up of several reinforcement elements each having *confinement hooks* (3.7) at both ends. A continuously wound spiral should have a confinement hook at both ends.

3.9
corrosion
gradual removal or weakening of metal from its surface that requires the presence of humidity and oxygen, and is helped by the presence of other materials

3.10
cover
thickness of concrete between surface of any reinforcing bar and the nearest face of the concrete member

3.11
crosstie
continuous reinforcing bar having a 135° *hook* (3.22) at one end and a hook not less than 90° at least a six-diameter extension at the other end

Note 1 to entry: The hooks should engage peripheral longitudinal bars. The 90° hooks of two successive crossties engaging the same longitudinal bars should be alternated end for end.

3.12
deformed reinforcement
steel reinforcement that has deformations in its surface to increase its bond to the concrete

Note 1 to entry: The following steel reinforcement should be considered deformed reinforcement in this document: deformed reinforcing bars, deformed wire, welded plain wire fabric, and welded deformed wire fabric conforming to the appropriate ISO standards.

3.13
design strength
product of the *nominal strength* (3.35) multiplied by a *strength reduction factor* (3.47)

3.14
development length
length of embedded reinforcement required to develop the *design strength* (3.13) of reinforcement at a critical section

3.15
development length
<hook> shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent to the outer edge of the 90° or 180° *hook* (3.22)

3.16
duct
material creating a conduit in a concrete member to accommodate the *prestressing steel* (3.38) of a *post-tensioning* (3.37) *tendon* (3.48)

3.17
durability
characteristic of a structure to resist gradual degradation of its serviceability in a given environment for the design service life

3.18
effective depth
distance measured from extreme compression fibre to centroid of tension reinforcement

3.19
embedment length
length of embedded reinforcement provided beyond a critical section

3.20
factored loads
specified *nominal loads* (3.34) (forces) multiplied by the *load factors* (3.30) prescribed in this document

3.21
girder
main horizontal support *beam* (3.3), usually supporting other beams

3.22
hook
bend at the end of a reinforcing bar

Note 1 to entry: They are defined by the angle that the bend forms with the bar as either 90°, 180° or 135° hooks.

3.23
jacking force
temporary force in prestressed concrete, exerted by the device that introduces tension into the *tendons* (3.48)

3.24
joist
T-shaped *beam* (3.3) used in parallel series directly supporting deck *loads* (3.31), and supported in turn by larger *girders* (3.21), beams, or bearing structural concrete walls

3.25
lap splice
splice between two reinforcing bars obtained by overlapping them for a specified length

3.26
limit state
condition beyond which a structure or member becomes unfit for service and is judged either to be no longer useful for its intended function (serviceability limit state) or to be unsafe (strength limit state)

3.27
live load
static and dynamic effect, in terms of forces applied on the structure, produced by the use of the bridge by pedestrians and/or vehicles and not including construction or environmental *loads* (3.31)

3.28
load combination
combination of *factored loads* (3.20) and forces as specified in this document

3.29
load effect
force and deformation produced in structural members by the applied *loads* (3.31)

3.30
load factor
factor that accounts for deviations of the actual *load* (3.31) from the *nominal load* (3.34), for uncertainties in the analysis that transforms the load into a *load effect* (3.29), and for the probability that more than one extreme load will occur simultaneously

3.31
load
force or other action that results from the weight of all bridge materials, pedestrians, vehicles, environmental effects, differential movement, and restrained dimensional changes

3.32
longitudinal reinforcement
reinforcement that is laid parallel to the longitudinal axis of the element, generally to account for flexural effects

3.33
mesh wire
welded-wire fabric reinforcement

3.34
nominal load
magnitude of the *loads* (3.31) specified in this document (dead, live, soil, wind, snow, rain, flood, and earthquake)

3.35
nominal strength
capacity of a structure or member to resist the effects of *loads* (3.31), as determined by computations using specified material strengths and dimensions and the Formulae set forth in this document

Note 1 to entry: Specified material strengths are derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modelling effects and differences between laboratory and field conditions.

3.36
permanent load
load (3.31) in which variations over time are rare or of small magnitude

Note 1 to entry: All other loads are variable loads (see also 3.34).

3.37
post-tensioning
method of prestressing reinforced concrete in which *tendons* (3.48) are tensioned after the concrete has attained a specified minimum strength or a specified minimum age

3.38
prestressing steel
high-strength steel elements such as wire, bar, or strands used to impart prestress forces to concrete

3.39
pretensioning
method of prestressing in which *prestressing steel* (3.38) is tensioned before the concrete is placed

3.40
required factored strength
strength of a member or cross-section required to resist *factored loads* (3.20) or related internal moments and forces in such combinations as are stipulated by this document

3.41
service load
load (3.31) without *load factors* (3.30)

3.42
shrinkage and temperature reinforcement
reinforcement normal to flexural reinforcement provided for shrinkage and temperature stresses in structural solid slabs and footings where flexural reinforcement extends in one direction only

3.43
skew
difference or deviation from an expected or optimal value; in the case of bridges, deviation of the longitudinal axis of the deck with respect to a line perpendicular to the length of the abutments

3.44
slab (deck)
upper flat part of a reinforced concrete deck carried by supporting *joists* (3.24), *beams* (3.3) or *girders* (3.21)

3.45
spiral reinforcement
continuously wound reinforcement in the form of a cylindrical helix

3.46
stirrup
reinforcement used to resist shear and torsion stresses in a structural member

Note 1 to entry: Typically, bars, wires or welded-wire fabric (plain or deformed) either single leg or bent into L, U, or rectangular shapes and located perpendicular to or at an angle to *longitudinal reinforcement* (3.32). (The term "stirrups" is usually applied to lateral reinforcement in *girders* (3.21), *beams* (3.3) and *joists* (3.24). The term "ties" to those in columns and walls, perhaps because they are intended also as confinement for the longitudinal reinforcement.) See also 3.49.

3.47
strength reduction factor
coefficient that accounts for deviations of the actual strength from the *nominal strength* (3.35), according to the manner and consequences of failure

Note 1 to entry: Including the probability of understrength members due to variations in material strengths and dimensions, approximations in the design Formulae, to reflect the degree of ductility and required reliability on the member under the *load effects* (3.29) being considered, and to reflect the importance of the element in the structure.

3.48
tendon
an assembly consisting of a tensioned element (such as a wire, bar, rod, strand, or a bundle of these elements) used to impart compressive stress in concrete, along with any associated components used to enclose and anchor the tensioned element

3.49
tie
loop of reinforcing bar or wire enclosing *longitudinal reinforcement* (3.32)

Note 1 to entry: A continuously wound bar or wire in the form of a circle, rectangle, or other polygon shape without re-entrant corners is acceptable.

3.50
transfer length
length from the end of the member where the *tendon* (3.48) stress is zero to the point along the tendon where the prestress is fully effective

3.51
transverse reinforcement
reinforcement located perpendicular to the longitudinal axis of the element, comprising *stirrups* (3.46), *ties* (3.49), *spiral reinforcement* (3.45), among others

3.52
yield strength
specified minimum yield strength or yield point of reinforcement

Note 1 to entry: The yield strength is expressed in units of megapascals (MPa).

Note 2 to entry: Applicable International Standards specify that the yield strength or yield point be determined in tension.

4 Symbols and abbreviated terms

Symbol	Explanation	Unit
a	depth of equivalent uniform compressive stress block	mm
a_0	acceleration limit	g
a_{eff}	lateral dimension of the effective bearing area measured parallel to the larger dimension of the cross-section	mm
a_l	lateral dimension of the anchorage device or group of devices in the direction considered	mm
a_p	peak acceleration	g
A	maximum area of the portion of the supporting surface that is similar to the loaded area and concentric with it and does not overlap similar areas for adjacent anchorage devices	mm^2
A_a	fraction of acceleration of gravity	—
A_b	effective bearing area	mm^2
A_c	area of concrete section	mm^2
A_{cc}	area of the confined column core, in a column with spiral reinforcement, measured centre to centre of the spiral	mm^2
A_{conf}	bearing area of the confined concrete in local zone	mm^2
A_{cv}	area of concrete section resisting shear transfer	mm^2
A_g	gross area of section of element	mm^2
A_{plate}	anchor bearing plate area	mm^2
A_{ps}	area of prestressing steel	mm^2
$A_{ps,d}$	area of prestressing steel corresponding to concrete deck	mm^2
A_s	area of longitudinal tension reinforcement	mm^2
A_s'	area of longitudinal compression reinforcement	mm^2
$A_{s,min}$	minimum area of longitudinal tension reinforcement	mm^2
A_{ss}	area of spiral reinforcement	mm^2

Symbol	Explanation	Unit
A_{st}	total area of longitudinal reinforcement	mm^2
A_v	area of shear reinforcement (stirrup) within a distance s	mm^2
A_{vf}	area of interface shear reinforcement	mm^2
A_{vpc}	area of additional reinforcement across the interface shear plane	mm^2
b	width of section of the member	mm
b_{eff}	lateral dimension of the effective bearing area measured parallel to the smaller dimension of the cross-section	mm
b_f	effective width of the compression flange in a T-shaped section	mm
$b_{g,uf}$	width of upper flange of girder	mm
b_{vi}	width of interface	mm
b_w	web width of girder or beam	mm
c_i	cohesion factor at interface	MPa
d	effective depth of reinforcement	mm
d'	distance from extreme compression fiber to centroid of compression reinforcement	mm
d_b	nominal diameter of reinforcing bar or strand	mm
d_{burst}	distance from anchorage device to the centroid of the bursting force	mm
d_c	distance from extreme tension fiber to centroid of tension reinforcement	mm
d_{cc}	centre-to-centre diameter of spiral	mm
d_{ce}	one-half the effective length of the failure plane in shear and tension for a curved element	mm
d_p	effective depth of prestressing tendon	mm
d_v	distance between the centroid of the tension steel and the mid-thickness of the slab to compute a factored interface shear stress	mm
e_a	eccentricity of the anchorage device or group of devices with respect to the centroid of the cross-section; always taken as positive	mm
E	modulus of elasticity	MPa
E_b	modulus of elasticity of the bearing plate material	MPa
E_c	modulus of elasticity of concrete	MPa
E_{ci}	modulus of elasticity of concrete when post-tensioned	MPa
E_{ct}	modulus of elasticity of concrete at transfer	MPa
E_p	modulus of elasticity of prestressing steel	MPa
f_b	stress in anchor plate at a section taken at the edge of the wedge hole or holes	MPa
f_c'	specified compressive strength of concrete	MPa
$f_{c,d}'$	specified compressive strength of concrete deck	MPa
f_{cd}'	compressive strength of concrete reduced by the material factor	MPa
$f_{c,g}'$	specified compressive strength of concrete girder	MPa
f_{ci}'	specified compressive strength of concrete at time of initial loading or prestressing	MPa
f_{cr}'	average (required) compressive strength of concrete	MPa
f_{cgp}	concrete stress at the centre of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment	MPa
$f_{c,QP}$	stress in the concrete adjacent to the tendons, due to self-weight and initial prestress and other quasi-permanent actions	MPa
f_{nf}	natural frequency of floor structure	1/s
f_{pe}	effective stress in the prestressing steel after losses	MPa

Symbol	Explanation	Unit
f_{pi}	prestressing steel stress immediately prior to transfer	MPa
f_{pj}	stress in the prestressing steel at jacking	MPa
f_{ps}	average stress in prestressing steel at the time for which the nominal resistance of the member is required	MPa
f_{pu}	tensile strength of prestressing strand and bar	MPa
f_{py}	yield strength of prestressing strand and bar	MPa
f_{pyd}	yield strength of prestressing strand and bar reduced by the material factor	MPa
f_s	stress in reinforcement	MPa
f_s'	stress in compression reinforcement	MPa
f_{se}	effective stress in the prestressing steel after losses	MPa
f_y	specified yield strength of reinforcement	MPa
f_{yd}	yield strength of reinforcement reduced by the material factor	MPa
f_{ys}	specified yield strength of transverse or spiral reinforcement	MPa
F	force	kg·m/s^2
F_a	site coefficient	—
$F_{u\text{-in}}$	in-plane deviation force effect per unit length of tendon	N/mm
$F_{u\text{-out}}$	out-of-plane force effect per unit length of tendon	N/mm
g	acceleration of gravity	m/s^2
h	overall depth or thickness of the member	mm
h_d	height or thickness of deck	mm
h_f	height or thickness of flange	mm
h_l	lateral dimension of the cross-section in the direction considered	mm
H	the largest height of bridge supports	m
H_P	height of the support where forces are being evaluated	m
H_r	average annual ambient relative humidity	%
I	second moment of area	mm^4
I_c	second moment of area of concrete	mm^4
I_D	second moment of area of deck	m^4
I_P	second moment of area of wall, frame or pier	m^4
k	stiffness or spring constant	kg/s^2
K	wobble friction coefficient	/mm
l_c	longitudinal extent of confining reinforcement of the local zone	mm
l_d	development length for reinforcing bar and pretensioning strand	mm
l_{set}	Influencing distance of anchorage set	mm
l_w	horizontal length of structural concrete wall	mm
L	span length	m
L_c	length of continuous deck	m
L_T	total length	m
m	mass	kg
m_T	total mass	kg
M_n	nominal flexural moment strength at section	N·mm
M_u	factored flexural moment at section	N·mm
n	number of anchorages in a row	—
n_b	projection of base plate beyond the wedge hole or wedge plate	mm
N	number of identical prestressing tendons	—

Symbol	Explanation	Unit
p	slope of prestressing force distribution due to friction	N/mm
P_c	net force across the interface shear plane	N
P_g	part of live loads used to calculate distribution factor for live loads	N or N/mm
P_r	bearing resistance of anchorages	N
P_t	tendon force	N
R	radius of curvature of the tendon at the considered location	mm
s	centre-to-centre spacing of reinforcements	mm
s_a	centre-to-centre spacing of anchorages	mm
s_c	clear spacing of reinforcements	mm
S_a	design response spectrum	—
S_g	spacing of girders or webs	m
t	time	days or h
t_b	average thickness of the bearing plate	mm
t_r	time of prestress release	h
T	natural period	s
T_{burst}	tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis	N
v_{ui}	factored interface shear stress	MPa
V_c	contribution of the concrete to the nominal shear strength at section	N
V_n	nominal shear strength at section	N
V_{ni}	nominal shear resistance of two shear planes per unit length	N/mm
V_r	shear resistance per unit length of the concrete cover against pull-out by deviation forces	N/mm
V_s	contribution of the shear reinforcement (stirrup) to the nominal shear strength at section	N
V_T	shear force caused by thermal expansion	N
V_u	factored shear force at section	N
V_{u1}	conservative envelope value of V_u	N
V_{ui}	factored interface shear force	N
w_e	equivalent lateral force	kN/m
w_s	seismic uniformly distributed load	kN/m
W	effective weight of floor structure	kN
x	length of a prestressing tendon from the jacking end to any point under consideration	mm
z_{cp}	distance between centre of gravity of concrete section and tendons	mm
α	sum of the absolute values of angular change of prestressing steel path from jacking end, or from the nearest jacking end if tensioning is done equally at both ends, to the point under investigation	rad
α_c	coefficient of thermal expansion for concrete	/°C
α_t	angle of inclination of a tendon force with respect to the centreline of the member; positive for concentric tendons or if the anchor force points toward the centroid of the section; negative if the anchor force points away from the centroid of the section	rad
β	modal damping ratio	—
γ	load factor	—
γ_{mc}	material factor for concrete	—
γ_{ms}	material factor for steel	—

Symbol	Explanation	Unit
δ_T	longitudinal change due to thermal expansion	mm
Δ	displacement	m
Δf_{pA}	prestress loss due to anchorage set	MPa
Δf_{pES}	sum of all prestress losses or gains due to elastic shortening or extension at the time of application of prestress and/or external loads	MPa
Δf_{pF}	prestress loss due to friction	MPa
Δf_{pLT}	prestress losses due to long-term shrinkage and creep of concrete, and relaxation of the steel	MPa
Δf_{pR}	prestress loss due to relaxation of a strand	MPa
Δf_{pT}	total loss of prestress	MPa
Δl	anchorage set	mm
Δ_T	temperature variation	°C
ε_{cs}	shrinkage strain	m/m
ε_{csu}	ultimate shrinkage strain	m/m
μ	curvature friction coefficient	/rad
μ_i	friction factor at interface	—
ρ	ratio of longitudinal tension reinforcement	—
ρ'	ratio of longitudinal compression reinforcement	—
ρ_h	ratio of horizontal reinforcement in structural concrete walls	—
ρ_{max}	maximum permissible ratio of longitudinal flexural tension reinforcement	—
ρ_{min}	minimum permissible ratio of longitudinal flexural tension reinforcement	—
ρ_t	ratio of total longitudinal reinforcement area	—
ρ_v	ratio of vertical reinforcement in structural concrete walls	—
ϕ	strength reduction factor	—
$\varphi(t,t_0)$	creep coefficient	—
φ_u	ultimate creep coefficient	—
ω	angular frequency	rad/s

5 Design and construction procedure

5.1 Procedure

The design procedure comprises the following steps (see Figure 1).

a) Step A: preliminary design of structure

Definition of the layout in plan and height of the structure, according to the provisions of Clause 7. Verify that the limitations of 6.1 are met.

b) Step B: definition of loads

Calculation of all gravity loads that act on the structure using the provisions of Clause 8, excluding the self-weight of the structural elements.

c) Step C: definition of an appropriate superstructure system, depending on the span lengths and the magnitude of the gravity loads.

d) Step D: design of slab (deck)

Trial dimensions for the slab (deck) of the superstructure system Calculation of the self-weight of the system, and design of the elements that comprise it, correcting the dimension as required by the ultimate and serviceability limit states.

e) Step E: design of beams and girders

Trial dimensions for the beams and girders and calculation of their self-weight. Flexural and shear design of the beams and girders, correcting the dimension as required by the ultimate and serviceability limit states.

f) Step F: design of substructure

Trial dimensions for the substructure system such as column/pier with pier cap and calculation of its self-weight. Elements slenderness verification and design for combination of axial load and moment, and shear, correcting the dimension as required by the strength and serviceability limit states.

g) Step G: definition of lateral forces

If lateral loads such as earthquake, wind, or lateral earth pressure exist, their magnitude is established using the provisions in Clause 8. Otherwise, the designer should proceed to Step I.

h) Step H: design of foundations

The loads at the foundation level are determined, and a definition of the foundation system is performed. The structural elements of the foundation are designed.

i) Step I: design of structural walls

Preliminary location and trial dimensions for structural concrete walls capable of resisting the lateral loads established. The influence of their self-weight is evaluated, and flexural and shear design of the structural concrete walls is performed.

j) Step J: production of structural drawings

k) Step K: construction of the structure (in line with local construction and practice)

Step A: preliminary design of structure

Step B: definition of loads

Step C: definition of superstructure system

Step D: design of slab (deck)

Step E: design of beams and girders

Step F: design of substructure

Step G: definition of lateral forces

Step H: design of foundations

Step I: design of structural walls

Step J: structural drawings

Step K: construction of structure

Figure 1 — Design and construction procedure

5.2 Design documentation

5.2.1 General

The design steps should be fully recorded in the following documents.

5.2.2 Calculation report

The structural designer should document all design steps in a calculation report. This report should contain, as a minimum:

a) the general structural requirements of the project, as required by 7.1;

b) a description of the structural system employed;

c) loads and assumptions employed;

d) grade, strength and fabrication standards for all structural materials;

e) presentation of all design computations; and

f) sketches of the reinforcement layout for all structural elements.

5.2.3 Geotechnical report

The geotechnical report should record, as a minimum:

a) the soil investigation performed;

b) the definition of the allowable bearing capacity of the bearing soil; and

c) the lateral soil pressures required for design of any soil retaining structure, side friction, end bearing capacity, and lateral soil stiffness required for design of deep foundation and all other information.

5.2.4 Structural drawings

All the drawings required for construction of the bridge.

5.2.5 Specifications

The construction specifications required.

6 General provisions

6.1 Limitations

6.1.1 General

This document should be employed only when the bridge being designed complies with all the limitations set forth in 6.1.2 to 6.1.14.

6.1.2 Permitted use

Design of bridges of mixed use according to this document should be permitted, but restricted to pedestrian and vehicular traffic. Bridges for trains are beyond the scope of this document.

6.1.3 Maximum number of spans

The maximum number of spans for a bridge designed according to this document should be as per Table 1.

Table 1 — Maximum allowed number of spans

	Number of spans
Bridge over water	[3]
Bridge for road overpass	[3]

6.1.4 Maximum span length

The maximum span length allowed is [60] m for each span.

6.1.5 Maximum difference in span length

Span should be approximately equal, with the larger of two adjacent spans not greater than the shorter by more than the percentage specified in Table 2.

Table 2 — Maximum difference in two consecutive span lengths

Total number of spans	Length difference, %
[2]	20
[3]	15

6.1.6 Maximum cantilever length

The maximum clear cantilever length for girders and beams should not exceed 3 m or 33 % of the length of the first adjacent interior span, whichever is smaller, in order to avoid cantilevers too long for the purposes of this document, as greater lengths can require detailed structural analysis to verify serviceability conditions such as deformation, vibration and fatigue, among other criteria.

Continuous deck slab cantilevers over intermediate beam supports may have lengths up to 63 % of the length of the adjacent span.

6.1.7 Maximum height of bridge

The height of the bridge supports, including abutments and piers, shall not exceed the values given in Table 3, according to seismic hazard level or wind intensity. The difference between the various supports heights for the same bridge shall not exceed 20 %, as more detailed analysis would be required to assess the impact of such differences on stiffness and force distribution.

Table 3 — Maximum allowable support height

Seismic hazard level or wind intensity	Low	Intermediate	High
Maximum height, m	[15]	[12]	[10]

6.1.8 Maximum number of lanes

The maximum number of lanes for a vehicular bridge designed according to this document should be [4]. Up to [2] sidewalks may be considered in addition to the vehicle traffic lanes.

6.1.9 Width limitations

Pedestrian bridges should not have widths of less than [1,5] m.

Vehicular bridges should not have roadways with widths, excluding sidewalks, of less than [3] m or in excess of [14] m. Sidewalk should comply with a minimum width of [1,7] m.

6.1.10 Clearances

6.1.10.1 General

The horizontal clearance shall be the clear width, and vertical clearance the clear height for the passage of vehicular traffic.

The roadway width shall generally equal the width of the approach roadway section including shoulders. Where curbed roadway sections approach a structure, the same section shall be carried across the structure.

6.1.10.2 General clearances

6.1.10.2.1 Vertical clearance

Vertical clearance shall not be less than [5,5] m over the entire roadway width with an allowance of [0,3] m for resurfacing.

6.1.10.2.2 Horizontal clearance

Horizontal clearance shall be at least the dimension of the approach roadway width, including curbs where necessary.

6.1.10.3 Clearances for underpasses

6.1.10.3.1 Width

The pier columns or walls for grade spacing structures shall generally be located a minimum of [9] m from the edges of the through traffic lanes. Where the practical limits of structure costs, type of structure, volume and design speed of through traffic, span arrangement, skew, and terrain make [9] m offset impractical, the pier may be placed closer than [9] m and protected by the use of guardrail or other barrier devices. The guardrail or other device shall be independently supported with the roadway face at least [0,7] m from the face of pier or abutment.

The face of the guardrail or other device shall be at least [0,7] m outside the normal shoulder line.

6.1.10.3.2 Vertical clearance

A vertical clearance of not less than [5,5] m shall be provided between curbs. If curbs are not used, it shall be provided over the entire width that is available for traffic.

6.1.10.4 Clearances for depressed roadways

6.1.10.4.1 Clearance between walls

The minimum width between walls for depressed roadways carrying two lanes of traffic shall be [9] m.

6.1.10.4.2 Curbs

Curbs, if used, shall match those of the approach roadway section.

6.1.11 Maximum skew angle

Bridges designed according to this document should have a low skew angle, not exceeding [15]°.

6.1.12 Maximum bridge horizontal curvature

Bridges designed according to this document should have a maximum length to horizontal curvature radius of [4] %.

6.1.13 Cross-section variation

Bridges designed according to this document should have a constant depth along the continuous portions of the bridge.

6.1.14 Interaction between superstructure and substructure

No framing effect is permitted along the longitudinal axis of the bridge. No direct transmission of moments shall be allowed from the bridge deck to the columns, piers, abutments or to any other element that carries the loads to the ground, due to gravity and to other loads longitudinal effects. The support at one of the abutments should allow movement in the deck longitudinal direction.

6.2 Limit states

6.2.1 General

The design approach of the present document is based on limit states, where a limit state is a condition beyond which a structure or member becomes unfit for service and is judged either to be no longer useful for its intended function or to be unsafe.

The following limit states are considered implicitly in the design procedure:

— structural integrity limit state;

— lateral load drift limit state;

— longitudinal drift limit state;

— durability limit state;

— fire limit state; and

— fatigue limit state.

Ultimate and serviceability limit states are to be verified through the different stages of design using the document.

6.2.2 Deflection serviceability verification

6.2.2.1 Vehicular bridges

The deflection of the deck or slab structure shall be less than $L/700$.

6.2.2.2 Pedestrian bridges

A simplified design criterion for the resonance response is given by Formula (1):

$$\frac{a_p}{g} = \frac{P_0 \exp(-0{,}35 f_{nf})}{\beta W} \leq \frac{a_0}{g} \tag{1}$$

where

$\frac{a_p}{g}$ is the estimated peak acceleration (in unit of g);

$\frac{a_0}{g}$ is the acceleration limit, which is equal to 5 % of gravity;

f_{nf} is the natural frequency of deck or slab structure;

P_0 is the constant force equal to 0,29 kN for deck or slab and 0,41 kN for footbridges;

W is the effective weight of deck or slab structure; and

β is the modal damping ratio. The natural frequency of deck or slab structure shall be greater than 3 Hz.

The natural period is given by Formula (2):

$$T = \frac{2\pi}{\omega} = 2\pi\sqrt{\frac{m}{k}} \tag{2}$$

The natural frequency is the inverse of the natural period as per Formula (3).

$$f_{\mathrm{nf}} = \frac{1}{T} = \frac{1}{2\pi}\sqrt{\frac{k}{m}} \tag{3}$$

Taking into account that $k = F/\Delta$ and $m = F/g$, Formula (4) is derived:

$$f_{\mathrm{nf}} = \frac{1}{2\pi}\sqrt{\frac{Fg}{F\Delta}} \cong 0{,}16\sqrt{\frac{9{,}81}{\Delta}} \tag{4}$$

Δ may be calculated as per Annex B. Δ should be interpreted as per Table 4 depending on the number of spans.

Table 4 — Application of Δ in Formula (4)

1 span	2 and 3 continuous spans
Δ	(0,40~0,50)Δ

6.3 Ultimate limit state design format

6.3.1 General

The ultimate limit state corresponds to the condition when one or more parts of the structure reach a point where they are incapable of carrying any additional loads. Therefore, for the ultimate limit state design the structure and the structural members should be designed to have design strength at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in this document.

The basic requirement for ultimate limit state should be as per Formula (5):

$$R \geq S \tag{5}$$

where

S is the load effect;

R is the resistance.

To allow for the possibility that the resistances may be less than computed, and the load effects may be larger than computed, material factors are to be used to reduce material strength and load factors, γ, generally greater than one, should be employed. Ultimate resistant force is obtained by reducing the specified yield strength for steel or reducing the specified compressive strength for concrete, or both, by means of dividing these values by the corresponding material factors [see Formula (6)]:

$$R = f\left(\frac{f_{\mathrm{c}}'}{\gamma_{\mathrm{mc}}}, \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{ms}}}\right) \geq \gamma_1 S_1 + \gamma_2 S_2 + \gamma_3 S_3 + \cdots \tag{6}$$

R stands for strength and S stands for load effects based on the nominal loads prescribed by this document. Therefore, the ultimate limit state design format requires that [see Formulae (7) and (8)]:

$$R_{\mathrm{d}} \geq U \tag{7}$$

where

R_d is the design strength;

U is the required factored strength.

or

$$R_d = f\left(\frac{f_c'}{\gamma_{mc}}, \frac{f_y}{\gamma_{ms}}\right) \geq U \tag{8}$$

where the required factored strength is $U = \gamma_1 S_1 + \gamma_2 S_2 + ...$

6.3.2 Required factored loads

The required factored load, U, should be computed by multiplying service loads, or forces, by load factors using the load factors and combinations in 8.10.1.

6.3.3 Design strength

The design strength provided by a member, its connections to other members, and its cross-sections, is then identified by the subindex r, and should be taken as the strength calculated in accordance with the requirements and assumptions for each particular force effect in each of the element types at the critical sections defined by this document, based on the limit stress reduced according to each corresponding material as per Table 5:

Table 5 — Material factor

Material	Factor
Concrete, γ_{mc}	[1,5]
Steel, γ_{ms}	[1,15]

A more detailed explanation on the material factor is provided in Annex A.

6.4 Serviceability limit state design format

Serviceability limit states correspond to conditions beyond which specified performance requirements for the structure, or the structural elements, are no longer met. Compliance with the serviceability limit state in this document should be obtained indirectly through the observance of the limiting dimensions, cover, detailing and construction requirements. For bridges, these serviceability conditions include effects such as:

— permanent deformations, either of the structure or its foundations, that cause public concern or make the structure unfit for use;

— dynamic movements that cause discomfort or public concern;

— dynamic movements that cause damage to non-structural elements such as railings;

— damage by scour;

— flooding or scour of adjacent properties; and

— damage due to corrosion that is sufficient to cause significant reduction in the strength of the structure or in its service life.

On the other hand, fatigue can be separately considered in the fatigue limit state design format.

7 Structural systems and layout

7.1 Description of the components of the structure

7.1.1 General

For the purposes of this document, the bridge structure should be divided in the following components:

7.1.2 Superstructure system

The superstructure or deck system consists of the structural elements that comprise the portion of the bridge that directly receive the live load. The superstructure system includes the girders, beams, joists (if employed) and the slab (deck) that spans between them, or the slab, when it is directly supported on piers, columns or walls. The superstructure should also act as a diaphragm that carries through its plane the lateral loads from their point of application to the vertical elements of the lateral load resisting system.

7.1.3 Substructure system

The substructure system holds up the superstructure and carries the accumulated gravity loads all the way down to the foundation of the structure. The substructure acts also as the lateral load resisting system supporting and transmitting to the ground the lateral loads arising from earthquake motions, wind and lateral earth pressure. The vertical elements of the lateral load resisting system collect the forces arising from the superstructure and carry them down to the foundation, and through the foundation to the underlying soil. According to this document, the main vertical elements of the substructure system should be cantilever piers, frames or structural concrete walls, and are described in Clause 14.

7.1.4 Foundation

The foundation comprises all structural elements that:

— serve to transmit loads from the structure to the underlying supporting soil;

— are in contact with the soil; or

— serve to contain it.

It includes elements such as spread footings, combined footings, foundation mats, retaining walls, grade beams and deep foundations, such as piles and caissons, and their pile footings and caps among others.

7.2 General guide

7.2.1 Architectural guide

It is advisable that an architect, an urban planner and a landscaper are involved in the project, but it is not mandatory. In any case, a general architectural guide of the bridge should be coordinated between the owner and the structural designer before actual structural design begins, even if no architect is part of the project.

The general architectural guide should be based on the following design aspects:

— location;

— alignment;

— roadway characteristics and details, bordering conditions;

— vistas and scenery;

— presence of open space and manufactured complexes; and

— environmental and visual impact.

7.2.2 General structural guides for the project

Based on the general architectural guide information, the structural designer should define the general structural guides for the structure being designed according to this document. These general structural guides should include, at least, the following items:

— intended use for the bridge;

— nominal loads related to the use of the bridge;

— special loads required by the owner or competent authorities;

— design earthquake motions, if the bridge is located in a seismic zone;

— wind requirements for the site;

— requirements for rain, hail, ice and snow consideration;

— site information related to slopes and site drainage;

— allowable soil bearing capacity, and recommended foundation system derived from the geotechnical investigation, and additional restrictions related to expected soil settlements;

— environmental requirements derived from local seasonal and daily temperature variations, humidity, presence of deleterious chemicals and salts;

— availability, type, and quality of materials such as reinforcing steel, cement and aggregates;

— availability of materials for formwork erection;

— availability of testing facilities for concrete mix design and quality control during construction; and

— availability of qualified workforce.

7.3 Structural layout

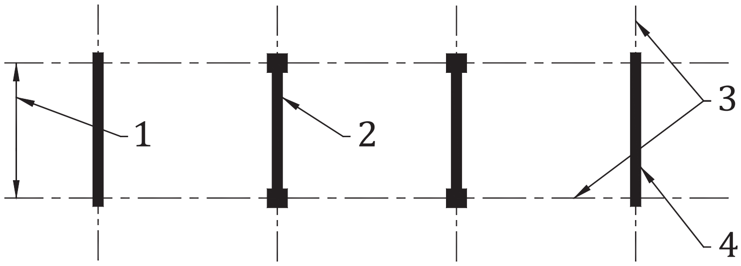
7.3.1 General structural layout

The structural designer should define a general structural layout in plan (see Figure 2). The general structural layout in plan should include:

— dimensioned grid for axes, or centrelines, in both principal directions in plan. These axes should intersect at the location of the vertical supporting elements (columns, piers, structural concrete walls, and abutments);

— location in plan for all vertical supporting elements. These vertical supporting elements should be aligned vertically, and should be continuous all the way down to the foundation; and

— horizontal distance between centrelines, S, which corresponds to the centre-to-centre span lengths, and horizontal distance, B, which corresponds to the centre-to-centre breadth, of the superstructure system.



Key

1 deck width
2 pier, frame or wall
3 centrelines grid
4 abutment

Figure 2 — General structural layout in plan

7.3.2 Vertical layout

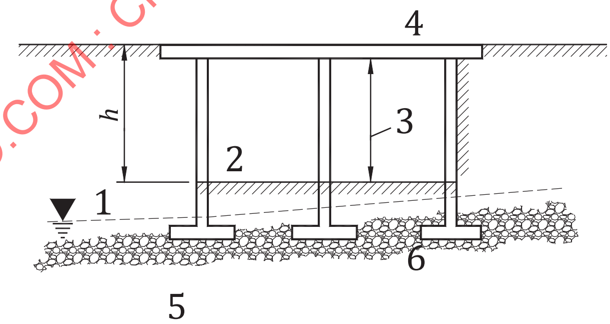
The structural designer should define a general structural vertical layout (see Figure 3). This vertical layout should include all relevant information in height of the structure, including:

— abutments, piers, frames or columns height, defined as the vertical distance from superstructure finish to the ground;

— slope and shape of the terrain;

— vertical clearance from roadway to superstructure lowermost surface, as specified by this document or local highway specifications, whichever is larger; and

— supporting soil stratum depth and water table depth.



Key

1 water table
2 roadway
3 clearance
4 overpass
5 soil
6 bearing soil stratum

Figure 3 — Vertical layout of the bridge

7.3.3 Cross beams

Cross beams that connect the girders shall be placed at supports. Other cross beams, e.g. at mid-span and quarter-spans, can be placed by approval of a chief engineer. In this case, effectiveness of the additional cross beams should be verified.

7.4 Feasibility under the document

Based on the layout information, the structural designer should verify the feasibility of performing the structural design according to this document. Compliance with the following limitations should be verified:

— the use of the bridge should be within the accepted uses of 6.1.2;

— the number of spans should not exceed the maximum permissible, given in 6.1.3;

— the span lengths should be within maximum lengths prescribed in 6.1.4;

— the difference between adjacent spans should not exceed the limit of 6.1.5;

— cantilever lengths should be within maximum lengths prescribed in 6.1.6;

— the height of the tallest support, measured from ground to superstructure finish, should not exceed the maximum permissible height given in 6.1.7, nor the difference between supports heights should exceed the limits given there;

— the number of lanes should not exceed the maximum permissible, given in 6.1.8;

— pedestrian bridge decks and vehicular roadways should comply with width limitations given in 6.1.9;

— bridge clearances shall be specified according to 6.1.10;

— bridge skew angle for girders and deck should not exceed the limit given in 6.1.11;

— bridge length to horizontal curvature ratio should not exceed the limit given in 6.1.12; and

— cross-section variation along bridge length shall comply with 6.1.13.

8 Actions (Loads)

8.1 General

This clause provides minimum load guides for the design of bridges according to this document. Loads and the appropriate load combinations should be used together.

Loads and forces explicitly considered in bridge design according to this document are:

— dead loads;

— live loads (static and dynamic effects);

— longitudinal forces;

— earth pressure;

— wind loads;

— earthquake inertial forces; and

— jacking and post-tensioning forces.

Loads and forces implicitly considered are:

— thermal forces;

— shrinkage forces;

— skew stress effects;

— elastomeric bearings shear resistance; and

— settlement of the ground.

8.2 Dead loads

8.2.1 General

Bridge dead loads comprise the total weight of the structure, calculated as the sum of the weights of all structural and non-structural elements, including substructure elements, superstructure elements, deck surface, median permanent or removable structures, sidewalks, railings, and all other elements supported by the bridge like public utility services and ducts.

8.2.2 Structural elements

Dead loads due to structural elements, referred to as self-weight, may be calculated as the sum of their weight, assuming the density of normal weight concrete as [2 500] kg/m^3. The use of lower values for normal concrete density shall be accompanied by supporting documents demonstrating that the value used does not reflect an average value, but rather a [95] percentile value for a normal distribution record of representative field data.

8.2.3 Non-structural elements

Dead loads due to non-structural elements may be calculated as the sum of their weights according to the density of their constitutive materials or to those specified by the producer in their technical data. Density values shown in Table 6 may be used for weight estimate.

Table 6 — Density values for materials used in bridge construction

Material	Density kg/m^3
Steel	[7 900]
Timber	[800]
Reinforced concrete	[2 500]
Prestressed concrete	[2 500]
Compacted filling soil	[1 900]
Loose filling soil	[1 600]
Stone masonry	[2 700]
Concrete masonry	[2 300]
Clay masonry	[1 400]
Asphalt	[1 800]

8.3 Live loads

8.3.1 General

Bridge live loads comprise the weights of all loads that might be applied to the superstructure according to the bridge use. Vehicular live loading on the roadways of bridges shall consist of a combination of design truck and design lane load.

Each design lane shall be occupied by design truck coincident with the lane load. The loads shall be assumed to occupy 3 000 mm transversely within a design lane.

8.3.2 Design truck

The weights and spacing of axles and wheels for the design truck shall be as specified in Figure 4. A dynamic load allowance shall be considered. The spacing between the two 145 000 N axles shall be varied between 4 300 and 9 000 mm to produce extreme force effects.

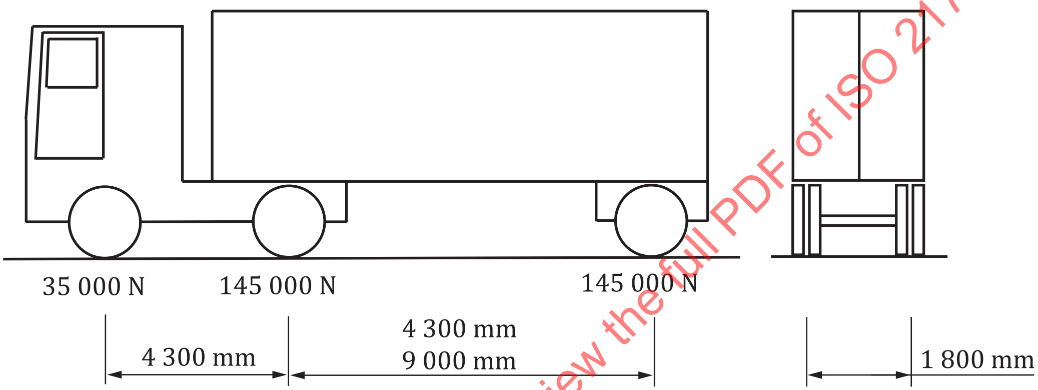


Figure 4 — Characteristics of design truck

8.3.3 Design lane load

The design lane load shall consist of a load of 9,3 N/mm uniformly distributed in the longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over a 3 000 mm width. The force effects from the design lane load shall not be subject to a dynamic load allowance.

8.3.4 Pedestrian bridges

Pedestrian live loads of 5 kN/m^2 should be applied on the deck walkable area, as to cause the most unfavourable effects. Additionally, one truck load should be considered to account for maintenance equipment, unless the bridge width is less than 2 m or vehicle entrance is prevented by permanent barriers. The truck load, shown in Figure 5 and Table 7 should not be applied simultaneously with the distributed pedestrian load.

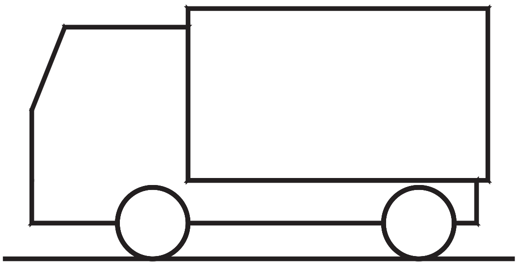


Figure 5 — Pedestrian bridge truck

Table 7 — Truck loads

Walkable width m	Q_1 kN	Q_2 kN
2 to 3	[10]	[40]
More than 3	[20]	[80]

8.3.5 Dynamic effect of live loads

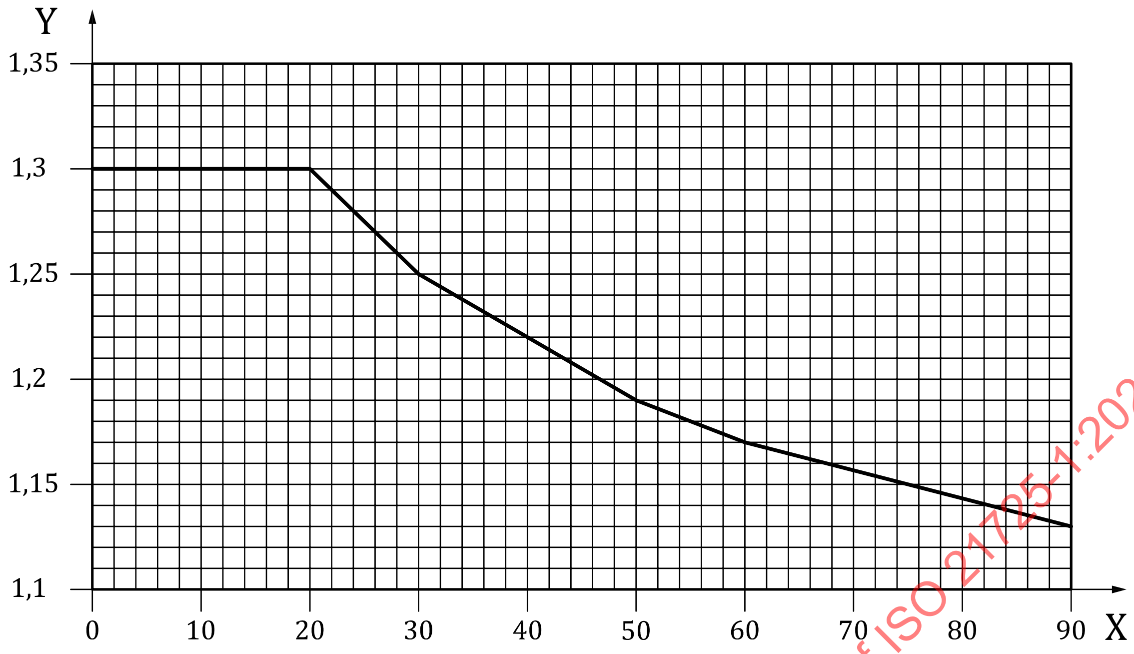
To account for the dynamic effects, such as impacts due to deck surface irregularities, vehicular live loads should be increased by the dimensionless factor given in Figure 6 according to the loaded area as to produce the most unfavourable effect on each element.

— for shear design, always increase the live load by 1,3;

— for slab and superstructure joints design, increase the live load by 1,2;

— for local analysis (joists, slabs, etc.), increase the live load by 1,2; and

— pedestrian live loads need not be increased.



Key

Y live load dynamic effect factor

X loaded length, m

Figure 6 — Live load dynamic effect factor

8.4 Longitudinal forces

Axial loads and moments due to traffic should also be considered as applied longitudinally, and within the plane of the deck, on the superstructure, without the dynamic effect increase. Axial loads should be taken as [5] % of live loads. Moments should be calculated using a lever arm of [2] m.

Only superstructure axial loads are transmitted to the substructure.

8.5 Earth pressure

Forces due to earth pressure acting on abutments, or on retaining walls that are part of the bridge substructure, should be calculated and applied adequately to substructure elements.

Earth pressure should not be taken as less than an equivalent fluid weight of [5] kN/m^3.

8.6 Wind loads

Wind loads on bridges complying with the limitation set forth in 6.1 do not control the structure's design and need not be taken into account, except in regions prone to hurricane, cyclone or typhoon winds, where a wind load case needs to be taken into account as per Table 8.

Table 8 — Wind loads for hurricane, cyclone or typhoon prone areas

Load condition	Load direction	Load kN/m^2
Load on structure[a]	Transverse	[2,5]
	Longitudinal	[0,6]

[a] Both longitudinal and transverse loads should be applied simultaneously.

Table 8 *(continued)*

Load condition	**Load direction**	**Load** kN/m^2
Load on live load[a]	Transverse	[1,5]
	Longitudinal	[0,4]
[a] Both longitudinal and transverse loads should be applied simultaneously.		

8.7 Earthquake inertial forces

8.7.1 General

Inertial forces due to earthquakes depend on the mass of the structure and on the structural response to ground acceleration which, in turn, is a function of the seismic hazard and of the soil characteristics at the site of the bridge.

The corresponding national standard can provide requirements for calculating the mass of bridge building materials. When no national standard is available, the requirements of ISO 9194 may be used. Table 6 may also be used to determine bridge masses.

For bridges designed according to this document, an equivalent lateral force applied directly to the substructure and superstructure elements may be employed to represent the dynamic response of the structure to the ground acceleration.

8.7.2 Seismic hazard

A level of seismic hazard should be defined for the bridge in terms of the intensity of the effective peak ground horizontal acceleration in rock at the structure site. The peak rock acceleration is calculated as the median spectral acceleration for one degree of freedom systems, with short periods of structural vibration, i.e. periods not exceeding 0,15 s, denoted as A_a, and usually expressed as a fraction of the acceleration of gravity, g (acceleration of gravity may be taken as 9,81 m/s^2).

The values for A_a can be taken from the applicable corresponding national standard. When the national code defines the maximum seismic ground motion for each considered site based on spectral response accelerations at 5 % of critical damping, S_s, A_a may be estimated as the value of S_s for a period of 0,15 s, divided by 375 ($A_a = S_s / 375$). When the national code defines the maximum seismic ground motion for each considered site based on a seismic zone factor Z, the value of A_a should be taken equal to Z. When no national code exists for the site of the bridge being considered, A_a may be estimated from the seismic hazard maps shown in Figure 7.

8.7.3 No seismic hazard zones:

A zone of the world where the value of the peak rock acceleration, A_a, expressed as a fraction of the acceleration of gravity, is estimated as less or equal to [0,05], may be deemed as a *no seismic hazard* zone.

8.7.4 Low seismic hazard zones:

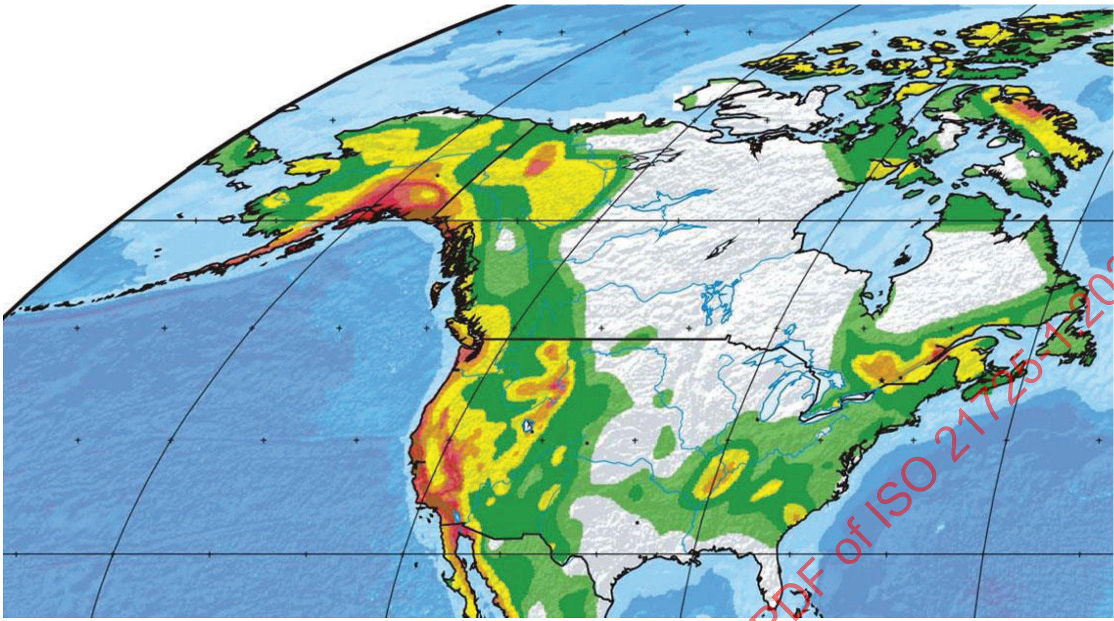
A zone where the value of A_a is estimated as more than [0,05] but less or equal to [0,1] may be deemed as a *low seismic hazard* zone.

8.7.5 Intermediate seismic hazard zones:

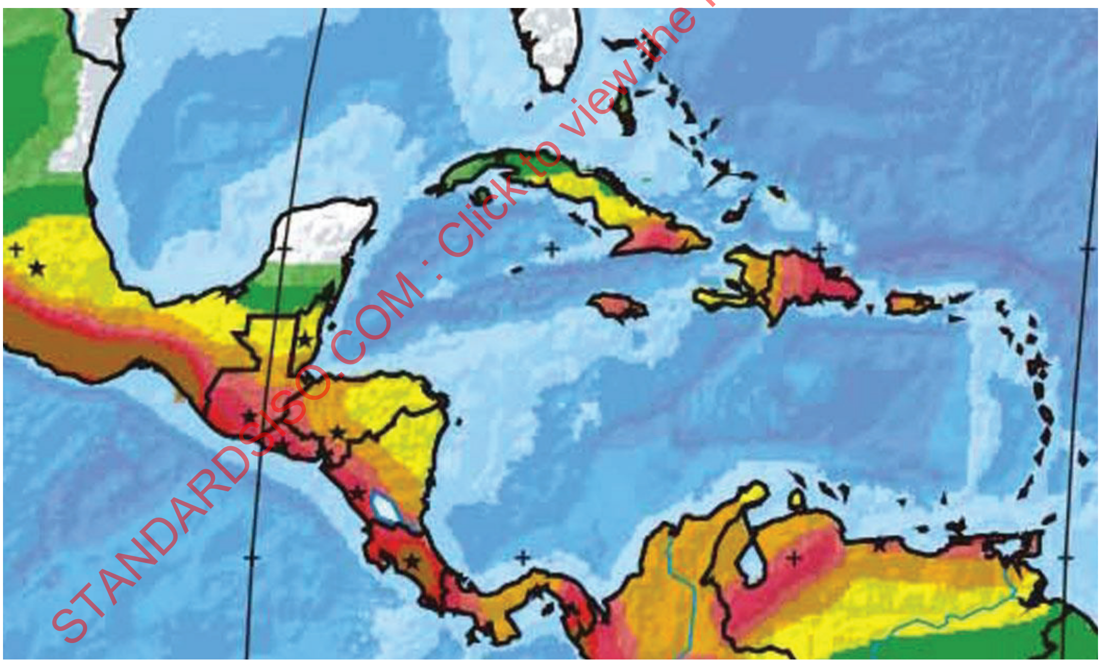
A zone where the value of A_a is estimated as more than [0,1] but less or equal to [0,2] may be deemed as an *intermediate seismic hazard* zone.

8.7.6 High seismic hazard zones:

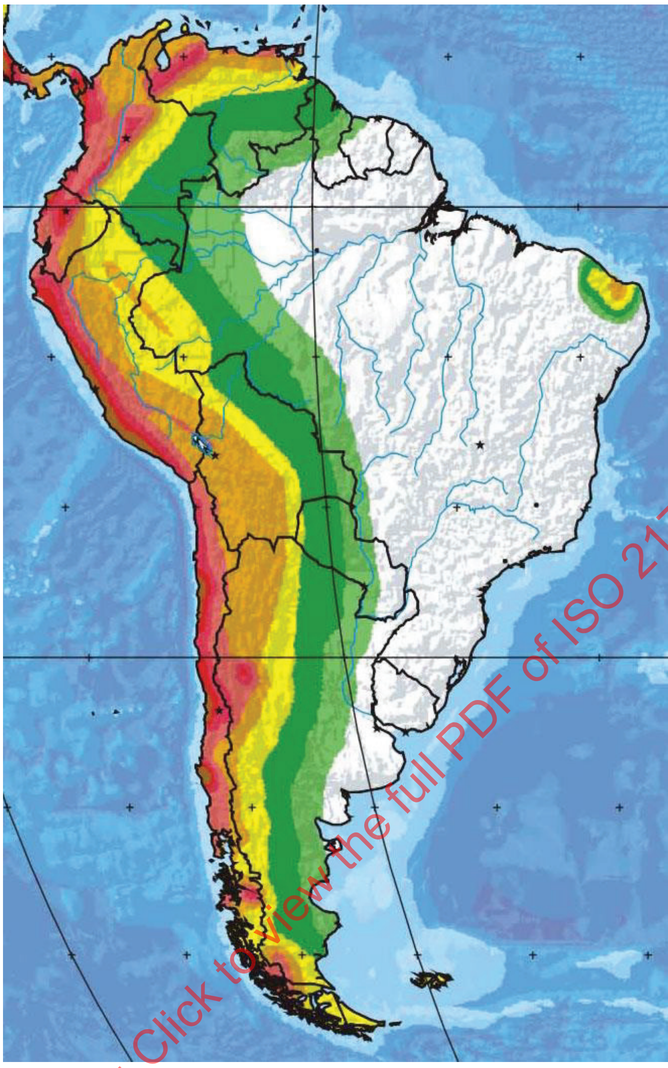
A zone where the estimated value of A_a exceeds [0,2] may be deemed as a *high seismic hazard* zone.



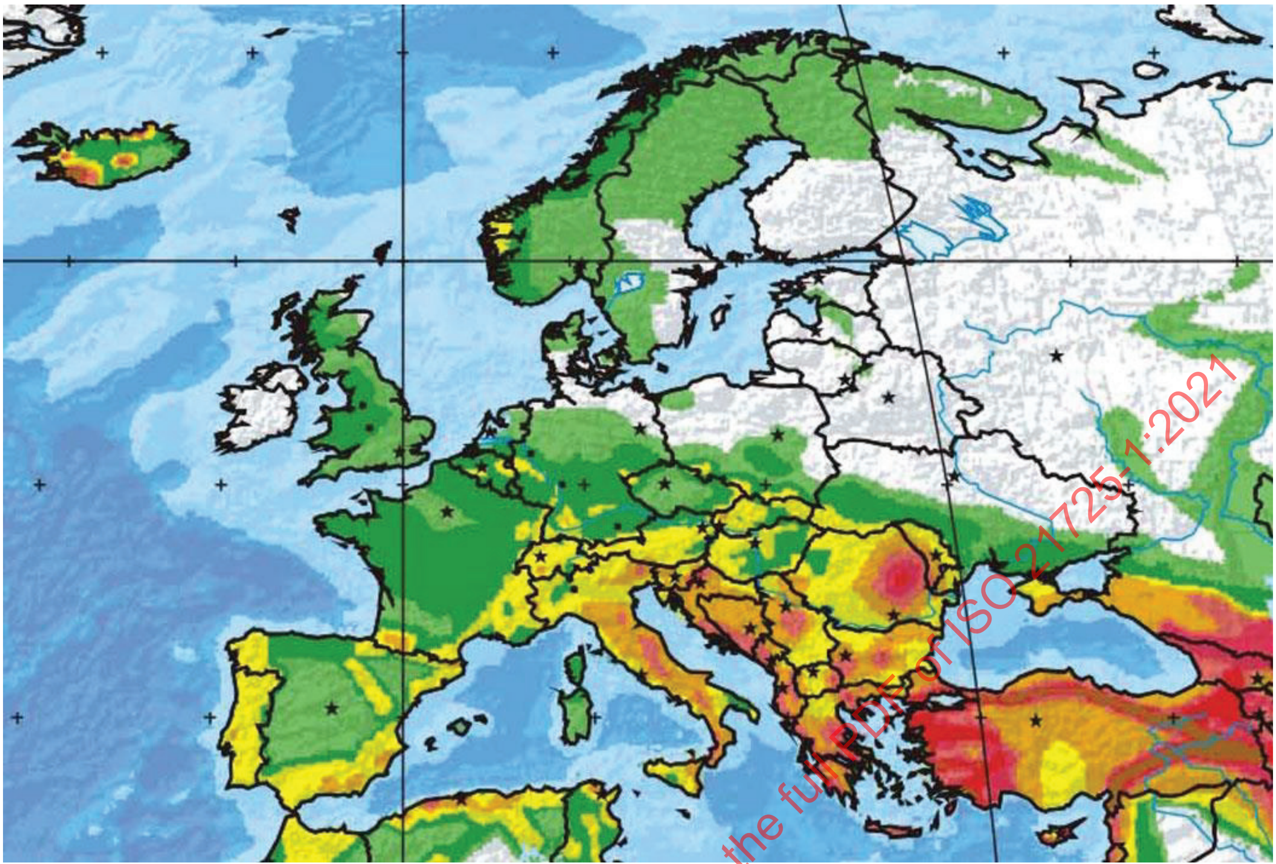
a) North America



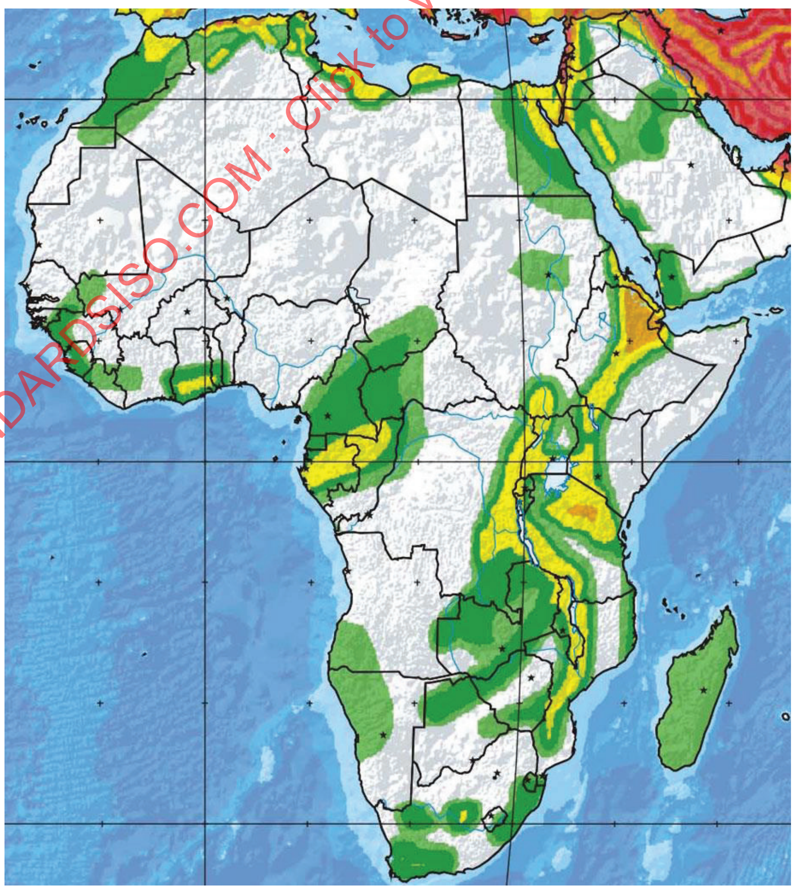
b) Central America and the Caribbean



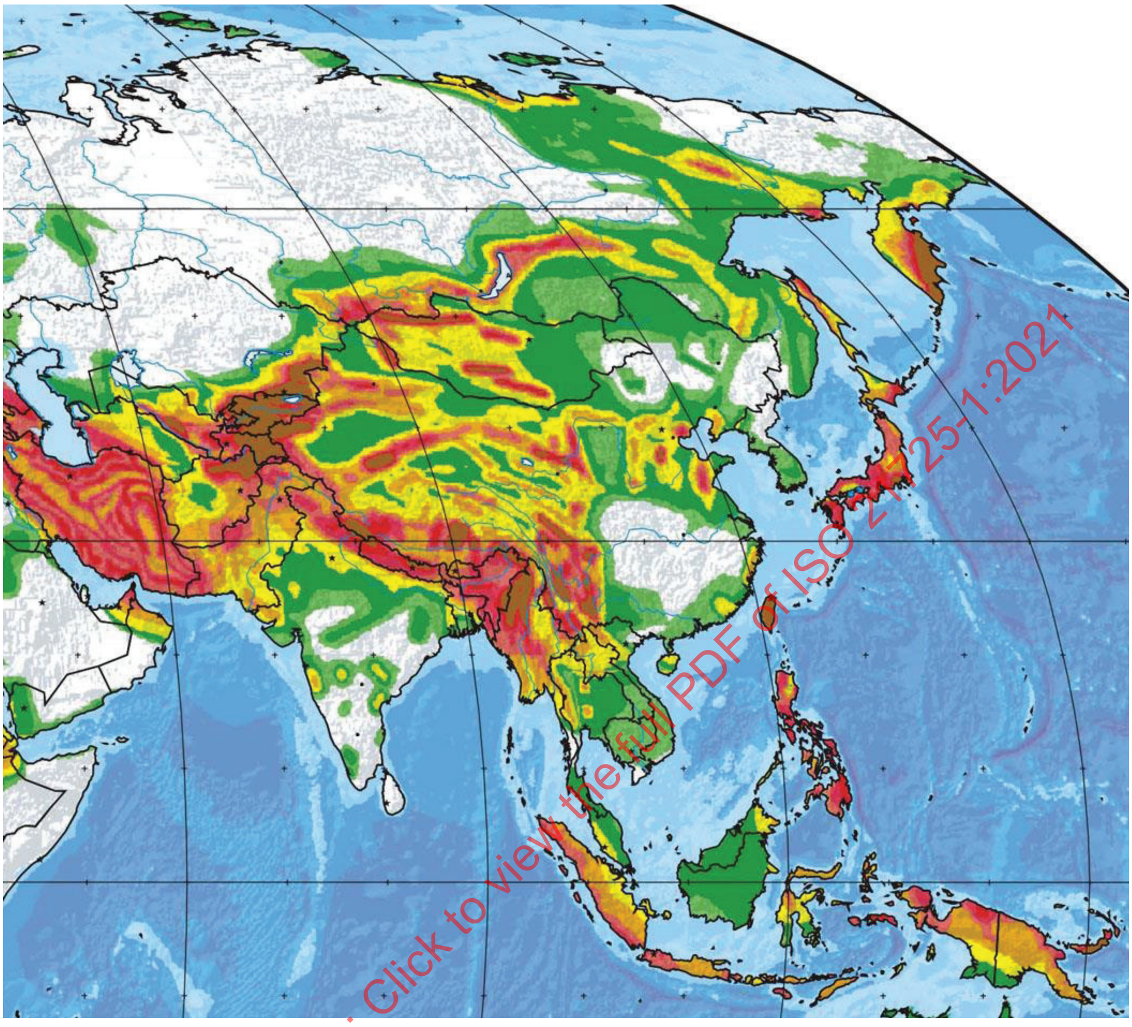
c) South America



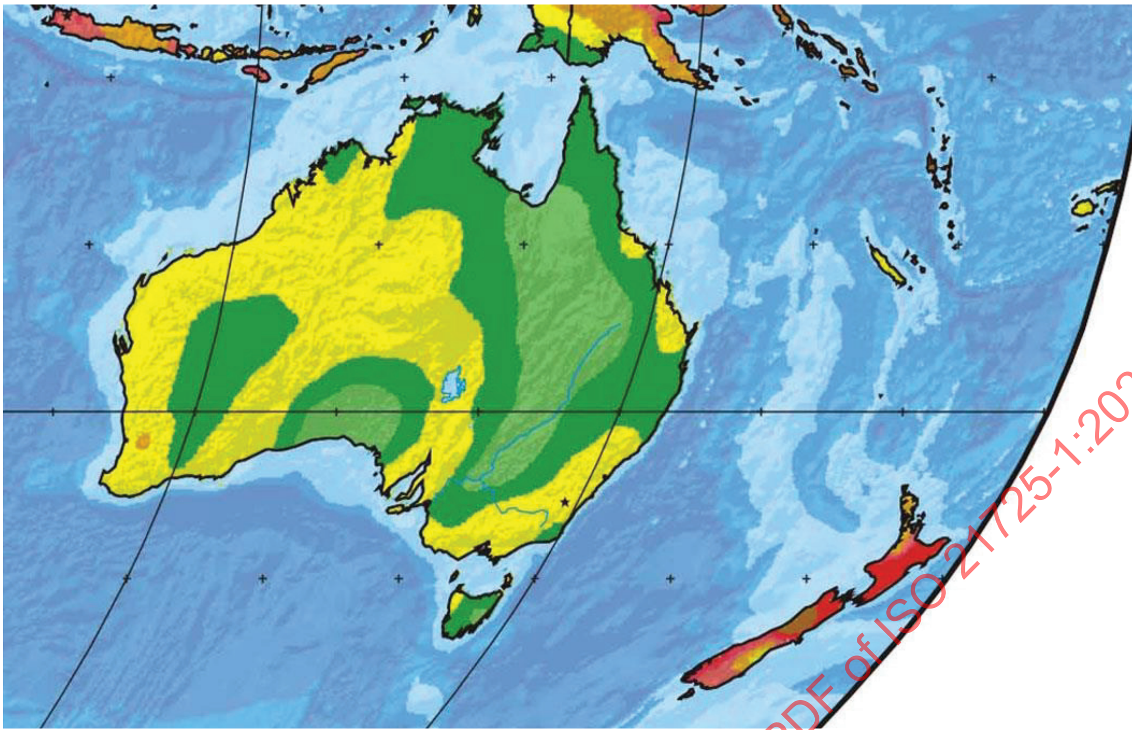
d) Europe



e) Africa



f) Asia



g) Oceania

Key

1 seismic hazard
2 none
3 low
4 intermediate
5 high

Figure 7 — Global seismic hazard map

8.7.7 Soil profile types

Based on the type of soil present at the bridge site, the soil profile shall be classified as one of the following:

— soil profile S_A: hard rock with a measured shear wave velocity v_s > 1 500 m/s;

— soil profile S_B: rock with moderate fracturing and weathering with a measured shear wave velocity in the range (1 500 m/s ≥ v_s > 750 m/s);

— soil profile S_C: soft weathered or fractured rock, or dense or stiff soil, where the measured shear wave velocity is in the range (750 m/s ≥ v_s > 350 m/s), or, in the upper 30 m, the standard penetration test resistance has an average value of N > 50 or a shear strength for clays s_u ≥ 100 kPa;

— soil profile S_D: predominately medium-dense to dense, or medium stiff to stiff soil, where the measured shear wave velocity is in the range (350 m/s ≥ v_s > 180 m/s), or where, in the upper 30 m, the standard penetration test resistance has an average value in the range ($15 < N \leq 50$), or a shear strength for clays in the range ($50 \text{ kPa} \leq s_u < 100 \text{ kPa}$);

— soil profile S_E: a soil profile where the measured shear wave velocity $v_s \leq 180$ m/s, or the standard penetration test resistance has an average value $N < 15$ in the upper 30 m, or has more than 3,5 m of plastic ($PI > 20$), high moisture content ($w > 40$ %) and low shear strength ($s_u < 25$ kPa) clays; and

— seismically vulnerable soils: sites where the soil profile contains soil having one or more of the following characteristics are beyond the scope of this document:

  — soils vulnerable to potential failure or collapse under seismic motions, such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soil;

  — peats, highly organic clays, or both, with more than 3 m of thickness;

  — very high plasticity clays ($PI > 75$) where PI means plasticity index, with more than 8 m of thickness; and

  — soft to medium-stiff clays with more than 40 m of thickness.

Soil exploration to obtain the needed values to classify shall always be conducted by a designer familiar with these processes.

8.7.8 Site effects

Site effects shall be described through the site coefficient for short periods of vibration, F_a. The values of the site coefficient for short periods of vibration, F_a, shall be determined from Table 9 as a function of A_a, and the soil profile type from 8.7.7. Linear interpolation can be used between values of A_a in Table 9.

Site effect of seismically vulnerable soils, as described in 8.7.7, are beyond the scope of this document. National standards or other applicable standards can provide requirements for designs.

Table 9 — Site coefficient, F_a

Soil profile	Site coefficient, F_a, for short periods of vibration				
	$A_a < [0,1]$	$[0,1] \leq A_a < [0,2]$	$[0,2] \leq A_a < [0,3]$	$[0,3] \leq A_a < [0,4]$	$[0,4] \leq A_a < [0,5]$
S_A	[0,80]	[0,80]	[0,80]	[0,80]	[0,80]
S_B	[1,00]	[1,00]	[1,00]	[1,00]	[1,00]
S_C	[1,20]	[1,20]	[1,10]	[1,00]	[1,00]
S_D	[1,60]	[1,40]	[1,20]	[1,10]	[1,00]
S_E	[2,50]	[2,70]	[1,20]	[0,90]	[0,90]

NOTE The values for $0,4 \leq A_a < 0,5$ can approximately be applied to the case of $A_a \geq 0,5$.

8.7.9 Design response spectral ordinates

For bridges complying with the limitations set forth in 6.1, natural periods of vibration may be assumed to fall within the range of short periods for which response to ground motion is constant.

The ordinates of the elastic design response spectrum, S_a, for a damping ratio of 5 % of critical damping, expressed as a fraction of the acceleration of gravity, shall be calculated in the short periods of vibration range, using Formula (9):

$$S_a = 2{,}5\, A_a F_a \tag{9}$$

8.7.10 Seismic equivalent uniformly distributed load

A seismic uniformly distributed load, w_s, equivalent to the total horizontal inertial effects caused by the seismic ground motions, distributed along the length of the bridge, should be determined using Formula (10):

$$w_s = \frac{m_T \, g \, S_a}{L_T} \tag{10}$$

where

g is the force of gravity;

L_T is the total length of the bridge;

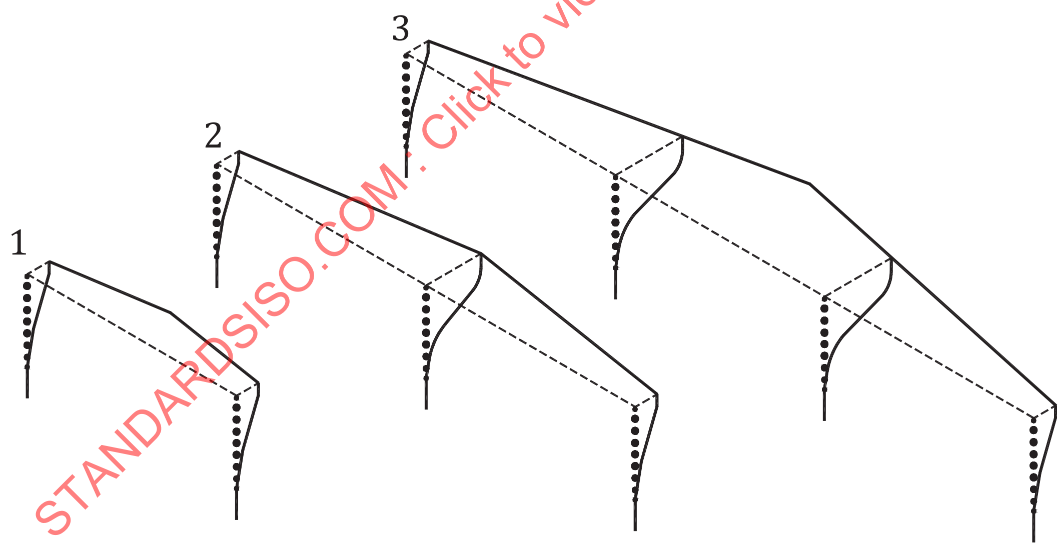
m_T is the total mass of the bridge that is not directly absorbed by the supports, including all structural elements, intermediate walls, piers, columns, etc., excluding footings and abutments;

S_a is the design spectrum ordinate.

This distribution is considered as uniformly distributed load, as simple as it is possible, from the viewpoint of simplified design because the distribution of lateral forces has a little effect on the inertia force of substructure.

8.7.11 Fundamental mode shape

For bridges complying with the limitations set forth in 6.1, dynamic response may be assumed to be dominated by the fundamental mode characteristics. The fundamental mode shape for each one of the possible cases of one, two or three spans, is shown in Figure 8.



Key

1 one span

2 two spans

3 three spans

Figure 8 — Fundamental mode shape

The fundamental shape for each case and for each span is described by a function $u(x)$, as per Table 10.

Table 10 — Unitary deformation distribution

Number of spans	Span	$u(x)$
1	Single span	$16\frac{x^2}{L^4}\left(2Lx-L^2-x^2\right)$
2	First span	$\left[16\frac{x^2}{L^4}\left(2Lx-L^2-x^2\right)+\frac{I_\mathrm{D}H^4x}{I_\mathrm{P}L^5}\right]\left(\frac{I_\mathrm{P}L^4}{I_\mathrm{D}H^4}\right)$
	Second span	$\left[16\frac{x^2}{L^4}\left(2Lx-L^2-x^2\right)+\frac{I_\mathrm{D}H^4(L-x)}{I_\mathrm{P}L^5}\right]\left(\frac{I_\mathrm{P}L^4}{I_\mathrm{D}H^4}\right)$
3	First span	$\left[16\frac{x^2}{L^4}\left(2Lx-L^2-x^2\right)+\frac{I_\mathrm{D}H^4x}{I_\mathrm{P}L^5}\right]\left(\frac{I_\mathrm{P}L^4}{I_\mathrm{D}H^4}\right)$
	Second span	$\left[16\frac{x^2}{L^4}\left(2Lx-L^2-x^2\right)+\frac{I_\mathrm{D}H^4x}{I_\mathrm{P}L^5}+\frac{I_\mathrm{D}H^4(L-x)}{I_\mathrm{P}L^5}\right]\left(\frac{I_\mathrm{P}L^4}{I_\mathrm{D}H^4}\right)$
	Third span	$\left[16\frac{x^2}{L^4}\left(2Lx-L^2-x^2\right)+\frac{I_\mathrm{D}H^4(L-x)}{I_\mathrm{P}L^5}\right]\left(\frac{I_\mathrm{P}L^4}{I_\mathrm{D}H^4}\right)$

where

H is the largest height of the bridge supports in metres;

I_D is the second moment of area of the deck section in m^4, for bending within its plane, due to horizontal forces;

I_P is the second moment of area of the wall, frame or pier, in m^4, for bending due to horizontal forces;

L is the span's length in metres;

x is any point along the considered span.

8.7.12 Lateral equivalent design forces

The equivalent lateral force, w_e, applied directly to the substructure and superstructure elements, employed to represent the dynamic response of the structure to the ground acceleration, should be determined using Formula (11):

$$w_\mathrm{e} = w_\mathrm{s}\, u(x) \tag{11}$$

where

$u(x)$ is the function describing the fundamental mode shape, as specified in 8.7.11;

w_s is a uniformly distributed load caused by the seismic ground motions, as specified in 8.7.10.

8.8 Jacking and post-tensioning forces

8.8.1 Jacking forces

The design jacking forces in service shall not be less than 1,3 times the permanent load reaction at the bearing, adjacent to the point of jacking.

8.8.2 Forces for post-tensioning anchorage

The design force for post-tensioning anchorage zones shall be taken as 1,2 times the maximum jacking force.

8.9 Thermal forces

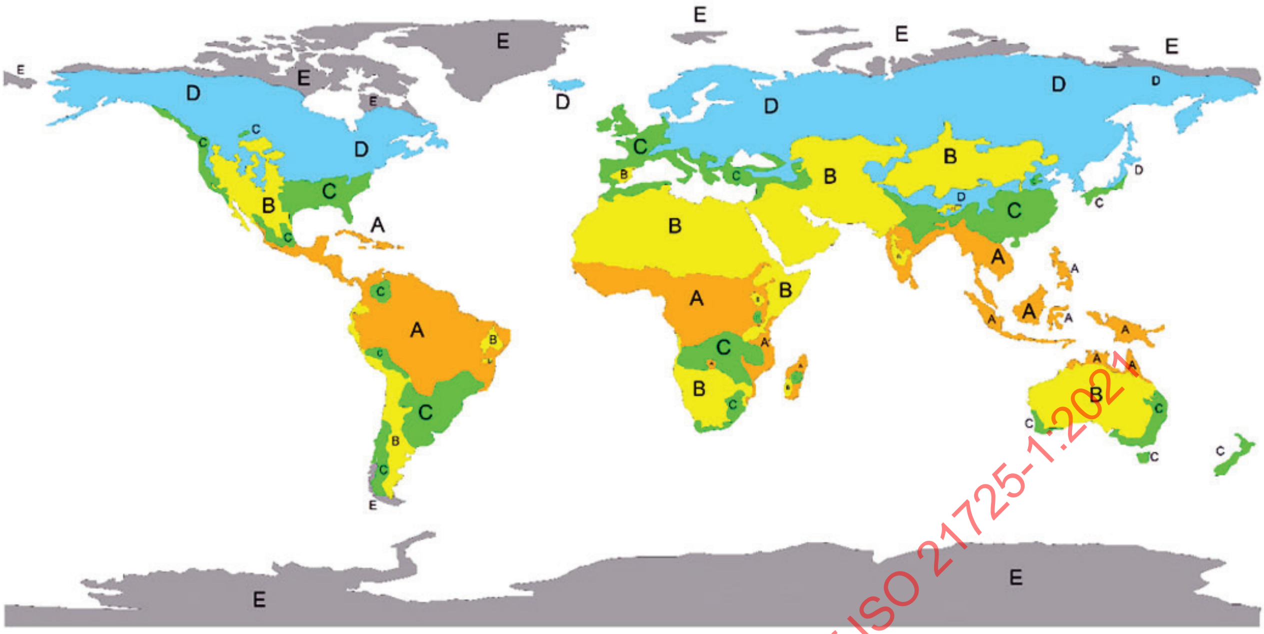
Longitudinal change due to thermal expansion, δ_T, shall be calculated for a continuous deck, as per Formula (12), at each vertical support.

$$\delta_T = \alpha_c \Delta_T L_c \tag{12}$$

The value for the coefficient of thermal expansion for concrete, α_c, depends mainly on the type of aggregate used. Accepted values range from 10×10^{-6} m/m/$_{\circ C}$ to $13{,}5 \times 10^{-6}$ m/m/$_{\circ C}$. A value of [$11{,}5 \times 10^{-6}$] m/m/$_{\circ C}$ may be used for α_c in Formula (12). Δ_T is the maximum variation in temperature recorded at the site of the bridge. In lieu of this information, the designer may use the data provided in Table 11. Classification of temperature region for the bridge site should be made according to Figure 9.

Table 11 — Temperature variation according to world region

Region		Δ_T, °C
A	Tropical	[20]
B	Dry	[30]
C	Temperate	[15]
D	Cold	[10]
E	Polar	[---]



Key

- A tropical
- B dry
- C temperate
- D cold
- E polar

Figure 9 — Map of temperature regions of the world

Shear force, V_T, caused by thermal expansion, δ_T, in the vertical supports should be calculated as per Formula (13). Moment due to V_T should be also calculated.

$$V_T = 60 \times 10^6 \frac{I_P \delta_T}{H_P} \tag{13}$$

where

H_P is the height of the support where forces are being evaluated;

I_P is the second moment of area, about an axis on the bridge deck perpendicular to the bridge length, of the pier, frame or wall serving as support for the superstructure.

When elastomeric pads without lateral movements restrictions are used between the superstructure and its supports, it is possible that thermal expansion of the bridge deck is not fully transmitted to the infrastructure. Therefore, it may be reduced by a factor of [0,4].

For one span or multi-simply–supported span bridges complying with limits set forth in 6.1, thermal expansion forces may be considered non-significant.

8.10 Load combinations

8.10.1 Ultimate loads

All elements of the superstructure, the substructure and the foundation should be designed for the simultaneous application of various groups of the loads and forces specified in 8.2 to 8.9, increased by amplification factors, A, depending on the loads being combined in each group, as per Table 12.

Table 12 — Load amplification factors and load combinations

Group	Amplification factor, A				
	Dead loads	Live loads, including dynamic effects	Longitudinal forces	Earth pressure	Earthquake inertial forces
1	[1,35]	[1,35]	[1]	[1,5]	[0]
2	[1,35]	[1,5]	[0]	[1,5]	[0]
3	[1,35]	[1,7]	[0]	[0,5]	[0]
4	[1,35]	[1,9]	[0]	[0]	[0]
5	[1]	[0]	[0]	[1,5]	[0]
6	[1,2]	[0]	[0]	[1,5]	[1]
7	[1]	[0]	[0]	[0]	[1]
8	[1]	[1,3]	[0]	[0]	[1]
9	[0,9]	[0]	[0]	[0]	[1]

8.10.2 Service loads

For service loads, the same load combinations shown in Table 12 should be used, except all factors for other than earthquake forces should be taken as 1. Earthquake loads factors should be taken as 0,75.

9 Design requirements

9.1 Scope

The present clause contains the provisions that are common to the structural concrete elements covered by this document. They include: provisions for materials, concrete cover of reinforcement, details and limits on the amount of reinforcement.

9.2 Additional requirements

The designer should comply with the additional requirements of this document.

9.3 Materials for structural concrete

9.3.1 General

All materials employed in the construction of the structure designed following this document should conform to the following ISO standards.

9.3.2 Cement

Cement should conform to ISO 679, ISO 863 or corresponding national cement standards.

9.3.3 Aggregates

Aggregates should conform to ISO 20290-2, ISO 20290-3 or corresponding national aggregate standards.

9.3.4 Water

Water used in mixing concrete should be potable, clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials or other substances deleterious to concrete or reinforcement, and should conform to the applicable ISO standards or corresponding national mixing water standard.

9.3.5 Steel reinforcement

9.3.5.1 General

Steel reinforcement should conform to ISO 10144. Welded-wire fabric (mesh wire) should be considered deformed reinforcement in this document.

9.3.5.2 Deformed reinforcement

The maximum specified yield strength for deformed reinforcement should be 500 MPa. Deformed reinforcing bars should conform to ISO 6935-2 or corresponding national deformed reinforcement standard. ISO 6935-2 covers grades RB 300, RB 400, and RB 500 (300 MPa, 400 MPa, and 500 MPa characteristic upper yield stress, respectively) and nominal diameters of 6 mm, 8 mm, 10 mm, 12 mm, 16 mm, 20 mm, 25 mm, 32 mm and 40 mm, although the nominal diameter of deformed reinforcement bars is limited to 32 mm in this document (see 9.3.11).

9.3.5.3 Welded-wire fabric

The maximum specified yield strength for wires being part of welded-wire fabric should be 400 MPa. Welded-wire fabric should conform to ISO 6935-3 or corresponding national welded-wire fabric standard. The nominal diameter of wire for welded-wire fabric is limited to 10 mm in this document (see 9.3.11).

9.3.5.4 Plain reinforcement

Plain reinforcement should be permitted only for stirrups, ties, spirals, and when it is part of a welded-wire fabric. The maximum specified yield strength for plain reinforcement should be 300 MPa. Plain reinforcing bars should conform to ISO 6935-1 or corresponding national plain reinforcement standard. ISO 6935-1 covers grades PB 240 and PB 300 (240 MPa and 300 MPa characteristic upper yield stress, respectively) and nominal diameters of (6, 8, 10, 12, 16, and 20) mm, although the nominal diameter of plain reinforcement bars is limited to 16 mm in this document (see 9.3.11).

9.3.6 Prestressing steel

Tensile and yield strengths for prestressing steel may be taken as specified in Table 13. Prestressing steels should conform to ISO 6934-4 and ISO 6934-5 or corresponding national deformed prestressing steel standard. ISO 6934-4 covers 2 wire strand, 3 wire strand, 7 wire strand, and 19 wire strand, although the present document covers 7 wire strand. ISO 6934-5 covers 1 030 MPa, 1 080 MPa, 1 180 MPa and 1 230 MPa characteristic tensile strength and nominal diameters of 15 mm, 17 mm, 20 mm, 23 mm, 26 mm, 32 mm, 36 mm and 40 mm, although this document covers 1 030 MPa characteristic tensile strength and 19 mm~35 mm nominal diameter steel bars.

Table 13 — Properties of prestressing strand and bar

Material	Type	Diameter mm	Tensile strength (f_{pu}) MPa	Yield strength (f_{py}) MPa
Strand	7 wire			
	1 720 MPa	9,3 to 15,2	1 720	1 410
	1 860 MPa	9,5 to 15,2	1 860	1 520
Bar	Plain Deformed	20 to 36	1 030	835

9.3.7 Post-tensioning anchorages and couplers

9.3.7.1 General

All anchorages and couplers shall develop at least 96 % of the actual ultimate strength of the prestressing steel, when tested in an unbonded state, without exceeding anticipated set. The coupling of tendons shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplers and/or coupler components shall be enclosed in housings long enough to permit the necessary movements.

Corrosion protection shall be provided for tendons, anchorages, end fittings and couplers.

Special anchorage device shall be verified by acceptance tests presented in References [19] and [20]. Special shaped anchorage can be used by approval of a chief engineer.

The acceptance tests conforming to EAD 160004-00-0301 are considered equivalent to References [19] and [20] and are accepted as an alternative method considering the adapted criteria, i.e. 95 % of the actual ultimate strength of the prestressing steel and at least 2,0 % of total elongation at maximum load.

9.3.7.2 Bonded systems

Bond transfer lengths between anchorages and the zone where full prestressing force is required under service and ultimate loads shall normally be sufficient to develop the minimum specified ultimate strength of the prestressing steel. When anchorages or couplers are located at critical sections under ultimate load, the ultimate strength required of the bonded tendons shall not exceed the ultimate capacity of the tendon assembly, including the anchorage or coupler, tested in an unbonded state.

Housings shall be designed so that complete grouting of all the coupler components is accomplished during grouting of the tendons.

9.3.7.3 Unbonded systems

For unbonded tendons, two dynamic tests shall be performed on a representative anchorage and coupler specimen and the tendon shall withstand, without failure, 500 000 cycles from 60 % to 66 % of its minimum specified ultimate strength, and also 50 cycles from 40 % to 80 % of its minimum specified ultimate strength. Each cycle shall be taken as the change from the lower stress level to the upper stress level and back to the lower. Different specimens may be used for each of the two tests. Systems utilizing multiple strands, wires or bars may be tested using a test tendon of smaller capacity than the full-size tendon. The test tendon shall duplicate the behaviour of the full-size tendon and, generally, shall not have less than 10 % of the capacity of the full-size tendon. Dynamic tests shall be required on bonded tendons where the anchorage is located or used in such a manner that repeated load applications can be expected on the anchorage. Otherwise, dynamic tests shall be required only if specified in the contract documents.

Anchorages for unbonded tendons shall not cause a reduction in the total elongation under ultimate load of the tendon to less than 2 % measured in a minimum gage length of 3 000 mm.

All the coupling components shall be completely protected with a coating material prior to final encasement in concrete.

9.3.8 Ducts

9.3.8.1 General

Ducts for tendons shall be rigid or semirigid either galvanized ferrous metal or polyethylene, or they shall be formed in the concrete with removable cores.

The radius of curvature of tendon ducts shall not be less than 6 000 mm, except in the anchorage areas where 3 600 mm may be permitted.

Polyethylene ducts shall not be used when the radius of curvature of the tendon is less than 9 000 mm.

Where polyethylene ducts are used and the tendons are to be bonded, the bonding characteristics of polyethylene ducts to the concrete and the grout should be investigated.

The effects of grouting pressure on the ducts and the surrounding concrete shall be investigated.

Polyethylene duct and metal duct for longitudinal and transverse post-tensioning in the flanges shall be supported at intervals not to exceed 600 mm. Polyethylene duct in webs for longitudinal post-tensioning shall be tied to stirrups at intervals not to exceed 600 mm, and metal duct for longitudinal post-tensioning in webs shall be tied to stirrups at intervals not to exceed 1 200 mm.

9.3.8.2 Size of ducts

The inside diameter of ducts shall be at least 6 mm larger than the nominal diameter of single bar or strand tendons. For multiple bar or strand tendons, the inside cross-sectional area of the duct shall be at least 2,0 times the net area of the prestressing steel with one exception where tendons are to be placed by the pull-through method, the duct area shall be at least 2,5 times the net area of the prestressing steel.

The size of ducts shall not exceed 0,4 times the least gross concrete thickness at the duct.

9.3.9 Admixtures

Admixtures should conform to the applicable ISO standards or corresponding national admixtures standard.

9.3.10 Storage of materials

Cement and aggregates should be stored in such a manner as to prevent deterioration and intrusion of foreign matter. Any material that has deteriorated or has been contaminated should not be used for concrete.

9.3.11 Minimum and maximum reinforcement bar diameter

Reinforcement employed in structures designed according to this document should not have a nominal diameter, d_b, less than the minimum diameter, nor should it be larger than the maximum diameter given in Table 14.

Table 14 — Minimum and maximum reinforcing bar diameters

Reinforcement	Minimum bar diameter d_b mm	Maximum bar diameter d_b mm
Deformed reinforcing bars (see 9.3.5.2)	[8]	[32]

Table 14 *(continued)*

Reinforcement	Minimum bar diameter d_b mm	Maximum bar diameter d_b mm
Prestressing steel – wire (see 9.3.6)	[9,3] for 1 720 MPa strand [9,5] for 1 860 MPa strand	[15,2]
Prestressing steel – bar (see 9.3.6)	[20]	[36]
Wire for welded-wire fabric (see 9.3.5.3)	[4]	[10]
For stirrups and ties	[6]	[16]
Plain reinforcing bars (see 9.3.5.4)	[6]	[16]
For non-seismic areas	[4]	[32]

9.4 Concrete mixture proportioning

9.4.1 General

Concrete shall be proportioned to provide an average compressive strength, f_{cr}', that shall minimize the frequency of strengths below f_c'. The requirements for f_c' shall be based on 28-day age tests on pairs of cylinders made and tested according to ISO 1920 [3]. The proportions of material for concrete shall be established to provide:

a) workability and consistency to permit concrete to be worked readily into forms and around reinforcement under the conditions of placement to be used, without segregation or excessive bleeding;

b) resistance to special exposures; and

c) conformance with strength test requirements.

Concrete proportions, including water-binder ratios, shall be established based on field experience, trial mixtures, or both, with the materials to be used.

9.4.2 Durability requirements

9.4.2.1 General

To obtain an appropriate durability of the concrete, a minimum amount of cement shall be provided by using water-binder ratios below specified values and by specifying a minimum compressive strength for the concrete.

9.4.2.2 Calculation of the water-binder ratio

The water-binder ratios shall be calculated using the weight of water in kg/m^3 of concrete divided by the binder used in the mixture in kg/m^3 of concrete. The use of fly ash, pozzolans, slag and silica fume is beyond the scope of this document. If used, it shall be in accordance with appropriate ISO standards.

9.4.2.3 Freezing and thawing exposures

Concrete exposed to freezing and thawing or de-icing chemicals shall be air-entrained with a total air content of 6 % for severe exposure and of 5 % for moderate exposure. Tolerance on air content in fresh concrete shall be ±1,5 %.

9.4.2.4 Requirements for special exposure conditions

Concrete maximum water-binder ratios and minimum specified compressive strength should comply with specification of Table 15, according to conditions of exposure.

Table 15 — Requirements for special exposure conditions

Exposure condition	Maximum water-binder ratio by weight	Minimum f_c', MPa
Concrete intended to have low permeability when exposed to water	0,5	28
Concrete exposed to freezing and thawing in a moist condition or to de-icing chemicals	0,45	31,5
For corrosion protection of reinforcement in concrete exposed to chlorides from de-icing chemicals, salt, salt water, brackish water, seawater, or spray from these sources	0,4	35

9.4.2.5 Sulfate exposures

When water soluble sulfate (SO_4) is present in soil and has a concentration greater than 0,10 % by weight or is present in water with more than 0,015 % (150 ppm), concrete exposed to these sulfate-containing solutions or soils shall have a water-binder ratio less than or equal to 0,45 by weight and a minimum compressive strength, f_c', of 31 MPa. Calcium chloride as an admixture shall not be used in concrete exposed to sulfates.

9.4.2.6 Chloride-ion exposure

For corrosion protection of reinforcement in concrete, maximum water-soluble chloride-ion concentrations in hardened concrete at ages from 28 to 42 days contributed from the ingredients including water, aggregates, cement, and admixtures shall not exceed the limits of Table 16.

Table 16 — Maximum chloride ion content for corrosion protection of reinforcement

Type of member	Maximum water-soluble chloride-ion (Cl^-) in concrete, percent by weight of cement
Reinforced concrete exposed to chloride in service	0,15
Reinforced concrete that will be dry or protected from moisture in service	1
Other reinforced concrete construction	0,3

9.4.3 Required average compressive strength

Required average compressive strength, f_{cr}', for concrete shall be 10,5 MPa greater than the specified concrete compressive strength, f_c'.

9.4.4 Proportioning of the concrete mixture

The proportions of the concrete mixture shall be established from trial mixtures using combinations of materials for the proposed work, using at least three different water-binder ratios that comply with the durability requirements of 9.4.2 and the slump requirements from Table 17, and that encompass the required average strength, f_{cr}'. The trial mixtures shall be designed to produce slumps within ±20 mm of the maximum permitted.

Table 17 — Slumps for various types of construction

Member	Slump mm	
	Maximum	Minimum
Reinforced foundation walls, columns and footings	150	50
Plain footings, caissons, and substructure walls and columns	150	50
Beams and reinforced walls	210	80
Columns	210	80
Slabs	150	50
Pavements	75	25
Mass concrete	120	50

9.5 Concrete cover of reinforcement

9.5.1 Minimum concrete cover

The following minimum concrete cover should be provided for unprotected prestressing and reinforcing steel, even in non-seismic areas (see Figures 10 to 13).

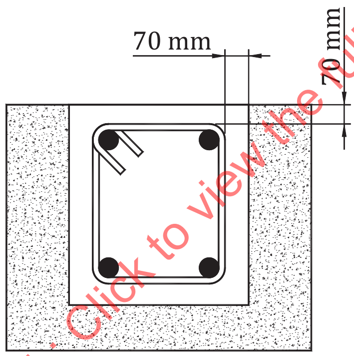


Figure 10 — All types of reinforcement of elements cast and permanently exposed to earth or water (Minimum concrete cover: 70 mm)

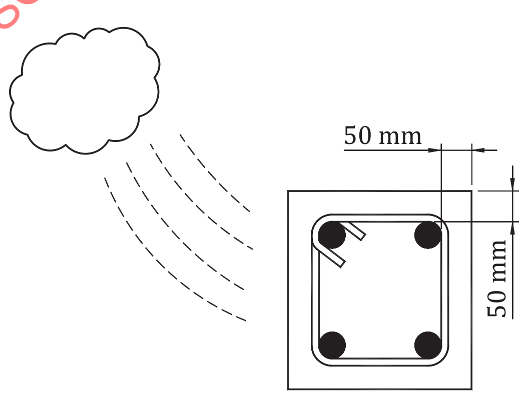


Figure 11 — All types of reinforcement of elements exposed to weather (Minimum concrete cover: 50 mm)

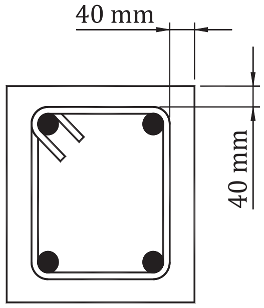


Figure 12 — All types of reinforcement of girders, beams, or columns, when not exposed to weather or in contact with ground (Minimum concrete cover: 40 mm)

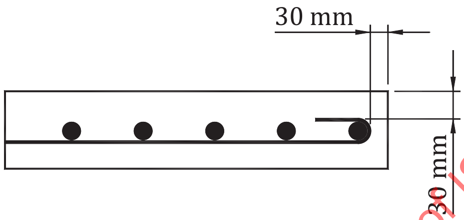


Figure 13 — All types of reinforcement of solid slabs, structural concrete walls or joists, when not exposed to weather or in contact with ground (Minimum concrete cover: 30 mm)

Cover for metal ducts for post-tensioned tendons shall not be less than:

— that specified for reinforcement; and

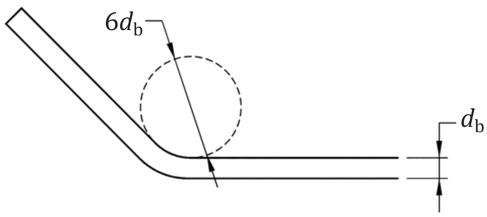
— one-half the diameter of the duct.

9.5.2 Special corrosion protection

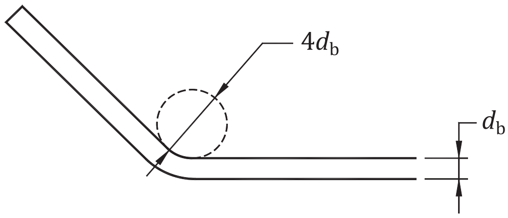
Protection against chloride-induced corrosion may be provided by epoxy coating or galvanizing of reinforcing steel, post-tensioning duct, and anchorage hardware and by epoxy coating of prestressing strand. This type of protection is beyond the scope of this document.

9.6 Minimum reinforcement bend diameter

The diameter of bend of the reinforcement, measured on the inside of the bar, should not be less than the values given in Figure 14.



a) Deformed reinforcing bars and plan reinforcing bars: $6d_b$



b) For stirrups and ties: $4d_b$

Figure 14 — Minimum reinforcement bend diameter

9.7 Standard hook dimensions

The term "standard hook" as used in this document should mean one of Figures 15 to 19.

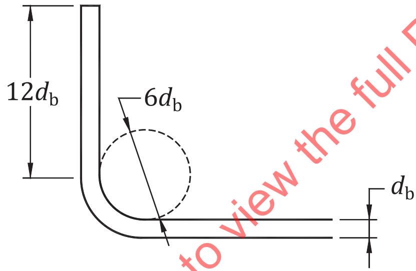


Figure 15 — 90° hook (90° bend plus $12d_b$ extension at free end of bar)

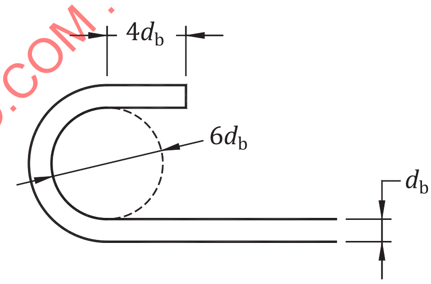
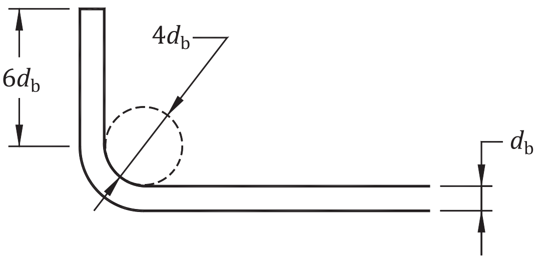
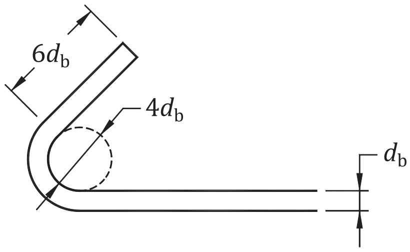


Figure 16 — 180° hook (180° bend plus $4d_b$ extension at free end of bar)



a) 90° bend plus $6d_b$ extension at free end of bar, or



b) 135° bend plus $6d_b$ extension at free end of bar

Figure 17 — For stirrup and tie hooks

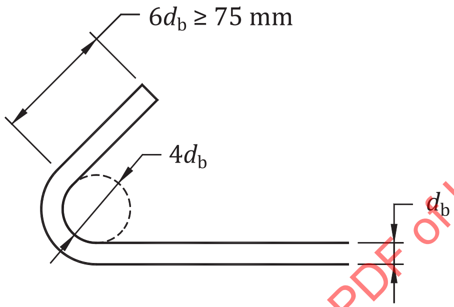


Figure 18 — For confinement stirrups and ties in seismic zones (135° bend plus $6d_b$ extension at free end of bar, but not less than 75 mm)

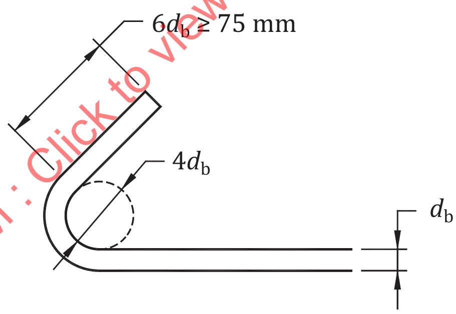


Figure 19 — For crossties in seismic zones (135° bend plus $6d_b$ extension at free end of bar, but not less than 75 mm)

9.8 Bar spacing and maximum aggregate size

9.8.1 General

The clear spacing between parallel bars in a layer and the maximum coarse aggregate size should be interrelated as follows.

9.8.2 Maximum nominal coarse aggregate size

Maximum nominal coarse aggregate size (see Figure 20) should not be larger than:

— 1/5 of the narrowest dimension between sides of forms;

— 1/3 of the depth of slabs; nor

— 3/4 the minimum clear spacing between parallel reinforcing bars or wires.

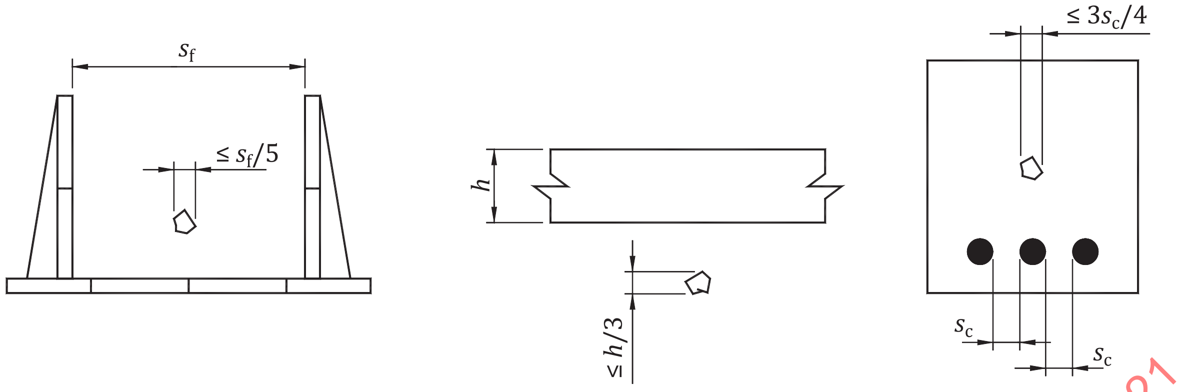


Figure 20 — Maximum nominal coarse aggregate size

9.8.3 Minimum clear spacing between parallel bars in a layer

In solid slabs, girders, beams and joists, the minimum clear spacing between parallel bars in a layer should be the largest nominal bar diameter, d_b, but not less than 25 mm (see Figure 21). This document should apply also for the spacing between parallel stirrups or ties.

9.8.4 Minimum clear spacing between parallel layers of reinforcement

In girders, beams and joists, where parallel reinforcement is placed in two or more layers, bars in the upper layer should be placed directly above bars in the bottom layer with clear distance between layers not less than 25 mm (see Figure 21).

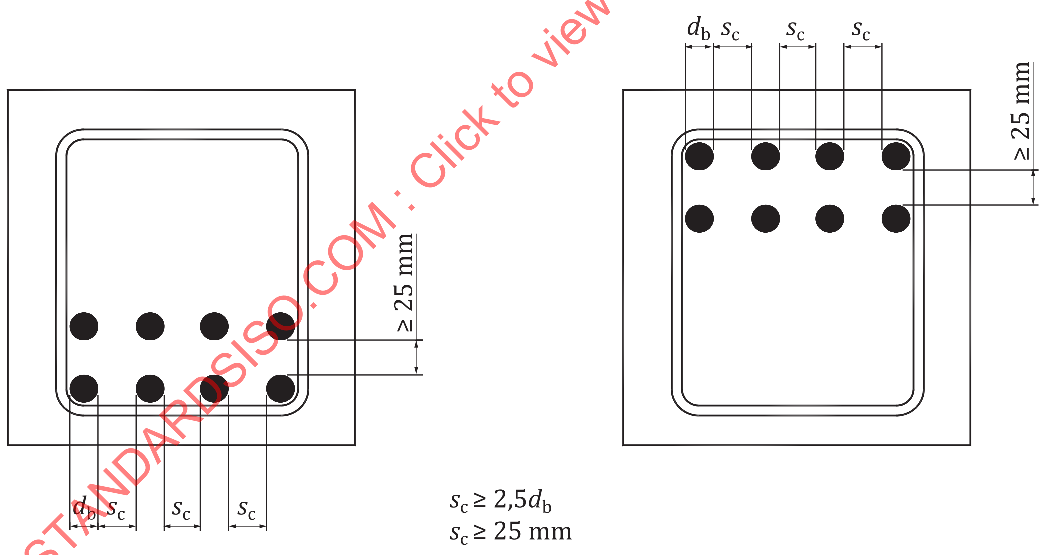


Figure 21 — Minimum clear spacing between parallel bars in a layer and clear distance between parallel layers of reinforcement

9.8.5 Minimum clear spacing between longitudinal bars in columns

In columns, clear distance between longitudinal bars should not be less than $1{,}5d_b$ or 40 mm (see Figure 22).

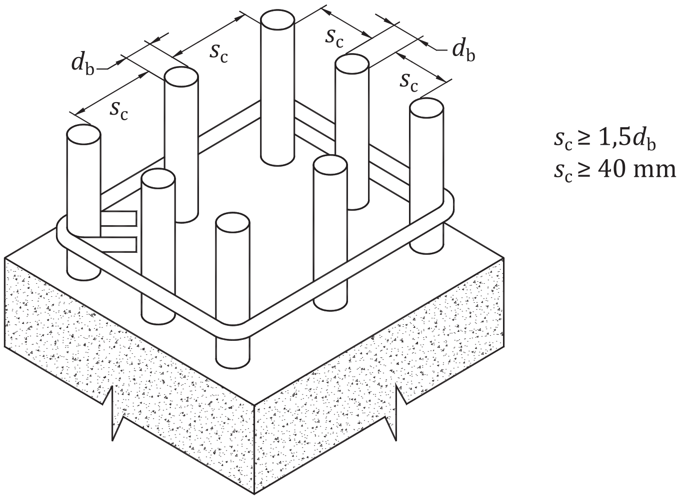


Figure 22 — Clear distance between longitudinal bars in columns

9.8.6 Clear spacing between parallel lap splices

Clear distance limitation between bars should apply also to the clear distance between a contact lap splice and adjacent splices or bars.

9.8.7 Maximum flexural reinforcement spacing in solid slabs

In solid slabs, primary flexural reinforcement should not be spaced farther apart than two times the slab thickness, nor more than 300 mm (see Figure 23).

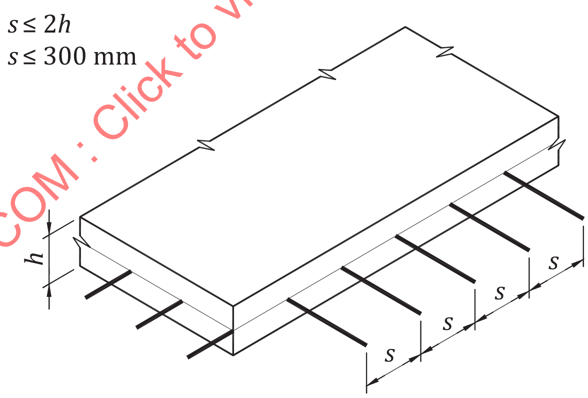


Figure 23 — Spacing between flexural reinforcement in solid slabs

9.8.8 Maximum shrinkage and temperature reinforcement spacing in solid slabs

In slabs, shrinkage and temperature reinforcement should not be spaced farther apart than three times the slab thickness, nor more than 300 mm (see Figure 24).

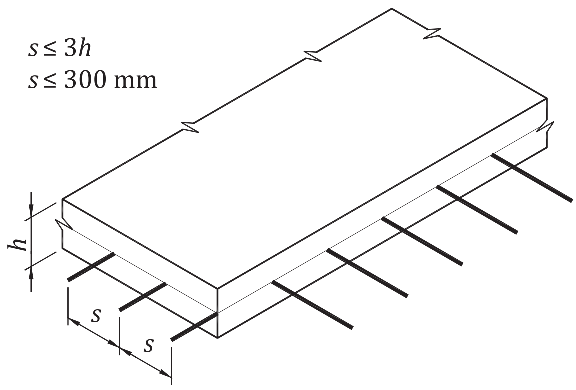


Figure 24 — Spacing between shrinkage and temperature reinforcement in slabs

9.8.9 Maximum reinforcement spacing in structural concrete walls

9.8.9.1 Vertical and horizontal reinforcement

In structural concrete walls, vertical and horizontal reinforcement should not be spaced farther apart than three times the structural concrete wall thickness, nor more than 300 mm (see Figure 25).

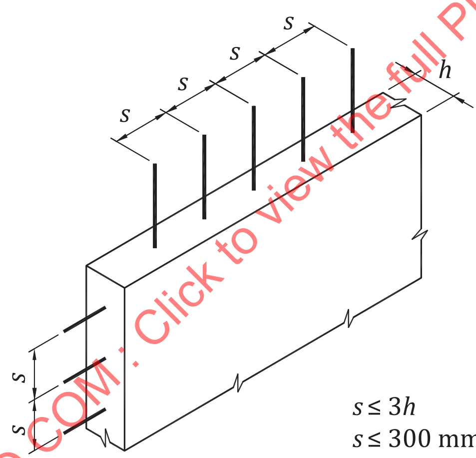


Figure 25 — Spacing between reinforcement in structural concrete walls

9.8.9.2 Number of layers of reinforcement

Structural concrete walls more than 250 mm thick should have vertical and horizontal reinforcement placed in two layers parallel with faces of wall. Each layer should have approximately half of the reinforcement in that direction. The layers should be placed no less than 30 mm nor more than one-third of the thickness of the wall from the surface of the wall. For exterior exposure, the exterior surface layer should be placed no less than 50 mm, instead of the 30 mm prescribed.

9.8.9.3 Special details per element type

The designer should comply with the additional reinforcement detail required for each individual element type.

9.8.10 Minimum spacing of prestressing tendons and ducts

9.8.10.1 Pretensioning strand

The distance between pretensioning strands, including shielded ones, at each end of a member within the transfer length, as specified in 9.9.1.3, shall not be less than a clear distance taken as 1,33 times the maximum size of the aggregate nor less than the centre-to-centre distances specified in Table 18.

Table 18 — Centre-to-centre spacings

Dimensions in millimetres

Strand size	Spacing
15,2	51
12,7 12,4 11,1 10,8	44
9,5 9,3	38

If justified by performance tests of full-scale prototypes of the design, the clear distance between strands at the end of a member may be decreased.

The minimum clear distance between groups of bundled strands shall not be less than 1,33 times the maximum size of the aggregate or 25 mm.

Pretensioning strands in a member may be bundled to touch one another in an essentially vertical plane at and between hold-down locations. Strands bundled in any manner, other than a vertical plane, shall be limited to four strands per bundle.

9.8.10.2 Post-tensioning ducts not curved in the horizontal plane

Unless otherwise specified in this document, the clear distance between straight post-tensioning ducts shall not be less than 38 mm or 1,33 times the maximum size of the coarse aggregate. For precast segmental construction, when post-tensioning tendons extend through an epoxy joint between components, the clear spacing between post-tensioning ducts shall not be less than the greater of the duct internal diameter or 100 mm.

Ducts may be bundled together in groups not exceeding three, provided that the spacing, as specified between individual ducts, is maintained between each duct in the zone within 900 mm of anchorages.

For groups of bundled ducts in construction other than segmental, the minimum clear horizontal distance between adjacent bundles shall not be less than 100 mm. When groups of ducts are located in two or more horizontal planes, a bundle shall contain no more than two ducts in the same horizontal plan.

The minimum vertical clear distance between bundles shall not be less than 38 mm or 1,33 times the maximum size of coarse aggregate.

For precast construction, the minimum clear horizontal distance between groups of ducts may be reduced to 75 mm.

9.8.10.3 Curved post-tensioning ducts

The minimum clear distance between curved ducts shall be as required for tendon confinement as specified in 12.1. The spacing for curved ducts shall not be less than that required for straight ducts.

9.8.11 Maximum spacing of prestressing tendons in slabs

Pretensioning strands for precast slabs shall be spaced symmetrically and uniformly and shall not be farther apart than 1,5 times the total composite slab thickness or 450 mm.

Post-tensioning tendons for slabs shall not be farther apart, centre-to-centre, than 4,0 times the total composite minimum thickness of the slab.

9.8.12 Couplers in post-tensioning tendons

The contract documents shall specify that:

— not more than 50 % of the longitudinal post-tensioning tendons is coupled at one section; and

— the spacing between adjacent coupler locations is not closer than the segment length or twice the segment depth.

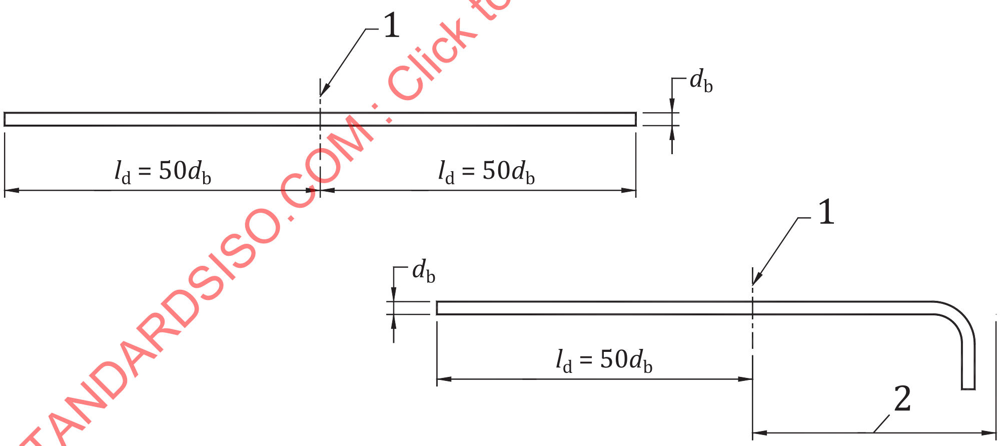
The void areas around couplers shall be deducted from the gross section area and second moment of area when computing stresses at the time post-tensioning force is applied.

9.9 Development length, lap splicing and anchorage of reinforcement

9.9.1 Development length

9.9.1.1 Reinforcing bars

The minimum embedment length, l_d, required on each side of a critical section, for a reinforcing bar to develop its full strength should be $50d_b$, for the bar diameters given in 9.3.11. It should be permitted to replace the development length in one side of the critical section by a length of bar ending in a standard hook complying with the minimum anchorage distance of 9.9.3 (see Figure 26).



Key

1 critical section

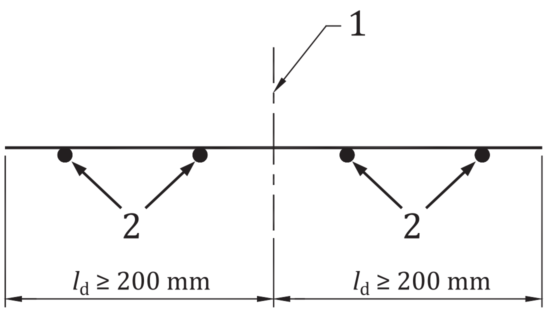
2 anchorage distance (see 9.9.3)

Figure 26 — Required development length for reinforcing bars

Whenever plain bars may be used instead of deformed bars, the development length specified here shall be multiplied by 1,8.

9.9.1.2 Welded-wire fabric

The development length, l_d, of welded-wire fabric measured on each side of the critical section to the end of wire should contain two cross-wires, but should not be less than 200 mm, for the wire diameters given in 9.3.11 (see Figure 27).



Key

1 critical section

2 two cross-wires

Figure 27 — Required development length for welded-wire fabric

9.9.1.3 Prestressing strand

9.9.1.3.1 General

In determining the resistance of pretensioned concrete components in their end zones, the gradual build-up of the strand force in the transfer and development lengths shall be taken into account.

The stress in the prestressing steel may be assumed to vary linearly from 0 at the point where bonding commences to the effective stress after losses, f_{pe}, at the end of the transfer length.

Between the end of the transfer length and the development length, the strand stress may be assumed to increase linearly, reaching the stress at nominal resistance, f_{ps}, at the development length.

For the purpose of this document, the transfer length may be taken as 60 strand diameters and the development length shall be taken as specified in 9.9.1.3.2.

The effects of debonding shall be considered as specified in 9.9.1.3.3.

9.9.1.3.2 Bonded strand

Pretensioning strand shall be bonded beyond the section required to develop f_{ps} for a development length, l_d, in mm, where l_d shall satisfy Formula (14):

$$l_d \geq \kappa\left(0{,}15 f_{ps} - 0{,}097 f_{pe}\right) d_b \tag{14}$$

where

κ = 1,0 for pretensioned members with a depth of less than or equal to 600 mm;

= 1,6 for pretensioned members with a depth greater than 600 mm.

9.9.1.3.3 Partially debonded strands

Where a portion or portions of a pretensioning strand are not bonded and where tension exists in the precompressed tensile zone, the development length, measured from the end of the debonded zone, shall be determined using Formula (14) with a value of $\kappa = 2,0$.

The number of partially debonded strands should not exceed 25 % of the total number of strands.

The number of debonded strands in any horizontal row shall not exceed 40 % of the strands in that row.

The length of debonding of any strand shall be such that all limit states are satisfied with consideration of the total developed resistance at any section being investigated. Not more than 40 % of the debonded strands, or four strands, whichever is greater, shall have the debonding terminated at any section.

Debonded strands shall be symmetrically distributed about the centreline of the member. Debonded lengths of pairs of strands that are symmetrically positioned about the centreline of the member shall be equal.

Exterior strands in each horizontal row shall be fully bonded.

9.9.2 Lap splice dimensions

9.9.2.1 Reinforcing bars

The minimum length of lap for splicing of reinforcing bars should be $50d_b$, for the bar diameters permitted by this document in 9.3.11 (see Figure 28).

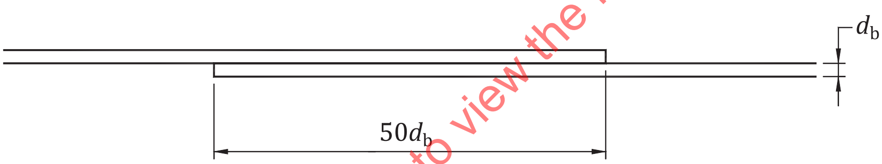
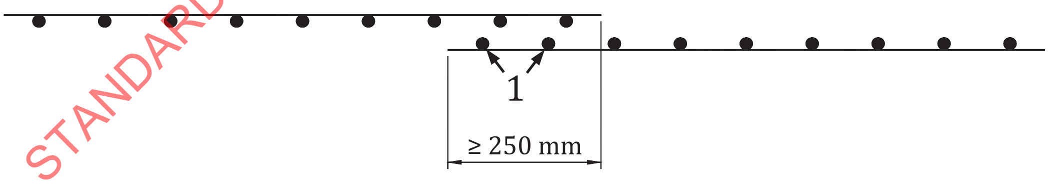


Figure 28 — Minimum lap splice length for reinforcing bars

9.9.2.2 Welded-wire fabric

Welded-wire fabric splicing should be attained by superimposing two cross-wires, but the distance between the edge cross-wires should not be less than 250 mm, for the wire diameters given in 9.3.11 (see Figure 29).



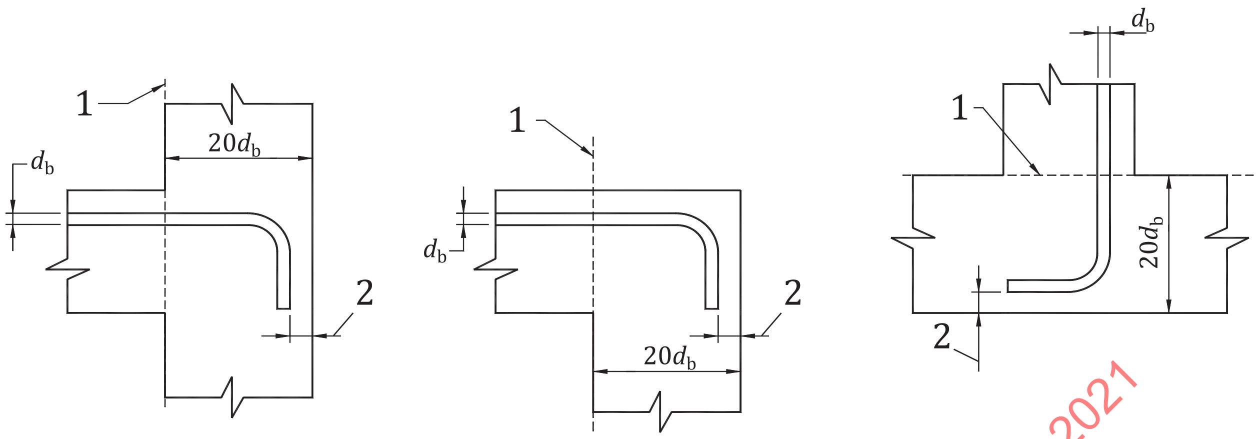
Key

1 two cross-wires

Figure 29 — Minimum lap splice length for welded-wire fabric

9.9.3 Minimum standard hook anchorage distance

The minimum distance between the outer face of concrete and the critical section where the hooked bar develops its full strength should not be less than $20d_b$ (see Figure 30).



Key

1 critical section

2 cover requirement

Figure 30 — Minimum standard hook anchorage distance

9.10 Limits for longitudinal reinforcement

9.10.1 General

Longitudinal reinforcement in reinforced concrete structural elements should be provided to resist axial tension, axial compression, flexure-induced tension and compression, and/or stresses induced by variation of temperature and drying shrinkage from the concrete. The amount of longitudinal reinforcement employed in the structural elements covered by this document should be that required to resist the factored loads and forces, but should be not less than the minimum values given in 9.10. The dimensions of the structural element should be appropriately modified when the amount of calculated reinforcement required to resist the factored loads and forces exceeds the maximum amounts given in 9.10.

9.10.2 Solid slabs and footings

9.10.2.1 Minimum area of shrinkage and temperature reinforcement

Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement should be provided in structural solid slabs and footings where flexural reinforcement extends in one direction only (see Figure 31). The maximum spacing for this reinforcement should comply with 9.8.8. The following minimum ratios of reinforcement area to gross concrete area, ρ_t, should be provided for shrinkage and temperature:

a) where deformed bars with f_y < 350 MPa are used: $\rho_t \geq 0{,}002\,0$;

b) where deformed bars or welded-wire fabric with $f_y \geq 350$ MPa are used: $\rho_t \geq 0{,}001\,8$.

9.10.2.2 Minimum area of tension flexural reinforcement

The minimum area of tension flexural reinforcement, $A_{s,min}$, in structural solid slabs and footings should be greater or equal to the reinforcement area required for shrinkage and temperature stresses as required by 9.10.2.1, ($A_{s,min} \geq \rho_t \cdot b \cdot h$) (see Figure 31). The maximum spacing of this reinforcement should comply with 9.8.7.

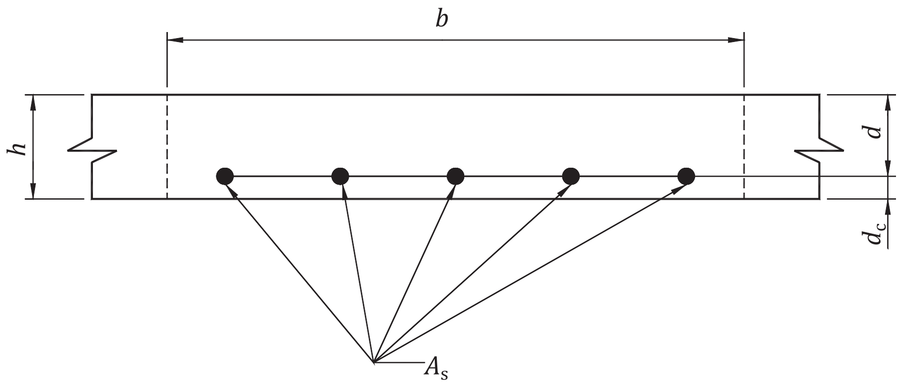


Figure 31 — Slab or footing section

9.10.2.3 Maximum area of tension flexural reinforcement

The maximum reinforcement ratio, $\rho = A_s/(b \cdot d)$, permitted for tension flexural reinforcement in solid slabs and footings should not exceed the value of ρ_{max} given in Table 19. In solid slabs and footings, flexural reinforcement in compression should not be taken into account in the computation of design moment strength.

Table 19 — Maximum flexural reinforcement ratio, ρ_{max}, for solid slabs and footings

		f_y MPa		
		[240]	[300]	[400]
f_c' MPa	[20]	[0,022 0]	[0,016 0]	[0,011 0]
	[25]	[0,027 0]	[0,020 0]	[0,014 0]
	[30]	[0,032 0]	[0,024 0]	[0,016 0]
It should be permitted to interpolate for different values of f_y and f_c'.				

9.10.3 Girders, beams and joists

9.10.3.1 Minimum area of tension flexural reinforcement

At every section of a girder, beam or joist, where tension flexural reinforcement is required, the minimum area of tension flexural reinforcement, $A_{s,min}$, should be greater or equal to the following values, where ρ_{min} is the value given in Table 20.

For rectangular sections and T sections where the flange is in compression [see Formula (15) and Figure 32]:

$$A_{s,min} = \rho_{min} \cdot d \cdot b_w \quad (15)$$

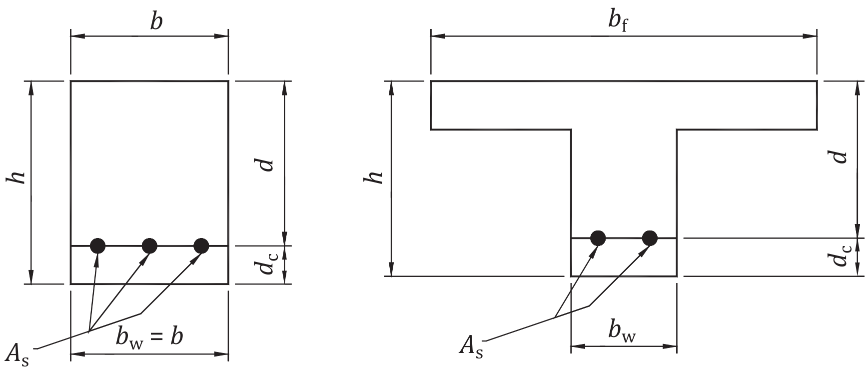


Figure 32 — Rectangular section and T-shaped section with flange in compression

For T sections where the flange is in tension (see Figure 33), $A_{s,min}$ should be greater or equal to the smaller value obtained from Formula (16) or Formula (17):

$$A_{s,min} = 2 \cdot \rho_{min} \cdot d \cdot b_w \tag{16}$$

$$A_{s,min} = \rho_{min} \cdot d \cdot b_f \tag{17}$$

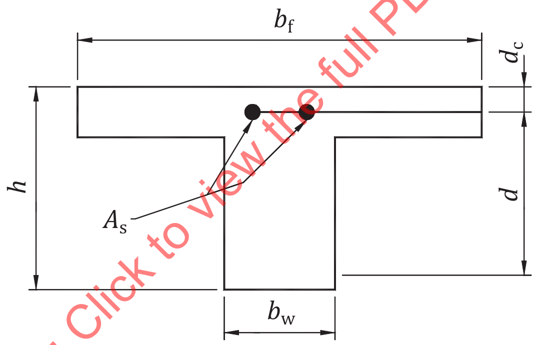


Figure 33 — T-shaped section with flange in tension

Table 20 — Minimum flexural reinforcement ratio, ρ_{min}, for girders, beams and joists

		f_y MPa		
		240	300	400
f_c' MPa	20	0,004 7	0,003 7	0,002 8
	25	0,005 2	0,004 2	0,003 1
	30	0,005 7	0,004 6	0,003 4
It should be permitted to interpolate for different values of f_y and f_c', or use: $\rho_{min} = 0{,}25 \frac{\sqrt{f_c'}}{f_y} \geq \frac{1{,}4}{f_y}$.				

9.10.3.2 Maximum flexural reinforcement ratios

The ratio of tension flexural reinforcement, ρ, should not exceed the following values expressed in function of ρ_{max} as given in Table 21:

In girders, beams and joists, having only tension flexural reinforcement [see Formula (18)]:

$$\rho = \frac{A_s}{b \cdot d} \leq \rho_{max} \tag{18}$$

In girders, beams and joists, having tension and compression flexural reinforcement [see Formula (19) and Figure 34]:

$$\rho - \rho' = \frac{A_s - A_s'}{b \cdot d} \leq \rho_{max} \tag{19}$$

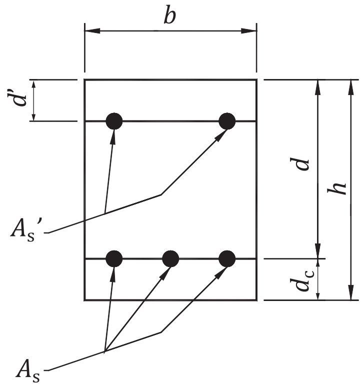


Figure 34 — Section with tension and compression reinforcement

Table 21 — Maximum flexural reinforcement ratio, ρ_{max}, for girders, beams and joists

		f_y MPa		
		240	300	400
f_c' MPa	15	0,024	0,018	0,012
	20	0,032	0,024	0,016
	25	0,04	0,03	0,02
	30	0,048	0,036	0,024

It should be permitted to interpolate for different values of f_y and f_c', or use:

$$\rho_{max} = 0{,}55 \frac{f_c'}{f_y} \cdot \frac{600}{600 + f_y}$$

9.10.4 Columns

9.10.4.1 Minimum and maximum area of longitudinal reinforcement

The total area of longitudinal reinforcement for columns, A_{st}, should not be less than 0,01 nor more than 0,06 times the gross area, A_g, of section [see Formula (20)]:

$$0{,}01 \leq \rho_t = \left[\frac{A_{st}}{A_g}\right] \leq 0{,}06 \tag{20}$$

9.10.4.2 Minimum diameter of longitudinal bars

Longitudinal bars in columns should have a nominal diameter, d_b, of 16 mm or more.

9.10.4.3 Minimum number of longitudinal bars

There should be at least one longitudinal bar in each corner of the section for a minimum 4 bars, in square and rectangular columns with ties, and a minimum of 6 longitudinal bars in round columns with spirals.

9.10.4.4 Distribution of longitudinal bars

The longitudinal bars in the column should be distributed along the perimeter of the section in such a way that the clear spacing between bars along all faces of the column is approximately equal.

9.10.5 Structural concrete walls

9.10.5.1 Minimum area of vertical reinforcement

The minimum ratio, ρ_v, of vertical reinforcement area to gross concrete horizontal section area should be 0,002 5.

9.10.5.2 Maximum area of vertical reinforcement

The maximum ratio, ρ_v, of vertical reinforcement area to gross structural concrete wall horizontal section area should be 0,06, but when the ratio, ρ_v, exceeds 0,01 the vertical reinforcement should be enclosed with ties as prescribed for columns in 9.11.4.2 [see Formula (21)]

$$0{,}0025 \leq \rho_v = \left[\frac{A_{st}}{h \cdot l_w} \right] \leq 0{,}06 \tag{21}$$

9.11 Minimum amounts of transverse reinforcement

9.11.1 General

Transverse reinforcement in reinforced concrete structural elements should be provided to resist shear, diagonal tension, and torsion stresses. It should be provided also to counteract the tendency of compression loaded bars to buckle out of the concrete by bursting the thin outer concrete cover, and to prevent displacement of the longitudinal reinforcement during construction operations. In seismic zones, it should be placed in special regions of the structural elements to provide confinement of concrete subjected to stresses in the non-linear range. The amount of transverse reinforcement employed in the structural elements covered by this document should be that required to resist the factored loads, forces and stresses, but should be not less than the minimum values given in 9.11.4. The dimensions of the structural element should be appropriately modified when the amount of calculated reinforcement required to resist the factored loads, forces and stresses, exceeds the maximum amounts given in 9.11.4.

9.11.2 Slabs

The design procedures for slabs in this document do not require the employment of transverse reinforcement in slabs. The procedures for design of transverse or shear reinforcement in slabs are beyond the scope of this document.

9.11.3 Girders, beams and joists

9.11.3.1 Minimum transverse reinforcement

The minimum transverse reinforcement in girders, beams and joist should be the required value for shear with the exceptions noted in 9.11.3.2.

9.11.3.2 Girders and beams in seismic zones

Girders and beams framing into columns and structural concrete walls located in seismic zones should be provided with confining transverse reinforcement.

9.11.4 Columns

9.11.4.1 General

All columns should have transverse reinforcement in the form of either tie reinforcement or spiral reinforcement conforming to the provisions of 9.11.4.2 or 9.11.4.3, respectively.

9.11.4.2 Ties

Transverse reinforcement in columns in the form of ties, should comply with the following provisions:

a) all longitudinal columns bars should be enclosed by lateral ties made with bars at least 8 mm in diameter ($d_b \geq 8$ mm);

b) ties should be arranged in such a manner that every corner and alternate longitudinal bar should have lateral support provided by the corner of a tie or a crosstie (see Figure 35);

c) no longitudinal bar should be farther than 150 mm clear on each side along the tie from a laterally supported longitudinal bar (see Figure 35);

d) the vertical spacing of ties, *s*, should not exceed a half of the effective depth of the column section (see Figure 36); and

e) the first tie should be located one-half spacing from the top of the slab, beam or footing, where the column is supported, and the uppermost one should be located no more than one-half tie spacing below the lowest horizontal reinforcement of shallowest member supported above.

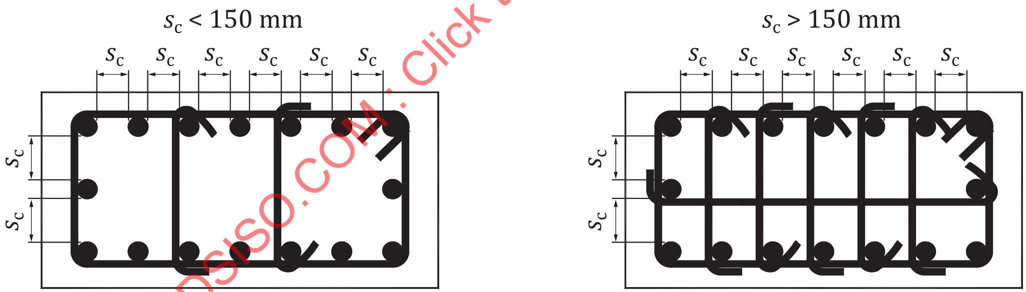
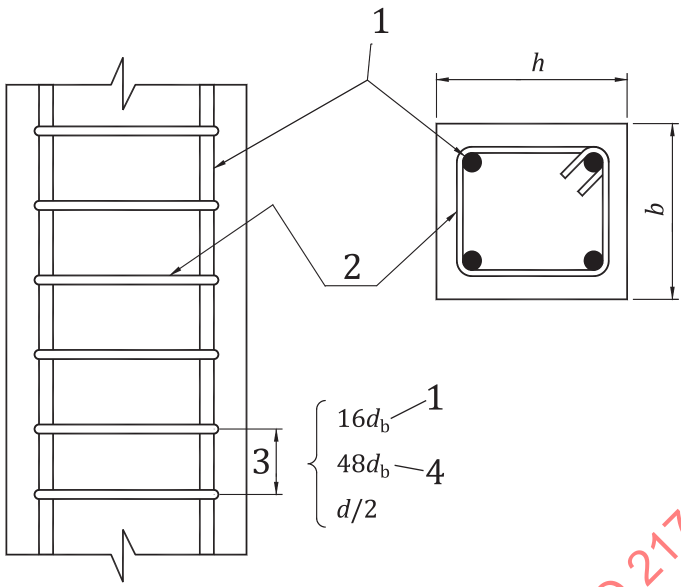


Figure 35 — Arrangement of ties in a tied column section



Key

1 longitudinal bars
2 tie
3 smaller of
4 tie bar

Figure 36 — Vertical spacing of ties in a tied column

9.11.4.3 Spirals

Columns with spiral reinforcement should comply with the following guides:

a) all longitudinal column bars should be enclosed by a spiral consisting of an evenly spaced continuous bar at least 8 mm in diameter ($d_b \geq 8$ mm);

b) clear spacing between spirals should not exceed 80 mm, nor be less than 25 mm, and should comply with the provisions of 9.8;

c) anchorage of the spiral reinforcement should be provided by 1½ extra turns at each end of a spiral unit;

d) splices in spiral reinforcement should comply with 9.9.2;

e) spirals should extend from top of footing or slab to level of lowest horizontal reinforcement of shallowest member supported above. In columns with capitals, the spiral should extend to a level at which the diameter or width of capital is two times that of the column; and

f) ratio of spiral reinforcement, ρ_s, defined as ratio of the volume of reinforcement contained in one loop of the spiral to the volume of concrete in the core of the column confined by the same loop of spiral, should be not less than any of the values given by Formula (22) (see Figure 37):

$$\rho_s = \frac{A_{ss} \cdot \pi \cdot d_{cc}}{A_{cc} \cdot s} \geq 0{,}12 \frac{f_c'}{f_{ys}} \text{ and } 0{,}45 \left[\frac{A_g}{A_{cc}} - 1 \right] \cdot \frac{f_c'}{f_{ys}} \quad (22)$$

where

A_{cc} is the area of the confined column core measured centre to centre of the spiral;

$A_{cc} = \pi d_{cc}^2/4$

A_g is the gross column section area;

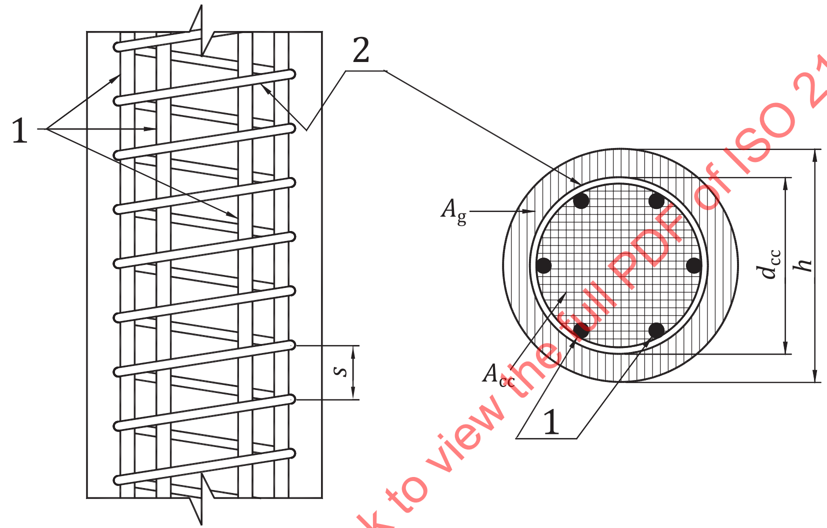
A_{ss} is the area of the bar of spiral;

d_{cc} is the centre-to-centre diameter of the spiral;

f_c' is the specified concrete strength of the column;

f_{ys} is the yield strength of the steel of the spiral;

s is the vertical spacing of the spiral.



Key

1 longitudinal bars

2 spiral

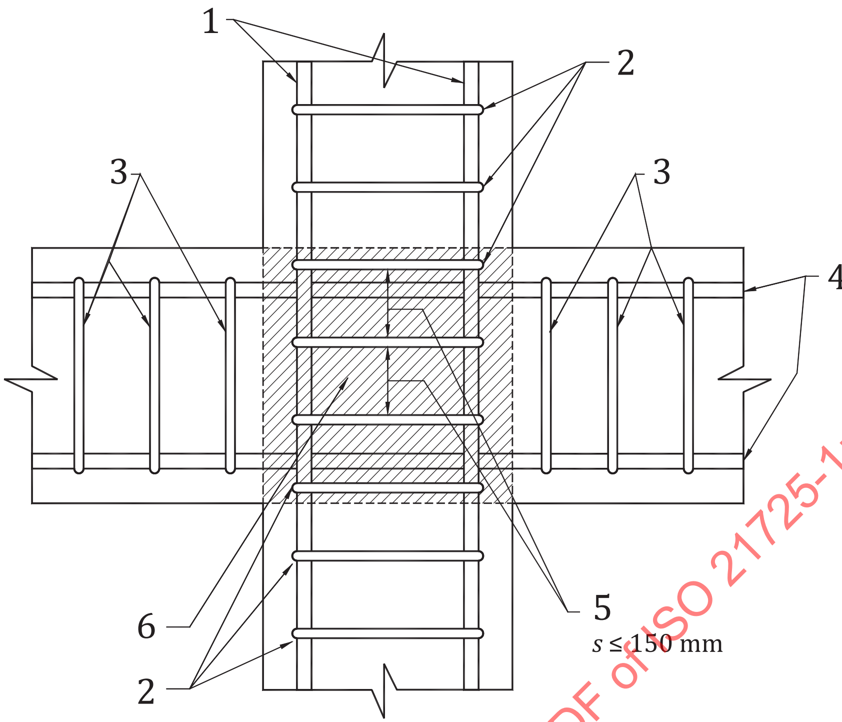
Figure 37 — Spiral reinforcement of column

9.11.4.4 Column-girder joints

At joints of frames where columns and girders meet, a minimum of three column ties, complying with 9.11.4.3 a) to 9.11.4.3 c), should be provided within the joint and the maximum vertical spacing between ties should be 150 mm. As many ties as necessary should be provided to comply with the maximum spacing (see Figure 38).

9.11.5 Structural concrete walls

The minimum ratio, ρ_h, of horizontal reinforcement area to gross concrete vertical section area should be 0,002 5.



Key

1 column longitudinal reinforcement

2 column ties

3 girder stirrups

4 girder longitudinal reinforcement

5 joint ties

6 joint

Figure 38 — Column ties in column-girder joints

10 Stress limitations

10.1 Stress limitations for prestressing tendons

The tendon stress due to prestress or at the serviceability limit state shall not exceed the values:

— specified in Table 22; or

— recommended by the manufacturer of the tendons or anchorages.

The tendon stress at the ultimate limit states shall not exceed the tensile strength limit specified in Table 13.

Table 22 — Stress limits for prestressing tendons

Condition	Tendon type		
	Stress-relieved strand and plain high-strength bars	Low relaxation strand	Deformed high-strength bars
Pretensioning			
Immediately prior to transfer (f_{pi})	0,70 f_{pu}	0,75 f_{pu}	—

Table 22 *(continued)*

At serviceability limit state after all losses (f_{pe})	0,80 f_{py}	0,80 f_{py}	0,80 f_{py}
Post-tensioning			
Prior to seating: short-term f_{pi} may be allowed	0,90 f_{py}	0,90 f_{py}	0,90 f_{py}
At anchorages and coupler immediately after anchor set	0,70 f_{pu}	0,70 f_{pu}	0,70 f_{pu}
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	0,70 f_{pu}	0,74 f_{pu}	0,70 f_{pu}
At serviceability limit state after losses (f_{pe})	0,80 f_{py}	0,80 f_{py}	0,80 f_{py}

10.2 Stress limitations for concrete

If prestressing to the girder is applied only once, stress levels shall be checked when the girder is prestressed and in service. If prestressing to the girder is applied in more than one step, the stress level check shall be added after the deck casting and the secondary prestressing.

Tables 23 and 24 show the construction stages when the stress levels should be checked in the cases of single-stage prestressing and multi-stage prestressing, respectively.

Table 23 — Construction stages for stress level check (single-stage prestressing)

Construction stage	Stress				
	Non-composite section		Composite section		
	Girder		Deck	Girder	
	Top fiber	Bottom fiber	Top fiber	Top fiber	Bottom fiber
1 Primary prestressing (initial prestress after short-term losses)	[Stress]	[Stress]			
2 Primary prestressing (effective prestress after short-term and long-term losses)				[Stress]	[Stress]
3 Dead load: self-weight of girder	[Stress]	[Stress]		[Stress]	[Stress]
4 Dead load: self-weight of deck and cross beams				[Stress]	[Stress]
5 Dead load: self-weight of pavement, barrier, median strip, curb, and sidewalk			[Stress]	[Stress]	[Stress]
6 Live load			[Stress]	[Stress]	[Stress]
Sum (1+3)	[Sum]	[Sum]			
Stress limit (for 1+3)	[Stress limit]	[Stress limit]			
Safety check (for 1+3)	O.K or N.G	O.K or N.G			
Sum (2+3+4+5+6)			[Sum]	[Sum]	[Sum]
Stress limit (for 2+3+4+5+6)			[Stress limit]	[Stress limit]	[Stress limit]

Table 23 *(continued)*

Construction stage	Stress				
	Non-composite section		Composite section		
	Girder		Deck	Girder	
	Top fiber	Bottom fiber	Top fiber	Top fiber	Bottom fiber
Safety check (for 2+3+4+5+6)			O.K or N.G	O.K or N.G	O.K or N.G

Table 24 — Construction stages for stress level check (multi-stage prestressing)

Construction stage	Stress					
	Non-composite section		Composite section			
	Girder		Deck		Girder	
	Top fiber	Bottom fiber	Top fiber	Bottom fiber	Top fiber	Bottom fiber
1 Primary prestressing (initial prestress after short-term losses)	[Stress]	[Stress]				
2 Primary prestressing (effective prestress after short-term and long-term losses)					[Stress]	[Stress]
3 Dead load: self-weight of girder	[Stress]	[Stress]			[Stress]	[Stress]
4 Dead load: self-weight of deck and cross beams					[Stress]	[Stress]
5 Secondary prestressing (initial prestress after short-term losses)			[Stress]	[Stress]	[Stress]	[Stress]
6 Secondary prestressing (effective prestress after short-term and long-term losses)			[Stress]	[Stress]	[Stress]	[Stress]
7 Dead load: self-weight of pavement, barrier, median strip, curb, and sidewalk			[Stress]	[Stress]	[Stress]	[Stress]
8 Live load			[Stress]	[Stress]	[Stress]	[Stress]
Sum (1+3)	[Sum]	[Sum]				
Stress limit (for 1+3)	[Stress limit]	[Stress limit]				
Safety check (for 1+3)	O.K or N.G	O.K or N.G				
Sum (2+3+4)					[Sum]	[Sum]
Stress limit (for 2+3+4)					[Stress limit]	[Stress limit]
Safety check (for 2+3+4)					O.K or N.G	O.K or N.G
Sum (2+3+4+5)			[Sum]		[Sum]	[Sum]
Stress limit (for 2+3+4+5)			[Stress limit]		[Stress limit]	[Stress limit]
Safety check (for 2+3+4+5)			O.K or N.G		O.K or N.G	O.K or N.G

Table 24 *(continued)*

Construction stage	Stress					
	Non-composite section		Composite section			
	Girder		Deck		Girder	
	Top fiber	Bottom fiber	Top fiber	Bottom fiber	Top fiber	Bottom fiber
Sum (2+3+4+6+7+8)			[Sum]	[Sum]	[Sum]	[Sum]
Stress limit (for 2+3+4+6+7+8)			[Stress limit]	[Stress limit]	[Stress limit]	[Stress limit]
Safety check (for 2+3+4+6+7+8)			O.K or N.G	O.K or N.G	O.K or N.G	O.K or N.G

10.2.1 For temporary stresses before losses-fully prestressed components

10.2.1.1 Compression stresses

The compressive stress limit for pretensioned and post-tensioned concrete components, including segmentally constructed bridges, shall be 0,60 f_{ci}' (MPa).

10.2.1.2 Tension stresses

The limits in Table 25 shall apply for tensile stresses.

Table 25 — Temporary tensile stress limits in prestressed concrete before losses-fully prestressed components

Bridge type	**Location**	**Stress limit** (MPa)
Other than segmentally constructed bridges	— In precompressed tensile zone without bonded reinforcement	N/A
	— In areas other than the precompressed tensile zone and without bonded reinforcement	$0{,}25\sqrt{f_{ci}'} \leq 1{,}38$
	— In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0,5 f_y, not to exceed 210 MPa.	$0{,}63\sqrt{f_{ci}'}$

Table 25 *(continued)*

Bridge type	Location	Stress limit (MPa)
Segmentally constructed bridges	Longitudinal stresses through joints in the precompressed tensile zone — Joints with minimum bonded auxiliary reinforcement through the joints, which is sufficient to carry the calculated tensile force at a stress of 0,5 f_y; with internal tendons or external tendons — Joints without the minimum bonded auxiliary reinforcement through the joints	 $0{,}25\sqrt{f_{ci}'}$ maximum tension No tension
	Transverse stresses through joints — For any type of joint	 $0{,}25\sqrt{f_{ci}'}$
	Stresses in other areas — For areas without bonded non-prestressed reinforcement — In areas with bonded reinforcement (reinforcing bars or prestressing steel) sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0,5 f_y, not to exceed 210 MPa.	 No tension $0{,}50\sqrt{f_{ci}'}$
	Principal tensile stress at neutral axis in web — All types of segmental concrete bridges with internal and/or external tendons, unless the owner imposes other criteria for critical structures	 $0{,}289\sqrt{f_{ci}'}$

10.2.2 For stresses at serviceability limit state after losses-fully prestressed components

10.2.2.1 Compression stresses

The limits in Table 26 shall apply.

Table 26 — Compressive stress limits in prestressed concrete at serviceability limit state after losses-fully prestressed components

Location	Stress limit (MPa)
— In other than segmentally constructed bridges due to the sum of effective prestress and permanent loads	$0{,}45\sqrt{f_c'}$
— In segmentally constructed bridges due to the sum of effective prestress and permanent loads	$0{,}45\sqrt{f_c'}$
— Due to the sum of effective prestress, permanent loads, and transient loads as well as during shipping and handling	$0{,}60\sqrt{f_c'}$

10.2.2.2 Tension stresses

The limits in Table 27 shall apply.

Table 27 — Tensile stress limits in prestressed concrete at serviceability limit state after losses-fully prestressed components

Bridge type	Location	Stress limit (MPa)
Other than segmentally constructed bridges	Tension in the precompressed tensile zone bridges, assuming uncracked sections	
	— For components with bonded prestressing tendons or reinforcement that are subjected to not worse than moderate corrosion conditions	$0{,}50\sqrt{f_c{'}}$
	— For components with bonded prestressing tendons or reinforcement that are subjected to severe corrosive conditions	$0{,}25\sqrt{f_c{'}}$
	— For components with unbonded prestressing tendons	No tension
Segmentally constructed bridges	Longitudinal stresses through joints in the precompressed tensile zone	
	— Joints with minimum bonded auxiliary reinforcement through the joints sufficient to carry the calculated longitudinal tensile force at a stress of 0,5 f_y; internal tendons or external tendons	$0{,}25\sqrt{f_c{'}}$
	— Joints without the minimum bonded auxiliary reinforcement through joints	No tension
	Transverse stresses through joints	
	— Tension in the transverse direction in precompressed tensile zone	$0{,}25\sqrt{f_c{'}}$
	Stresses in other areas	
	— For areas without bonded reinforcement	No tension
	— In areas with bonded reinforcement sufficient to resist the tensile force in the concrete computed assuming an uncracked section, where reinforcement is proportioned using a stress of 0,5 f_y, not to exceed 210 MPa.	$0{,}50\sqrt{f_c{'}}$
	Principal tensile stress at neutral axis in web	
	— All types of segmental concrete bridges with internal and/or external tendons, unless the owner imposes other criteria for critical structures	$0{,}289\sqrt{f_c{'}}$

11 Loss of prestress

11.1 Total loss of prestress

Values of prestress losses specified herein shall be applicable for specified concrete strengths up to 105 MPa.

Prestress losses in members constructed and prestressed in a single stage, relative to the stress immediately before transfer, may be taken as per Formulae (23) and (24):

In pretensioned members:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \tag{23}$$

In post-tensioned members:

$$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT} \tag{24}$$

11.2 Instantaneous losses

11.2.1 Anchorage set

The magnitude of the anchorage set shall be that required to control the stress in the prestressing steel at transfer or that recommended by the manufacturer of the anchorage, whichever the greater. The magnitude of the set assumed for the design and used to calculate set loss shall be shown in the contract documents and verified during construction.

Formula (25) can be used to obtain the anchorage set loss at anchorage, which is linearly reduced to 0 along the distance l_{set}, obtained by Formula (26), from the anchorage.

$$\Delta f_{pA} = \sqrt{\frac{4pE_p\Delta l}{A_{ps}}} \quad (25)$$

$$l_{set} = \sqrt{\frac{E_p A_{ps}\Delta l}{p}} \quad (26)$$

11.2.2 Friction

11.2.2.1 Pretensioned construction

For draped prestressing tendons, losses that may occur at the hold-down devices should be considered.

11.2.2.2 Post-tensioned construction

Losses due to friction between the internal prestressing tendons and duct wall may be taken as per Formula (27):

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-(Kx+\mu\alpha)}\right) \quad (27)$$

Losses due to friction between the external tendon across a single deviator pipe may be taken as per Formula (28):

$$\Delta f_{pF} = f_{pj}\left(1 - e^{-\mu(\alpha+0{,}04)}\right) \quad (28)$$

Values of K and μ should be based on experimental data for the materials specified and shall be shown in the contract documents. In the absence of such data, a value within the ranges of K and μ as specified in Table 28 may be used.

For tendons confined to a vertical plane, α shall be taken as the sum of the absolute values of angular changes over length x.

For tendons curved in three dimensions, the total tridimensional angular change α shall be obtained by vectorially adding the total vertical angular change, α_v, and the total horizontal angular change, α_h.

Table 28 — Friction coefficients for post-tensioning tendons

Type of steel	Type of duct	K (/mm)	μ (/rad)
Wire or strand	Rigid and semirigid galvanized metal sheathing	[6,6 × 10^{-7}]	[0,15~0,25] (Recommended value: 0,20)
	Polyethylene	[6,6 × 10^{-7}]	[0,23]
	Rigid steel pipe deviators for external tendons	[6,6 × 10^{-7}]	[0,25]

Table 28 *(continued)*

Type of steel	Type of duct	K (/mm)	μ (/rad)
High-strength bars	Galvanized metal sheathing	[6,6 × 10^-7]	[0,30]

11.2.3 Elastic shortening

11.2.3.1 Pretensioned members

The loss due to elastic shortening in pretensioned members shall be taken as per Formula (29):

$$\Delta f_{\mathrm{pES}} = \frac{E_{\mathrm{p}}}{E_{\mathrm{ct}}} f_{\mathrm{cgp}} \tag{29}$$

The total elastic loss or gain may be taken as the sum of the effects of prestress and applied loads.

11.2.3.2 Post-tensioned members

The loss due to elastic shortening in post-tensioned members, other than slab systems, may be taken as per Formula (30):

$$\Delta f_{\mathrm{pES}} = \frac{N-1}{2N} \frac{E_{\mathrm{p}}}{E_{\mathrm{ci}}} f_{\mathrm{cgp}} \tag{30}$$

f_{cgp} values may be calculated using a steel stress reduced below the initial value by a margin dependent on elastic shortening, relaxation, and friction effects.

For post-tensioned structures with bonded tendons, f_{cgp} may be taken at the centre section of the span or, for continuous construction, at the section of maximum moment.

For post-tensioned structures with unbonded tendons, the f_{cgp} value may be calculated as the stress at the centre of gravity of the prestressing steel averaged along the length of the member.

11.2.3.3 Combined pretensioning and post-tensioning

In applying the provisions of 11.2.3.1 and 11.2.3.2 to components with combined pretensioning and post-tensioning, and where post-tensioning is not applied in identical increments, the effects of subsequent post-tensioning on the elastic shortening of previously stressed prestressing tendons shall be considered.

11.3 Approximate estimate of time-dependent losses

Approximate estimate of time-dependent losses may be determined by either Method 1 or Method 2. Method 1 shall only be applicable to pretensioned members, while Method 2 can be applicable to both pretensioned and post-tensioned members.

Method 1

For standard precast, pretensioned members subjected to normal loading and environmental conditions, where:

— members are made from normal-density concrete;

— the concrete is either steam- or moist-cured;

— prestressing is by bars or strands with normal and low relaxation properties; and

— average exposure conditions and temperatures characterize the site.

The long-term prestress loss, Δf_{pLT}, due to creep of concrete, shrinkage of concrete, and relaxation of steel shall be estimated using Formula (31):

$$\Delta f_{pLT} = 10 \frac{f_{pi} A_{ps}}{A_g} \gamma_h \gamma_{st} + 83 \gamma_h \gamma_{st} + \Delta f_{pR} \tag{31}$$

where

$$\gamma_h = 1{,}7 - 0{,}01 H_r \tag{32}$$

$$\gamma_{st} = \frac{35}{(7 + f_{ci}')} \tag{33}$$

Method 2

The time-dependent losses may be calculated by considering the following two reductions of stress.

— due to time-dependent losses that may be calculated by considering concrete creep and shrinkage, under the permanent loads; and

— the reduction of stress in the steel due to the relaxation under tension.

The time-dependent losses at location x under the permanent loads shall be estimate using Formula (34):

$$\Delta f_{pLT} = \frac{\varepsilon_{cs} E_p + 0{,}8 \Delta f_{pR} + \frac{E_p}{E_c} \varphi(t,t_0) f_{c,QP}}{1 + \frac{E_p}{E_c} \cdot \frac{A_{ps}}{A_c} (1 + \frac{A_c}{I_c} z_{cp}^2)[1 + 0{,}8 \varphi(t,t_0)]} \tag{34}$$

where

A_{ps} is the area of all the prestressing tendons at the location x;

A_c is the area of the concrete section;

E_p is the modulus of elasticity for the prestressing steel;

E_c is the modulus of elasticity for the concrete;

$f_{c,QP}$ is the stress in the concrete adjacent to the tendons, due to self-weight and initial prestress and other quasi-permanent actions where relevant;

I_c is the second moment of area of the concrete section;

z_{cp} is the distance between the centre of gravity of the concrete section and the tendons;

Δf_{pR} is the absolute value of the variation of stress in the tendons at location x, at time t, due to the relaxation of the prestressing steel;

Δf_{pLT} is the absolute value of the variation of stress in the tendons due to creep, shrinkage and relaxation at location x, at time t;

ε_{cs} is the estimated shrinkage strain in absolute value at time t;

$\varphi(t, t_0)$ is the creep coefficient at a time t and load application at time t_0.

Compressive stresses and the corresponding strains given in Formula (34) should be used with a positive sign.

Formula (34) applies for bonded tendons when local values of stresses are used and for unbonded tendons when mean values of stresses are used. The mean values should be calculated between straight sections limited by the idealized deviation points for external tendons or along the entire length in case of internal tendons.

ε_{cs} after the end of initial wet curing, e.g. after 7 days for moist cured concrete and after 1~3 days for steam cured concrete, can be obtained by Formulae (35) and (36), respectively.

$$\varepsilon_{cs} = \frac{t}{35+t}\varepsilon_{csu} \tag{35}$$

$$\varepsilon_{cs} = \frac{t}{55+t}\varepsilon_{csu} \tag{36}$$

where

t is the time in days;

ε_{csu} = [780]×10^{-6} m/m.

The shrinkage that occurs after the end of the initial wet curing and before tensioning shall not be considered in calculation of prestress loss.

$\varphi(t,t_0)$ for a prestressing age t_0 of 7 days for moist cured concrete and 1~3 days for steam cured concrete can be obtained by Formula (37).

$$\varphi(t,t_0) = \frac{t^{0,6}}{10+t^{0,6}}\varphi_u \tag{37}$$

where

t is the time in days after prestressing;

φ_u = [2,35].

If the prestressing age is 14 days or 28 days, φ_u shall be adjusted to [2,0] or [1,76], respectively.

Δf_{pR} can be obtained by Formulae (38) and (39) for post-tensioned members and pretensioned members, respectively.

$$\Delta f_{pR} = f_{pi}\frac{\log_{10} t}{10}\left(\frac{f_{pi}}{f_{py}} - 0,55\right) \tag{38}$$

$$\Delta f_{pR} = f_{pi}\frac{\log_{10} t - \log_{10} t_r}{10}\left(\frac{f_{pi}}{f_{py}} - 0,55\right) \tag{39}$$

where

t is the time in hours;

t_r is the time of prestress release in hours for pretensioned members.

The shrinkage strain, creep and relaxation loss can be estimated for each construction stage based on Formulae (35)-(39) as the following examples of Table 29 and Table 30.

Table 29 — Standard construction stage and long-term losses of post-tensioned girder bridge

Construction stage	Time month	Accumulated time month	Shrinkage	Creep	Relaxation
Girder casting[a]	0	0	0	0	0
Primary prestressing	1	1	0	0	0
Deck slab casting	1	2	0,21	0,43	0,61
Secondary prestressing (optional)	1	3	0,31	0,54	0,68
Completion of bridge	2	5	0,41	0,64	0,75
In service (5 years later)[b]	—	60	0,60[c]	1,0[c]	1,0[c]

[a] Moist cured concrete for 7 days is assumed.

[b] Although the exact values corresponding to a certain time can be calculated from Formulae (35) to (39), 5 years can be regarded as a sufficiently long time.

[c] 1,0 means 100 % of shrinkage strain, creep coefficient and relaxation loss.

Table 30 — Standard construction stage and long-term losses of pretensioned girder bridge

Construction stage	Time month	Accumulated time month	Shrinkage	Creep	Relaxation
Primary pre-stressing and girder casting[a]	0	0	0	0	0
Prestress release	0,1	0,1	0	0	0
Deck slab casting	1	1,1	0,35	0,43	0,36
Secondary prestressing (optional)	1	2,1	0,52	0,54	0,47
Completion of bridge	2	4,1	0,69	0,64	0,58
In service (5 years later)[b]	—	60	1,0[c]	1,0[c]	1,0[c]

[a] Steam cured concrete for 3 days is assumed.

[b] Although the exact values corresponding to a certain time can be calculated from Formulae (35) to (39), 5 years can be regarded as a sufficiently long time.

[c] 1,0 means 100 % of shrinkage strain, creep coefficient and relaxation loss.

12 Details of tendon

12.1 Tendon confinement

12.1.1 General

Tendons shall be located within the reinforcing steel stirrups in webs, and, where applicable, between layers of transverse reinforcing steel in flanges. For ducts in the bottom flanges of variable depth