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**Simplified design of prestressed  
concrete bridges —**

**Part 2:  
Box-girder bridges**

*Conception simplifiée des ponts en béton précontraint —  
Partie 2: Ponts à poutres caissons*

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## 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](http://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](http://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](http://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](http://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 box 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 in 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.

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# Simplified design of prestressed concrete bridges —

## Part 2: Box-girder bridges

### 1 Scope

This document provides information to perform the design of the prestressed concrete box girder bridge for road that complies with the limitations established in 6.1. The rules of design as set forth in the document are simplifications of more elaborate requirements. Among several erection methods of box girder bridges, the provisions of this document are mainly applicable to full staging method (FSM).

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 terminology 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 B-region

portion of a member in which it is reasonable to assume that strains due to flexure vary linearly through section

**3.4**

**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, and subjected primarily to flexure

**3.5**

**clearance**

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

**3.6**

**compression reinforcement**

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

**3.7**

**specified compressive strength of concrete**

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  $f_c'$  is 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).

**3.8**

**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

**3.9**

**cover**

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

**3.10**

**crosstie**

continuous reinforcing bar having a 135° hook 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.

**3.11**

**D-region**

portion of a member with a force discontinuity or a geometric discontinuity

**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 according to 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 multiplied by a strength reduction factor

**3.14**

**development length**

length of embedded reinforcement required to develop the design strength of reinforcement at a critical section

**3.15****diaphragm**

transverse stiffener which is provided between girders in order to maintain section geometry, to resist lateral forces, and to transfer loads to support

**3.16****duct**

material creating a conduit in a concrete member to accommodate the prestressing steel of a post-tensioning tendon

**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****factored load**

specified nominal load multiplied by the load factors prescribed in this document

**3.20****girder**

main horizontal support beam, usually supporting other beams

**3.21****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.

**3.22****jacking force**

<prestressed concrete> temporary force exerted by the device that introduces tension into the tendons

**3.23****joist**

T-shaped beam used in parallel series directly supporting deck loads, and supported in turn by larger girders, beams, or bearing structural concrete walls

**3.24****lap splice**

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

**3.25****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)

**3.26****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

**3.27****load effect**

force and deformation produced in structural members by the applied loads

**3.28**

**load combination**

combination of factored loads and forces as specified in this document

**3.29**

**load factor**

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

**3.30**

**load**

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

**3.31**

**longitudinal reinforcement**

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

**3.32**

**mesh wire**

welded-wire fabric reinforcement

**3.33**

**nominal load**

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

**3.34**

**nominal strength**

capacity of a structure or member to resist the effects of loads, 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.

**3.35**

**permanent load**

load in which variations over time are rare or of small magnitude

Note 1 to entry: All other loads are variable loads (see also [3.33](#)).

**3.36**

**post-tensioning**

method of prestressing reinforced concrete in which tendons are tensioned after the concrete has attained a specified minimum strength or a specified minimum age

**3.37**

**prestressing steel**

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

**3.38**

**pretensioning**

method of prestressing in which prestressing steel is tensioned before the concrete is placed

**3.39**

**required factored strength**

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

**3.40****service load**

load specified by this document (without load factors)

**3.41****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

**3.42****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

**3.43****slab**

deck

upper flat part of a reinforced concrete deck carried by supporting joists or beams or girders

**3.44****spiral reinforcement**

continuously wound reinforcement in the form of a cylindrical helix

**3.45****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. (The term "stirrups" is usually applied to lateral reinforcement in girders, beams, and joists; the term "ties" to those in columns and walls, perhaps because they are intended also as confinement for the longitudinal reinforcement). See also [3.48](#).

**3.46****strength reduction factor**

coefficient that accounts for deviations of the actual strength from the nominal strength, 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 being considered, and to reflect the importance of the element in the structure.

**3.47****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

**3.48****tie**

loop of reinforcing bar or wire enclosing longitudinal reinforcement

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.

**3.49****transfer length**

the length from the end of the member where the tendon stress is zero to the point along the tendon where the prestress is fully effective

**3.50**

**transverse reinforcement**

reinforcement located perpendicular to the longitudinal axis of the element, comprising stirrups, ties, spiral reinforcement, among others

**3.51**

**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
$a_{eff}$	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
$A$	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 <sup>2</sup>
$A_a$	fraction of acceleration of gravity	—
$A_b$	effective bearing area	mm <sup>2</sup>
$A_c$	area of concrete section	mm <sup>2</sup>
$A_{conf}$	bearing area of the confined concrete in local zone	mm <sup>2</sup>
$A_g$	gross area of section of element	mm <sup>2</sup>
$A_{plate}$	anchor bearing plate area	mm <sup>2</sup>
$A_{ps}$	area of prestressing steel	mm <sup>2</sup>
$A_{ps,s}$	area of prestressing steel corresponding to concrete top slab	mm <sup>2</sup>
$A_s$	area of longitudinal tension reinforcement	mm <sup>2</sup>
$A_s'$	area of longitudinal compression reinforcement	mm <sup>2</sup>
$A_{s,st}$	area of shrinkage and temperature reinforcement in each direction and each face	mm <sup>2</sup> /mm
$A_v$	area of shear reinforcement (stirrup) within a distance $s$	mm <sup>2</sup>
$b$	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
$b_w$	web width of girder or beam	mm
$d$	effective depth of reinforcement	mm
$d'$	distance from extreme compression fibre 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
$d_{ce}$	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
$e_a$	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

Symbol	Explanation	Unit
$E_b$	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
$f'_{cd}$	compressive strength of concrete reduced by the material factor	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
$f_{cpe}$	compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fibre of section where tensile stress is caused by externally applied loads	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
$f_{pe}$	effective stress in the prestressing steel after losses	MPa
$f_{pi}$	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
$f_r$	modulus of rupture of concrete	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
$F_a$	site coefficient	—
$F_{u-in}$	in-plane deviation force effect per unit length of tendon	N/mm
$F_{u-out}$	out-of-plane force effect per unit length of tendon	N/mm
$g$	acceleration of gravity	m/s <sup>2</sup>
$h$	overall depth or thickness of the member	mm
$h_s$	height or thickness of top slab	mm
$h_l$	lateral dimension of the cross-section in the direction considered	mm
$H$	the largest height of bridge supports	m
$H_r$	average annual ambient relative humidity	%
$I$	second moment of area	mm <sup>4</sup>
$I_c$	second moment of area of concrete	mm <sup>4</sup>
$I_D$	second moment of area of deck	m <sup>4</sup>
$I_p$	second moment of area of wall, frame or pier	m <sup>4</sup>
$K$	wobble friction coefficient	/mm

Symbol	Explanation	Unit
$l_c$	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
$L$	span length	m
$L_T$	total length	m
$m_T$	total mass	kg
$M_{cr}$	cracking moment	N·mm
$M_{dnc}$	total unfactored dead load moment acting on the monolithic or non-composite section	N·mm
$M_n$	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	—
$p$	slope of prestressing force distribution due to friction	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	—
$S_c$	section modulus for the extreme fibre of the composite section where tensile stress is caused by externally applied loads	mm <sup>3</sup>
$S_{nc}$	section modulus for the extreme fibre of the monolithic or non-composite section where tensile stress is caused by externally applied loads	mm <sup>3</sup>
$t$	time	days or h
$t_b$	average thickness of the bearing plate	mm
$t_r$	time of prestress release	h
$T_{burst}$	tensile force in the anchorage zone acting ahead of the anchorage device and transverse to the tendon axis	N
$V_c$	contribution of the concrete to the nominal shear strength at section	N
$V_n$	nominal shear strength at section	N
$V_r$	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
$V_u$	factored shear force at section	N
$w_e$	equivalent lateral force	kN/m
$w_s$	seismic uniformly distributed load	kN/m
$x$	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
$\alpha$	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

Symbol	Explanation	Unit
$\alpha_t$	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
$\gamma$	load factor	—
$\gamma_{mc}$	material factor for concrete	—
$\gamma_{ms}$	material factor for steel	—
$\Delta f_{pA}$	prestress loss due to anchorage set	MPa
$\Delta f_{pES}$	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
$\Delta f_{pF}$	prestress loss due to friction	MPa
$\Delta f_{pLT}$	prestress losses due to long-term shrinkage and creep of concrete, and relaxation of the steel	MPa
$\Delta f_{pR}$	prestress loss due to relaxation of a strand	MPa
$\Delta f_{pT}$	total loss of prestress	MPa
$\Delta l$	anchorage set	mm
$\epsilon_{cs}$	shrinkage strain	m/m
$\epsilon_{csu}$	ultimate shrinkage strain	m/m
$\mu$	curvature friction coefficient	/rad
$\phi$	strength reduction factor	—
$\varphi(t, t_0)$	creep coefficient	—
$\varphi_u$	ultimate creep coefficient	—

## 5 Design and construction procedure

### 5.1 Procedure

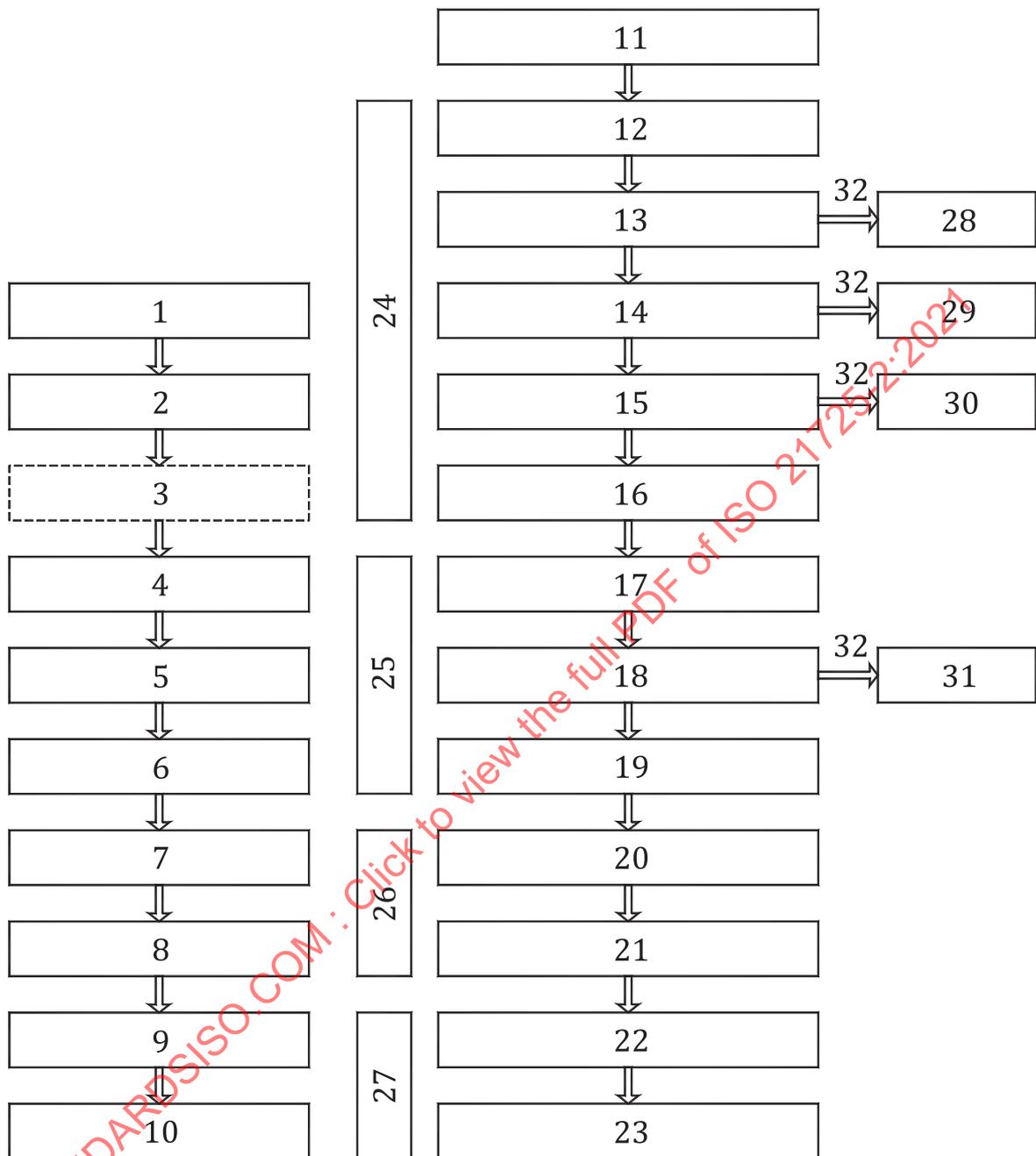
#### 5.1.1 General

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

- Step A: 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.
- Step B: Calculation of all gravity loads that act on the structure using the provisions of [Clause 8](#), excluding the self-weight of the structural elements.
- Step C: Trial dimensions for the girders and calculation of their self-weight. Flexural and shear design of the girders, correcting the dimension as required by the ultimate and serviceability limit states.
- Step D: Trial dimensions for the substructure system 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.
- Step E: 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 G.
- Step F: 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.

- g) Step G: 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.
- h) Step H: Production of structural drawings.
- i) Step I: Construction of the structure should be performed complying with the local construction and practice.

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**Key**

- |   |  |    |   |
|---|--|----|---|
| 1 | step A: preliminary design of structure                | 17 | structural modelling and analysis                                   |
| 2 | step B: definition of loads                            | 18 | check of allowable stress (serviceability limit state)              |
| 3 | construction stage analysis (FSM: full staging method) | 19 | check of flexural strength and minimum rebar (ultimate limit state) |
| 4 | step C: design of girders                              | 20 | design of central diaphragm   |
| 5 | step D: design of substructure                         | 21 | design of end diaphragm   |
| 6 | step E: definition of lateral forces                   | 22 | design of local zone  |
| 7 | step F: design of foundations                          | 23 | design of general zone  |
| 8 | step G: design of structural walls                     | 24 | longitudinal design   |
| 9 | step H: structural drawings                            | 25 | transverse design   |

10	step I: construction of structure	26	design of diaphragm
11	design conditions and assumption of section	27	design of anchorage zone
12	structural modelling and analysis	28	redesign of prestressing tendons
13	check of stress limitations (serviceability limit state)	29	redesign of section and prestressing tendons
14	check of deflection (serviceability limit state)	30	redesign of section and prestressing tendons
15	flexural design (ultimate limit state)	31	redesign of section
16	shear and torsion (ultimate limit state)	32	N.G. (no good)

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, the following:

- the general structural requirements of the project, as required by 7.1;
- a description of the structural system employed;
- loads and assumptions employed;
- grade, strength and fabrication standards for all structural materials;
- presentation of all design computations; and
- sketches of the reinforcement layout for all structural elements.

### 5.2.3 Geotechnical report

The geotechnical report should record, as a minimum, the soil investigation performed, the definition of the allowable bearing capacity of the bearing soil, 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 using 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 Recommended span length

The recommended span length is from [40] m to [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 should comply with [10.3.3](#) 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.

#### 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 Clearance 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, or if curbs are not used, over the entire width that is available for traffic.

#### 6.1.10.4 Clearance 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 [30]°.

#### 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 cross-section along the continuous portions of the bridge, except in cantilever sections.

#### 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

The design approach of this 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.

### 6.3 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 \(1\)](#):

$$R \geq S \tag{1}$$

where

$R$  is the resistance;

$S$  is the load effect.

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,  $\gamma$ , 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 of concrete, or both, by means of dividing these values by the corresponding material factors [see [Formula \(2\)](#)]:

$$R = f \left( \frac{f_c'}{\gamma_{mc}}, \frac{f_y}{\gamma_{ms}} \right) \geq \gamma_1 S_1 + \gamma_2 S_2 + \gamma_3 S_3 + \dots \tag{2}$$

where

$R$  is the strength;

$S$  is the load effect based on the nominal loads prescribed by this document.

Therefore, the ultimate limit state design format shall meet [Formula \(3\)](#) or [Formula \(4\)](#):

$$R_d \geq U \tag{3}$$

$$R_d = f \left( \frac{f_c'}{\gamma_{mc}}, \frac{f_y}{\gamma_{ms}} \right) \geq U \tag{4}$$

where

$R_d$  is the design strength;

$U$  is the required factored strength;

$$U = \gamma_1 S_1 + \gamma_2 S_2 + \dots$$

#### 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 4](#):

**Table 4 — Material factor**

Material	Factor
Concrete, $\gamma_{mc}$	[1,5]
Steel, $\gamma_{ms}$	[1,5]

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 consists of the structural elements that comprise the portion of the bridge that directly receive the live load. The superstructure system corresponds to the box girder in box girder bridges. 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 15](#).

#### 7.1.4 Foundation

The foundation comprises all structural elements that serve to transmit loads from the structure to the underlying supporting soil, or 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;

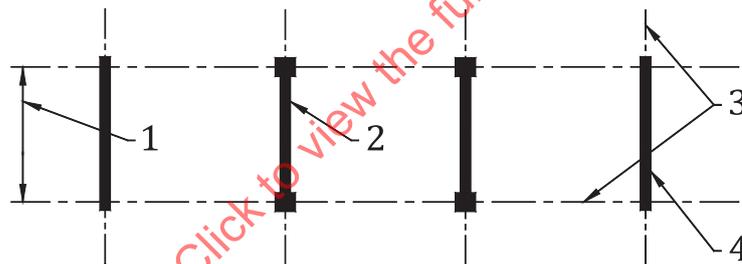
- availability, type, and quality of materials such as reinforcing steel, cement and aggregates;
- availability of materials for formwork erection;
- availability of testing facilities for concrete mix design and quality control during construction; and
- availability of qualified workforce.

### 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

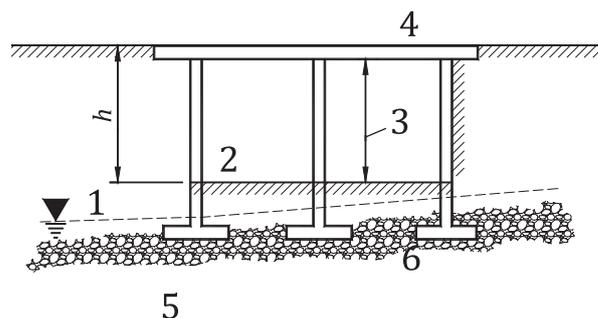
- 1 deck width
- 2 pier, frame or wall
- 3 centrelines grid
- 4 abutment

**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**

**7.4 Feasibility under the guidelines**

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<sup>3</sup>. 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 5](#) may be used for weight estimate.

**Table 5 — Density values for materials used in bridge construction**

Material	Density kg/m <sup>3</sup>
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**

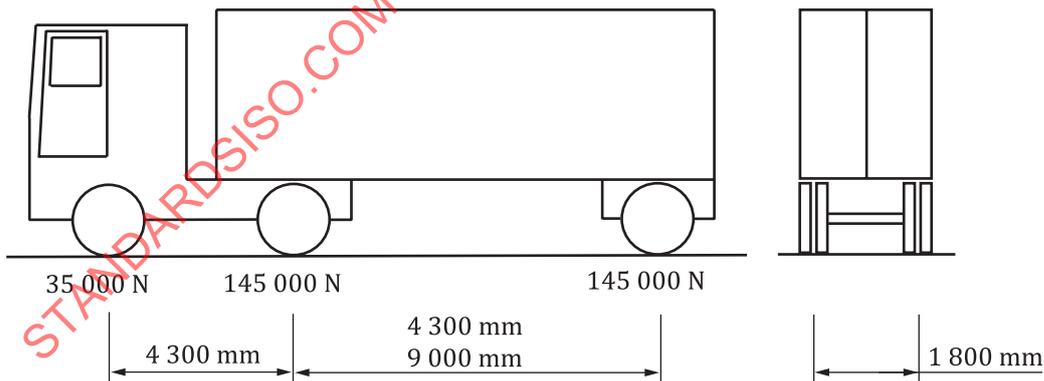
**8.3.1 General**

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**

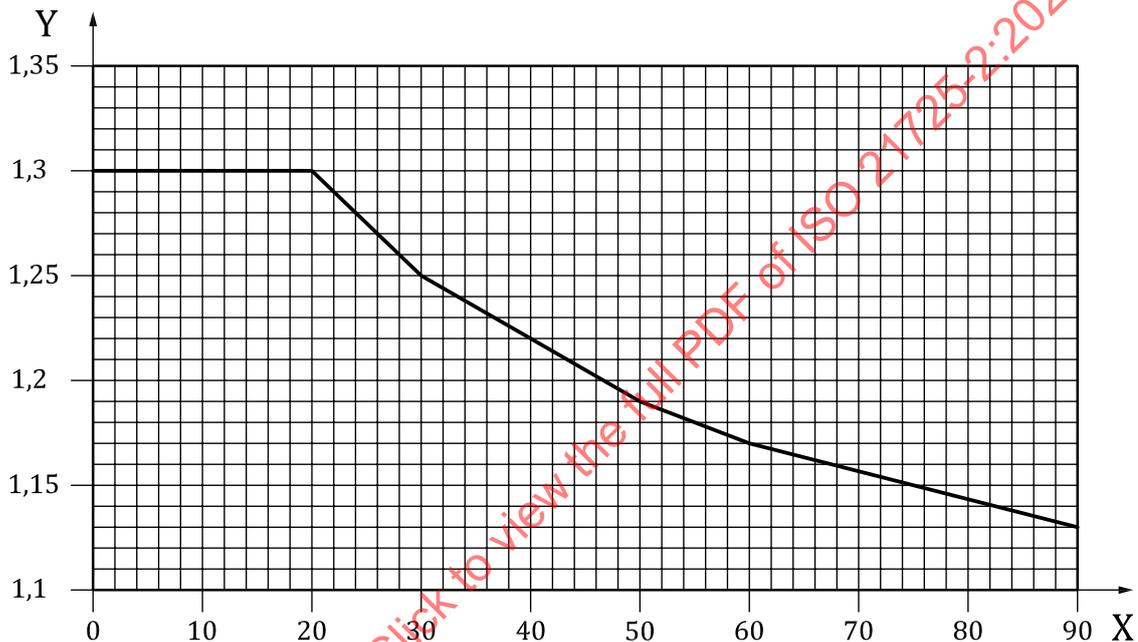
**8.3.3 Design lane load**

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 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 5](#) 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

X loaded length, m

Y live load dynamic effect factor

**Figure 5 — 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<sup>3</sup>.

### 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 6.

**Table 6 — Wind loads for hurricane, cyclone or typhoon prone areas**

Load condition	Load direction	Load kN/m <sup>2</sup>
Load on structure <sup>a</sup>	Transverse	[2,5]
	Longitudinal	[0,6]
Load on live load <sup>a</sup>	Transverse	[1,5]
	Longitudinal	[0,4]
<sup>a</sup> 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 5 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<sup>2</sup>).

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 6.

#### 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:**

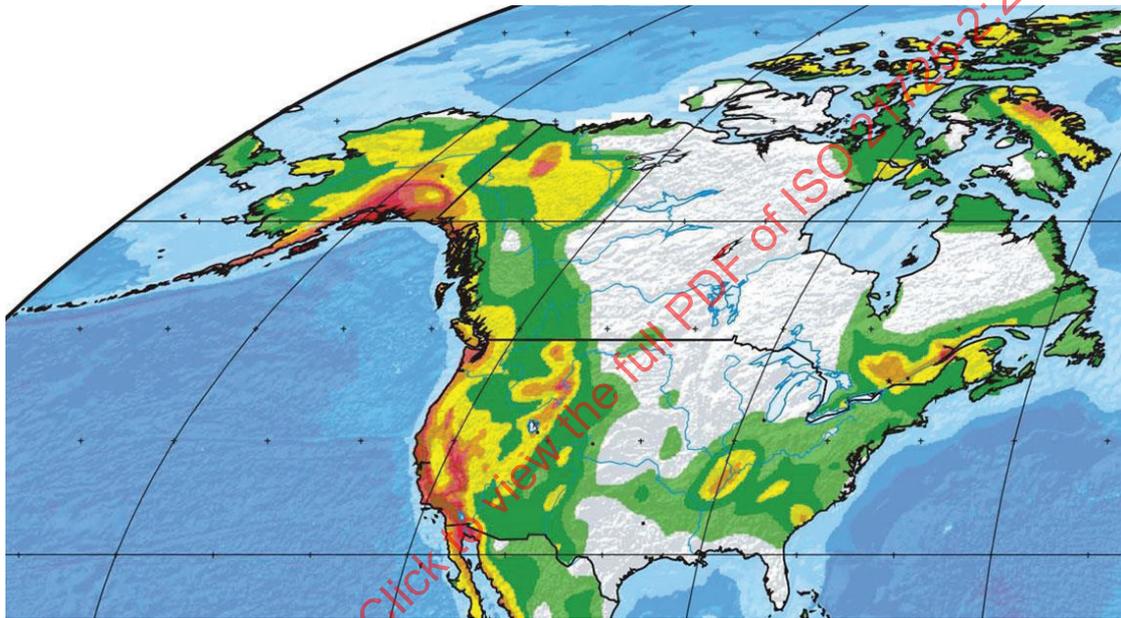
A zone where the value of  $A_a$  is estimated as more than [0,05] but less or equal to [0,1] may be deemed as a *low seismic hazard zone*.

**8.7.5 Intermediate seismic hazard zones:**

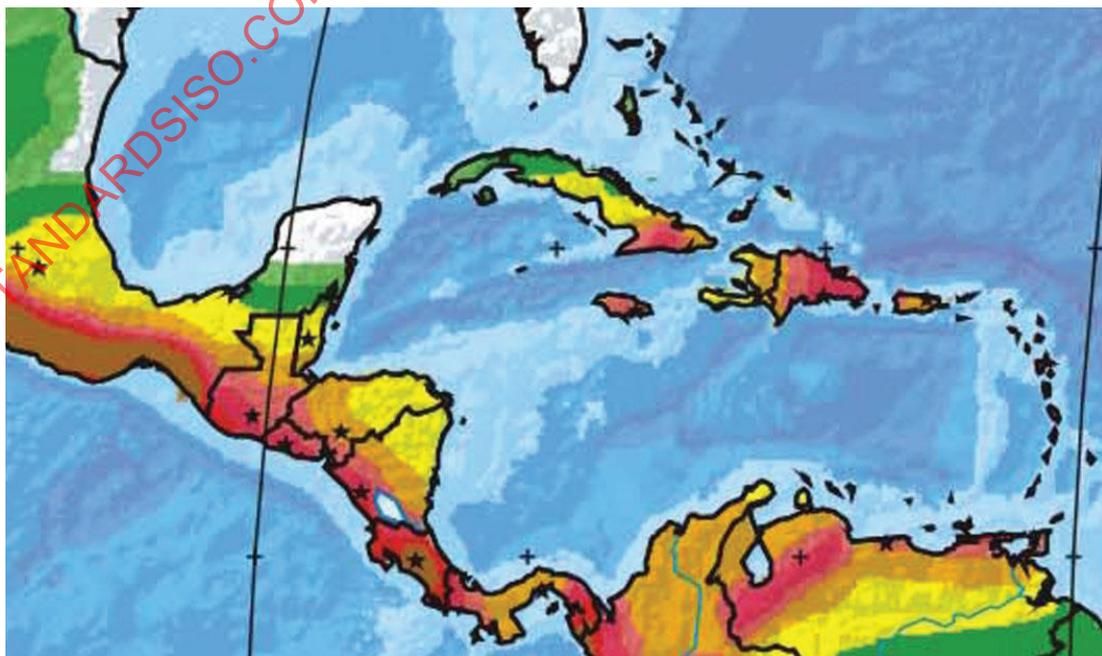
A zone where the value of  $A_a$  is estimated as more than [0,1] but less or equal to [0,2] may be deemed as an *intermediate seismic hazard zone*.

**8.7.6 High seismic hazard zones:**

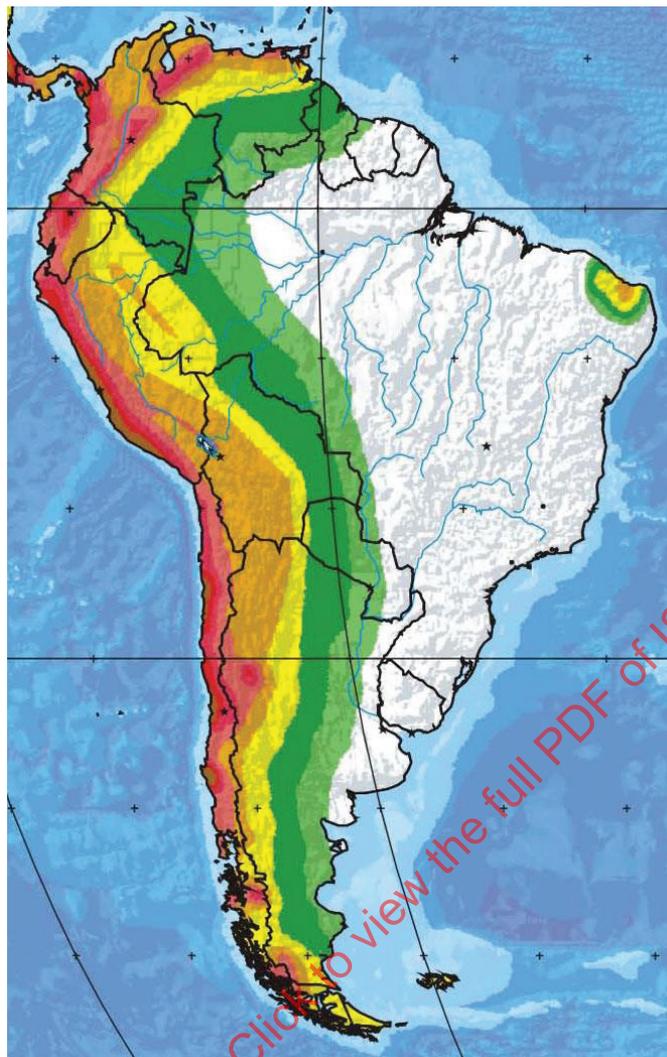
A zone where the estimated value of  $A_a$  exceeds [0,2] may be deemed as a *high seismic hazard zone*.



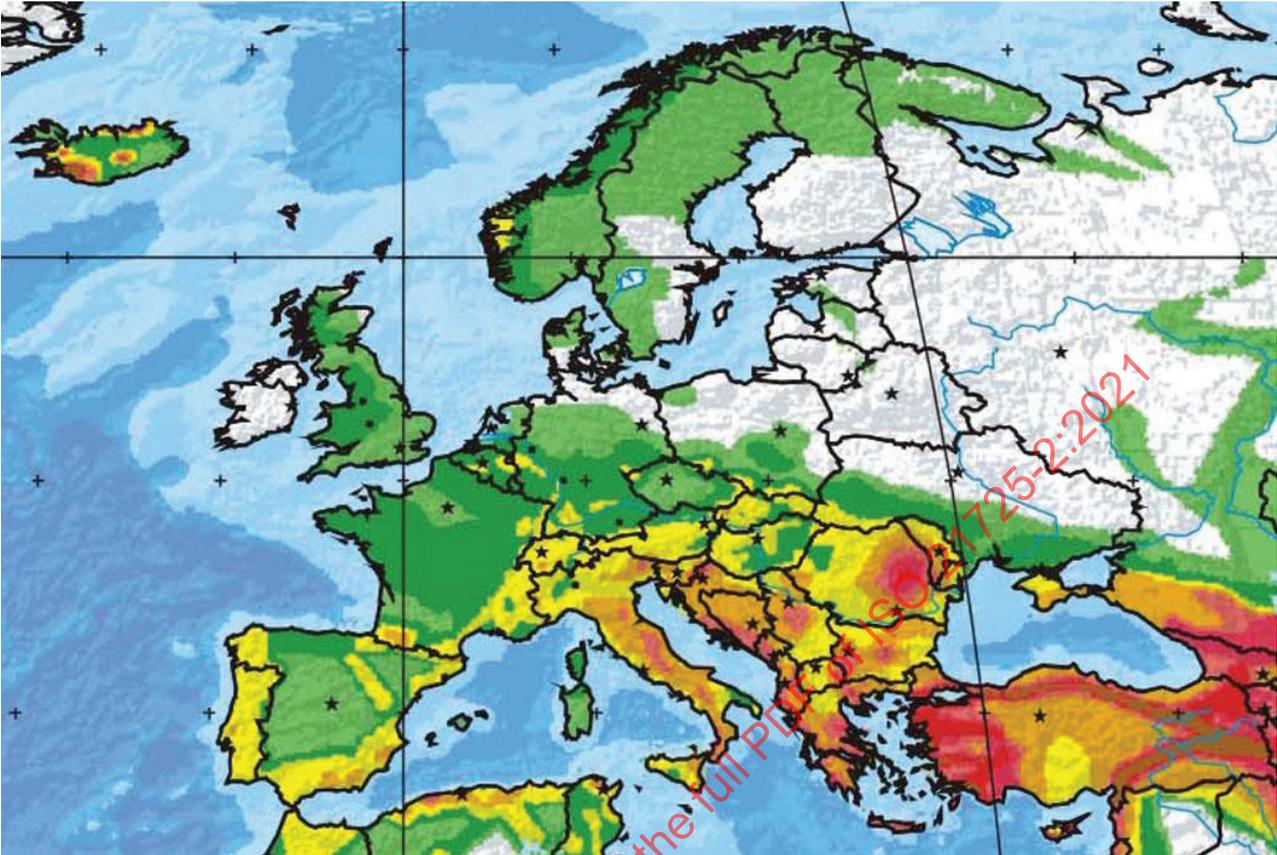
a) North America



b) Central America and the Caribbean



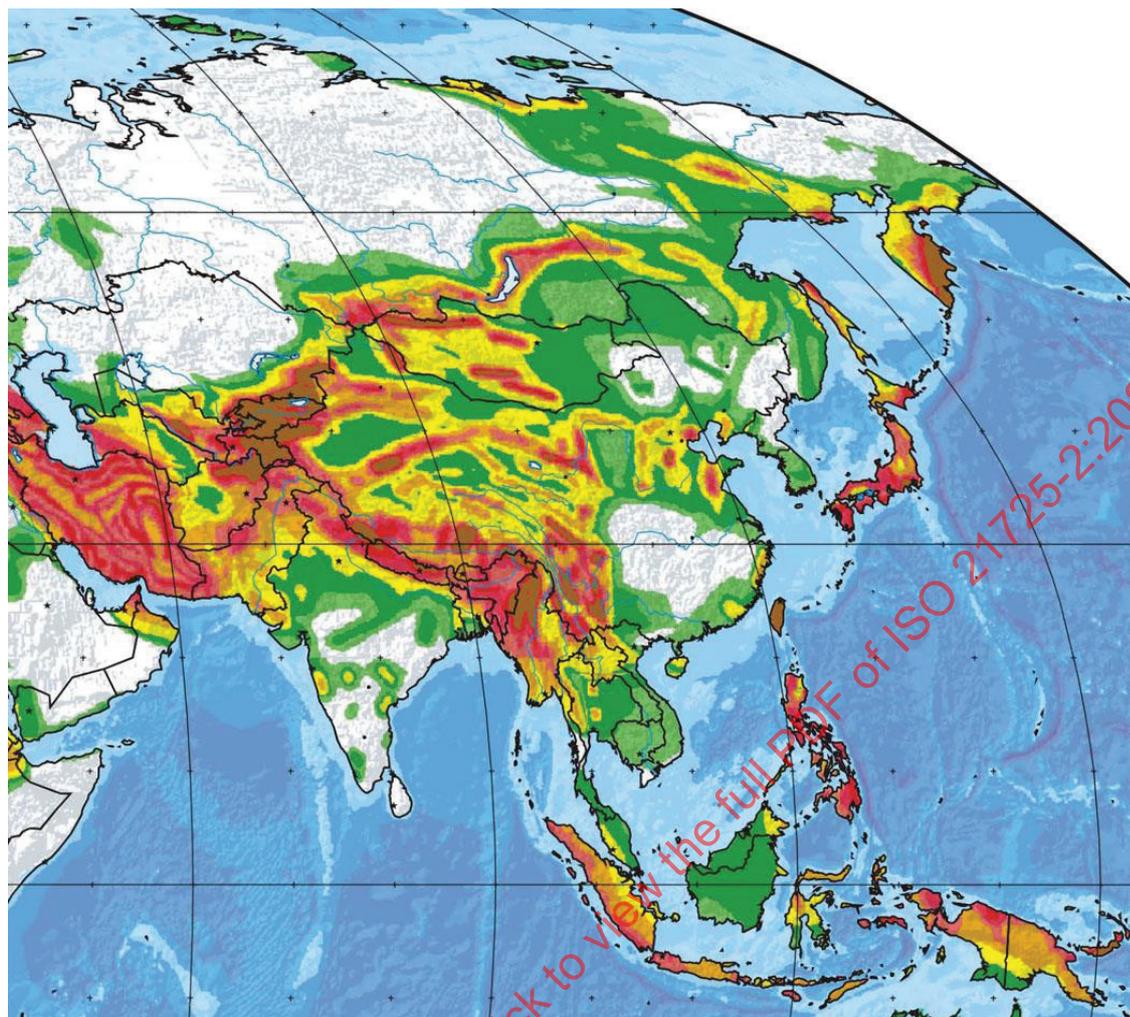
c) South America



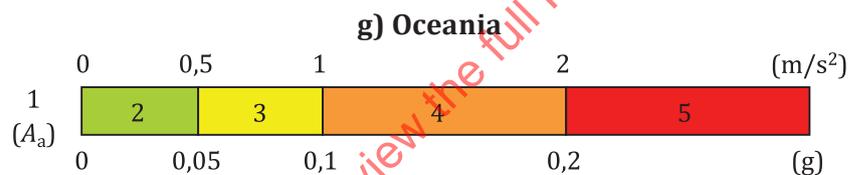
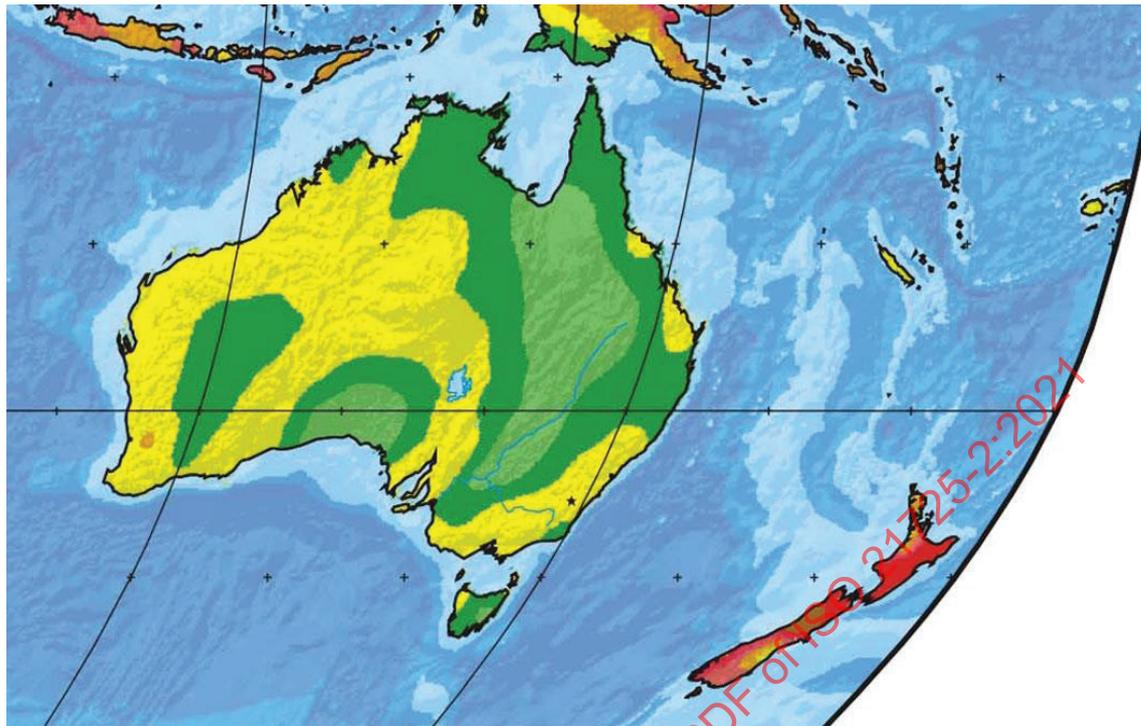
d) Europe



e) Africa



f) Asia

**Key**

- 1 seismic hazard
- 2 none
- 3 low
- 4 intermediate
- 5 high

**Figure 6 — 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<sub>A</sub>: hard rock with a measured shear wave velocity  $v_s > 1\,500$  m/s;
- soil profile S<sub>B</sub>: rock with moderate fracturing and weathering with a measured shear wave velocity in the range ( $1\,500$  m/s  $\geq v_s > 750$  m/s);
- soil profile S<sub>C</sub>: soft weathered or fractured rock, or dense or stiff soil, where the measured shear wave velocity is in the range ( $750$  m/s  $\geq v_s > 350$  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  $s_u \geq 100$  kPa;
- soil profile S<sub>D</sub>: predominately medium-dense to dense, or medium stiff to stiff soil, where the measured shear wave velocity is in the range ( $350$  m/s  $\geq v_s > 180$  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 ( $50$  kPa  $\leq s_u < 100$  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 7](#) 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 7](#).

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 7 — 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 \(5\)](#):

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

### 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 \(6\)](#):

$$w_s = \frac{m_T g S_a}{L_T} \quad (6)$$

where

$g$  is the force of gravity;

$L_T$  is the total length of the bridge;

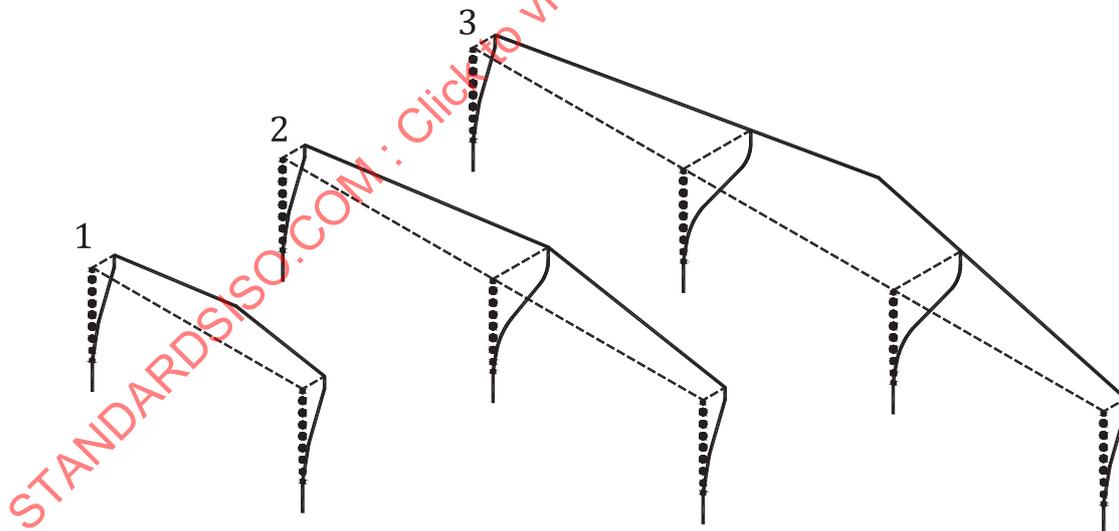
$m_T$  is the total mass of the bridge that is not directly absorbed by the supports, including all structural elements, intermediate walls, piers, columns, etc., excluding footings and abutments;

$S_a$  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 7](#).



#### Key

- 1 one span
- 2 two spans
- 3 three spans

**Figure 7 — Fundamental mode shape**

The fundamental shape for each case and for each span is described by a function  $u(x)$ , as per [Table 8](#).

**Table 8 — Unitary deformation distribution**

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

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 \(7\)](#):

$$w_e = w_s u(x) \tag{7}$$

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 effects

### 8.9.1 Seasonal variation

- For the purpose of design of a structure, the minimum and maximum overall temperatures for the location shall be taken from local data.
- The temperature setting variations for bearings and expansion joints shall be stated on the bridge plans.

### 8.9.2 Thermal coefficient

Coefficient of thermal expansion of concrete used to determine temperature effects shall be taken as  $[1,0] \times 10^{-5}$  m/m/°C unless more precise data is available.

### 8.9.3 Differential temperature

Positive and negative differential temperature gradient in superstructure should be considered in accordance with References [19], [20] or [24] unless local data is available.

## 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 9.

**Table 9 — Load amplification factors and load combinations**

Group	Amplification factor, <i>A</i>				
	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 9 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 Structural analysis

### 9.1 General

#### 9.1.1 B-region and D-region

In general, strain varies linearly through the depth of a member and that, as a result, plane sections remain plane. The strut-and-tie model should be considered for the design of deep footings and pile caps or other situations in which the distance between the centres of applied load and the supporting reactions is less than about twice the member thickness.

#### 9.1.2 Elastic behaviour

For the purpose of analysis, structural materials shall be considered to behave linearly up to an elastic limit and inelastically thereafter.

#### 9.1.3 Small deflection theory

If the deformation of the structure does not result in a significant change in force effects due to an increase in the eccentricity of compressive or tensile forces, such secondary force effects may be ignored.

#### 9.1.4 Secondary moments

The effect of secondary moments due to prestressing shall be included in stress calculations at the service limit state. At the strength limit state, the secondary force effects induced by prestressing, with a load factor of 1,0, shall be added algebraically to other applicable factored loads.

#### 9.1.5 Time-dependent properties of concrete

In general, force effects due to changes in time-dependent properties, such as creep and shrinkage of concrete, shall be accounted for serviceability limit state design.

#### 9.1.6 Geometric imperfections

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

#### 9.1.7 Frame analysis

In general, plane frame analysis shall be allowed for structural analysis.

### 9.2 Longitudinal analysis

The structural system shall be analyzed for redistribution of construction-stage force effects due to internal deformations and changes in support and restraint conditions, including accumulated locked-in force effects resulting from the construction process.

Joints in segmental girders made continuous by unbonded post-tensioning steel shall be investigated for the simultaneous effect of axial force, moment, and shear that may occur at the joint. These force effects, the opening of the joint, and the remaining contact surface between the components shall be determined by global consideration of strain and deformation. Shear shall be assumed to be transmitted through the contact area only.

### 9.3 Transverse analysis

The cross-section of a concrete box girder bridge is designed to resist bending moments acting transverse to the longitudinal direction of the bridge span. Transverse analyses are performed to evaluate transverse bending moments resulting from both permanent and live loads. The magnitude and distribution of transverse bending moments due to permanent and live loads vary along the length of a span as a function of the deflection behaviour of the box girder. Within the span, general longitudinal deflections reduce maximum transverse moments as they are distributed along a greater portion of the span. There are three levels of analysis for determining transverse design moments.

#### 9.3.1 Empirical method

This method, presented in Article 9.7 of References [19] and [20], presents required top slab reinforcing for bridges meeting the criteria of Article 9.7.2.4 [19] [20]. No guidance for reinforcing or forces in the remainder of the cross-section is presented. The use of this method of analysis is not recommended for concrete box girders presented in this document.

#### 9.3.2 Approximate method

This method presents an approximate method of analysis, where the deck is subdivided into strips perpendicular to the supporting elements (webs). Internal forces are determined first by structural analysis of a unit width of transverse cross-section and then by dividing these force and moment results by the width of the strip to determine the forces per metre along the superstructure.

Figure 8 a) shows a perspective of a span with the cross-section under consideration. A typical cross-section with a unit length is identified in the span for a two-dimensional analysis in the plane of the cross-section. The extracted typical section for analysis is shown in Figure 8 b).

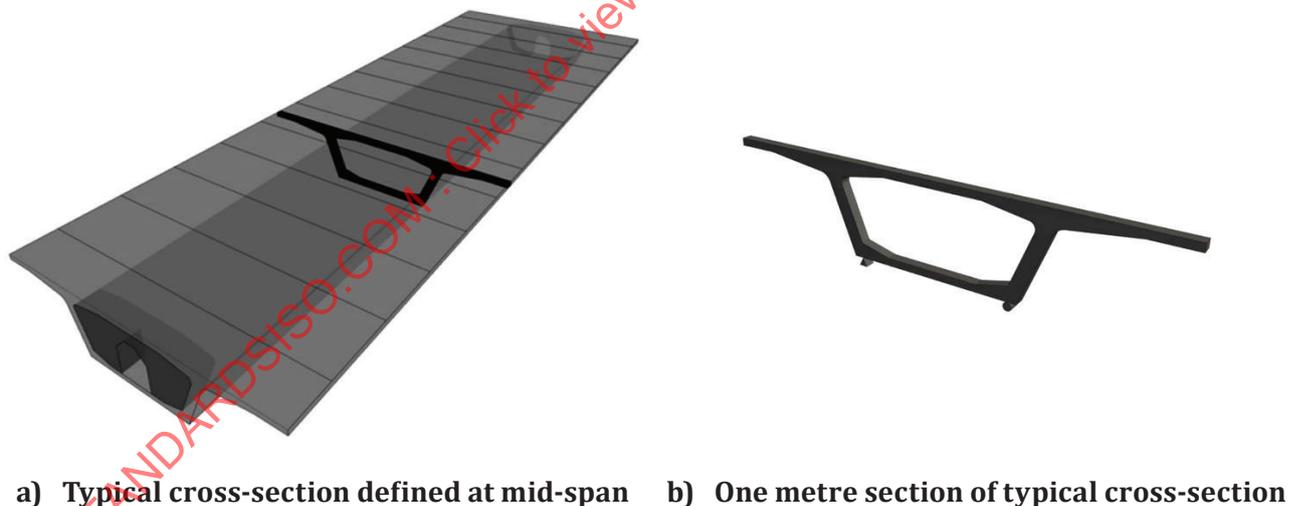
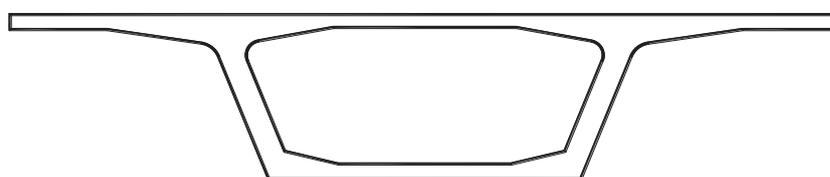


Figure 8 — Transverse strip for approximate design method

The typical cross-section is modelled using beam elements in a general two-dimensional structural analysis program. Figure 9 shows the transition from typical cross-section, to idealized beam members, to a node and element layout for a typical analysis model.



a) Typical cross-section

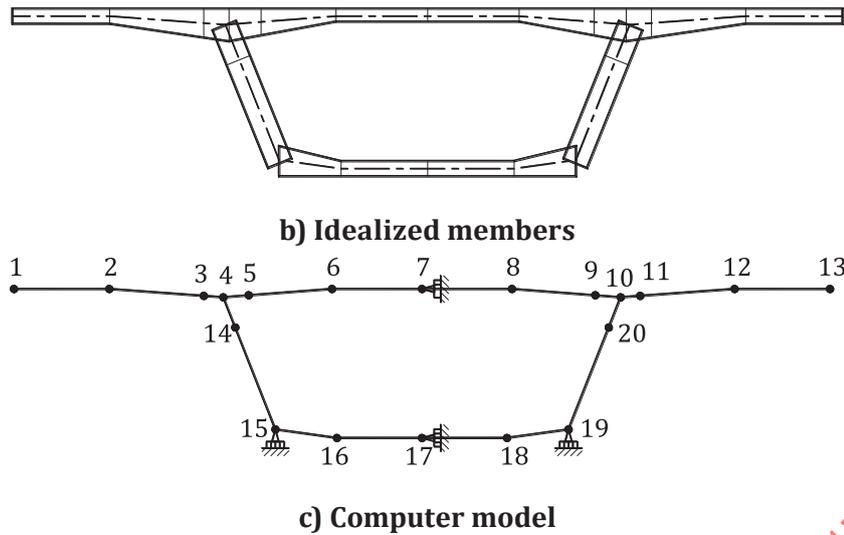


Figure 9 — Development of two-dimensional transverse model

### 9.3.2.1 Analysis for uniformly repeating loads

Transverse bending moments for loads that repeat uniformly along the span can be determined directly from the two-dimensional analysis mode.

### 9.3.2.2 Analysis for concentrated live loads

Transverse bending moments resulting from the application of concentrated loads in the span are determined in a three-step process:

- determine maximum transverse bending moments at critical sections of the cantilever wing and top slab considering these members as separate three dimensional, fixed-end slab structures;
- use the two-dimensional model to distribute "fixed-end" slab moments around the cross-section model; and
- sum the fixed-end moments and redistributed moments to determine live load moments for design;

The fixed-end slab structures are typically analysed using influence surfaces such as those by Homberg [25] and Pucher [26]. Influence surface is adequate for deck transverse design.

### 9.3.3 Refined method

Prestressed concrete box girder bridges are complex spatial structures, and their behaviour can be only roughly approximated using frame structural analysis program. This type of analysis can give an indication of the longitudinal distribution of total reactions, internal total stress resultants, and centreline deflections, but cannot provide any information on the transverse distribution of these quantities or the internal forces in the individual plate type elements making up the cellular cross-section.

Many computer programs are available for bridge analysis. Various methods of analysis, ranging from simple formulae to detailed finite element procedure, are implemented in such programs.

Two representative methods of analysis make it possible to perform detailed linear elastic analyses of a wide range of prestressed concrete box girder bridges in an expeditious and accurate manner with a minimum of simplifying assumptions. Folded plate method is used for the analysis of straight prismatic bridges, whereas finite strip method is for the analyses of circularly curved prismatic bridges.

Typical aspects of the prestressing analysis (tendon profile definitions, computation of short-term prestress losses due to friction and anchorage slip, and computation of equivalent loads on the structure) are common to the programs by two methods.

When using a computer program, the designer should clearly understand the basic assumptions of the program and the methodology that is implemented. A computer program is only a tool, and the user is responsible for the generated results. Accordingly, all output should be verified to the possible extent.

Another typical approach is to use three-dimensional finite element method with sufficient load cases to generate an envelope of transverse design moments. Most often, however, the practicality of detailing and constructing superstructures with uniform reinforcing and post-tensioning details typically overrides small quantity savings that the refined methods can offer.

## 10 Design requirements

### 10.1 General

This 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 and prestressing steel, and basic dimension criteria of box girder.

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

### 10.2 Box girder cross-section dimensions and details

#### 10.2.1 Minimum flange thickness

Top and bottom flange thickness shall not be less than any of the following:

- 1/30 the clear span between webs or haunches. A lesser dimension requires transverse ribs at a spacing equal to the clear span between webs or haunches; and
- top flange thickness shall not be less than 225 mm in anchorage zones where transverse post-tensioning is used and 200 mm beyond anchorage zones or for pretensioned slabs.

Transverse post-tensioning or pretensioning shall be used where the clear span between webs or haunches is 4 500 mm or larger. Strands used for transverse pretensioning shall be 12,7 mm diameter or less.

#### 10.2.2 Minimum web thickness

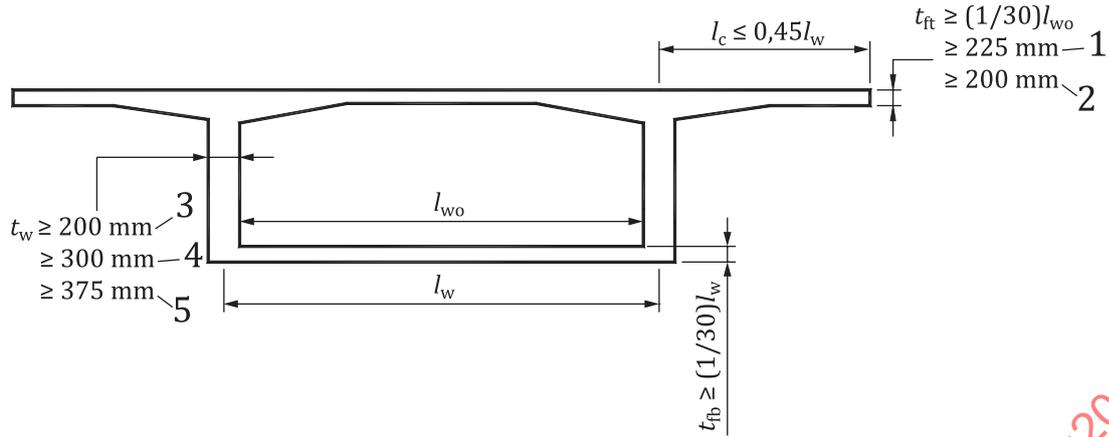
The following minimum values shall apply, except as specified in this document:

- webs with no longitudinal or vertical post-tensioning tendons: 200 mm;
- webs with only longitudinal (or vertical) post-tensioning tendons: 300 mm; and
- webs with both longitudinal and vertical tendons: 375 mm.

The minimum thickness of ribbed webs may be taken as 175 mm.

#### 10.2.3 Length of top flange cantilever

The cantilever length of the top flange measured from the centreline of the web should preferably not exceed 0,45 the interior span of the top flange measured between the centreline of the webs as shown in [Figure 10](#).



**Key**

- 1 anchorage zone
- 2 beyond anchorage zone
- 3 webs with no longitudinal or vertical post-tensioning tendons
- 4 webs with only longitudinal (or vertical) post-tensioning tendons
- 5 webs with both longitudinal and vertical tendons

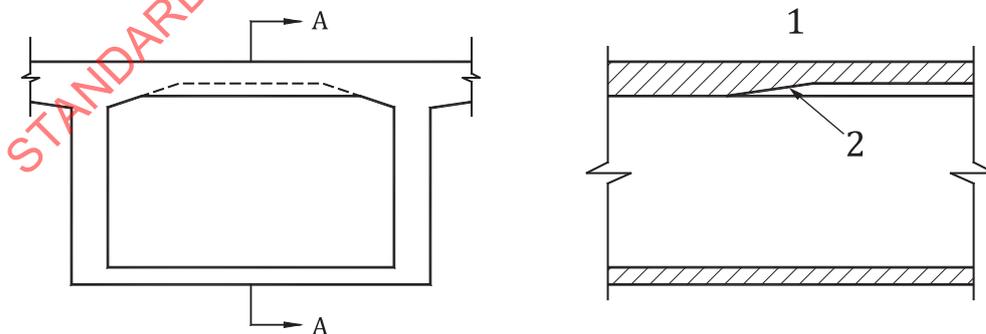
**Figure 10 — Cross-section dimensions**

**10.2.4 Overall cross-section dimensions**

Overall dimensions of the box girder cross-section should preferably not be less than those required to limit live load plus impact deflection calculated using the gross section moment of inertia and the secant modulus of elasticity to 1/1 000 of the span. The live loading shall consist of all traffic lanes fully loaded and adjusted for the number of loaded lanes. The live loading shall be considered to be uniformly distributed to all longitudinal flexural members.

**10.2.5 Longitudinal slope**

- Flange transitions in thickness shall be made on a slope no greater than 1:5 as shown in [Figure 11](#).
- Changes in girder web thickness shall be tapered for a minimum distance of 12,0 times the difference in web thickness.



**Key**

- 1 section A-A
- 2 slope no greater than 1:5

**Figure 11 — Flange transitions in thickness**

## 10.3 Materials for structural concrete

### 10.3.1 General

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

### 10.3.2 Cement

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

### 10.3.3 Aggregates

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

### 10.3.4 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.

### 10.3.5 Steel reinforcement

#### 10.3.5.1 General

Steel reinforcement should conform to the following limitations and comply with the corresponding ISO standards, especially ISO 10144. Welded-wire fabric (mesh wire) should be considered deformed reinforcement according to this document.

#### 10.3.5.2 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, 8, 10, 12, 16, 20, 25, 32, and 40) mm, although under this document the nominal diameter of deformed reinforcement bars is limited to 32 mm (see [10.3.11](#)).

#### 10.3.5.3 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. In this document, the nominal diameter of wire for welded-wire fabric is limited to 10 mm (see [10.3.11](#)).

#### 10.3.5.4 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 mm, 8 mm, 10 mm, 12 mm, 16 mm and 20 mm, although in this document the nominal diameter of plain reinforcement bars is limited to 16 mm (see [10.3.11](#)).

### 10.3.6 Prestressing steel

Tensile and yield strengths for prestressing steel may be taken as specified in [Table 10](#). 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 this 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 10 — Properties of prestressing strand and bar**

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

### 10.3.7 Post-tensioning anchorages and couplers

#### 10.3.7.1 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.

#### 10.3.7.2 Bonded systems

Bond transfer lengths between anchorages and the zone where full prestressing force is required under service and ultimate loads shall 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 tendons.

#### 10.3.7.3 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 utilizing 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.

Anchorage 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.

### 10.3.8 Ducts

#### 10.3.8.1 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.

#### 10.3.8.2 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.

### 10.3.9 Admixtures

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

### 10.3.10 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.

**10.3.11 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 11](#).

**Table 11 — Minimum and maximum reinforcing bar diameters**

Reinforcement	Minimum bar diameter, $d_b$ mm	Maximum bar diameter, $d_b$ mm
Deformed reinforcing bars (see <a href="#">10.3.5.2</a> )	[8]	[32]
Prestressing steel – wire (see <a href="#">10.3.6</a> )	[9,3] for 1 720 MPa strand [9,5] for 1 860 MPa strand	[15,2]
Prestressing steel – bar (see <a href="#">10.3.6</a> )	[20]	[36]
Wire for welded-wire fabric (see <a href="#">10.3.5.3</a> )	[4]	[10]
For stirrups and ties	[6]	[16]
Plain reinforcing bars (see <a href="#">10.3.5.4</a> )	[6]	[16]
For non-seismic areas	[4]	[32]

**10.4 Concrete mixture proportioning**

**10.4.1 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  $f_c'$  should 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.

**10.4.2 Durability requirements**

**10.4.2.1 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.

**10.4.2.2 Calculation of the water-binder ratio**

The water-binder ratios shall be calculated using the weight of water in  $\text{kg/m}^3$  of concrete divided by the binder used in the mixture in  $\text{kg/m}^3$  of concrete. The use of fly ash, pozzolans, slag, and silica fume is beyond the scope of this document and, if used, should be in accordance with ISO 22904 [27].

### 10.4.2.3 Freezing and thawing exposures

Concrete exposed to freezing and thawing or deicing 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  $\pm 1,5$  %.

### 10.4.2.4 Requirements for special exposure conditions

Concrete maximum water-binder ratios and minimum specified compressive strength should comply with specification of [Table 12](#), according to conditions of exposure.

**Table 12 — 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 deicing chemicals	0,45	31,5
For corrosion protection of reinforcement in concrete exposed to chlorides from deicing chemicals, salt, salt water, brackish water, seawater, or spray from these sources	0,4	35

### 10.4.2.5 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.

### 10.4.2.6 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 13](#).

**Table 13 — 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

### 10.4.3 Required average compressive strength

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

### 10.4.4 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 [10.4.2](#) and the slump requirements from [Table 14](#), and that encompass the

required average strength,  $f_{cr}$ . The trial mixtures shall be designed to produce slumps within  $\pm 20$  mm of the maximum permitted.

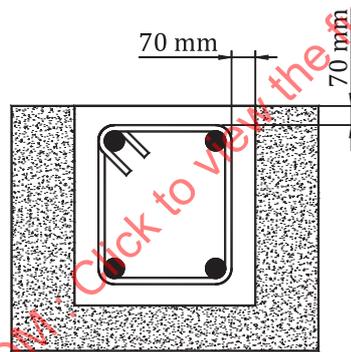
**Table 14 — Slumps for various types of construction**

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

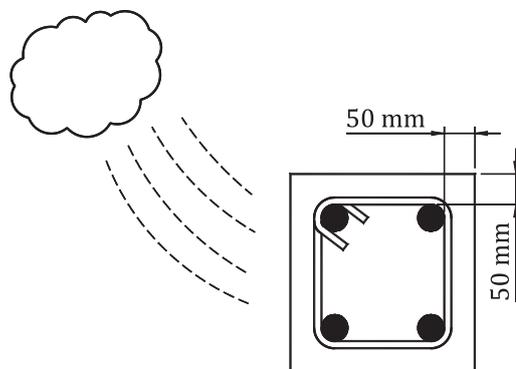
**10.5 Concrete cover of reinforcement**

**10.5.1 Minimum concrete cover**

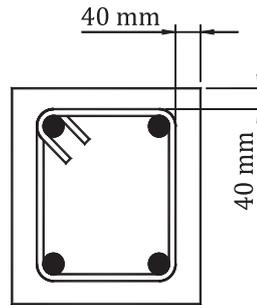
The following minimum concrete cover should be provided for unprotected prestressing and reinforcing steel, even in non-seismic areas (see [Figures 12 to 15](#)).



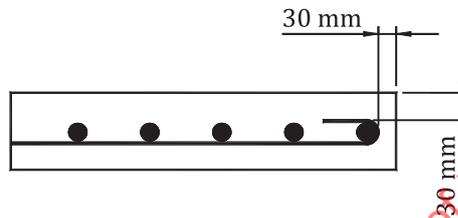
**Figure 12 — All types of reinforcement of elements cast and permanently exposed to earth or water (Minimum concrete cover: 70 mm)**



**Figure 13 — All types of reinforcement of elements exposed to weather (Minimum concrete cover: 50 mm)**



**Figure 14** — 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 15** — 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.

### 10.5.2 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.

### 10.6 Minimum reinforcement bend diameter

ISO 28842 shall apply.

### 10.7 Standard hook dimensions

ISO 28842 shall apply.

### 10.8 Bar spacing and maximum aggregate size

#### 10.8.1 General

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

#### 10.8.2 Maximum nominal coarse aggregate size

Maximum nominal coarse aggregate size (see [Figure 16](#)) should not be larger than:

- 1/5 of the narrowest dimension between sides of forms; nor

- 1/3 of the depth of slabs; nor
- 3/4 the minimum clear spacing between parallel reinforcing bars or wires.

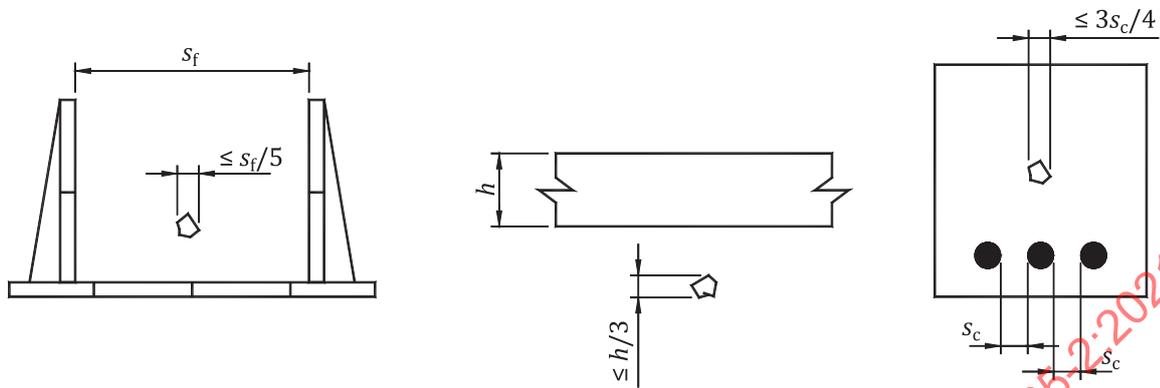


Figure 16 — Maximum nominal coarse aggregate size

**10.8.3 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 17). This document should apply also for the spacing between parallel stirrups or ties.

**10.8.4 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 17).

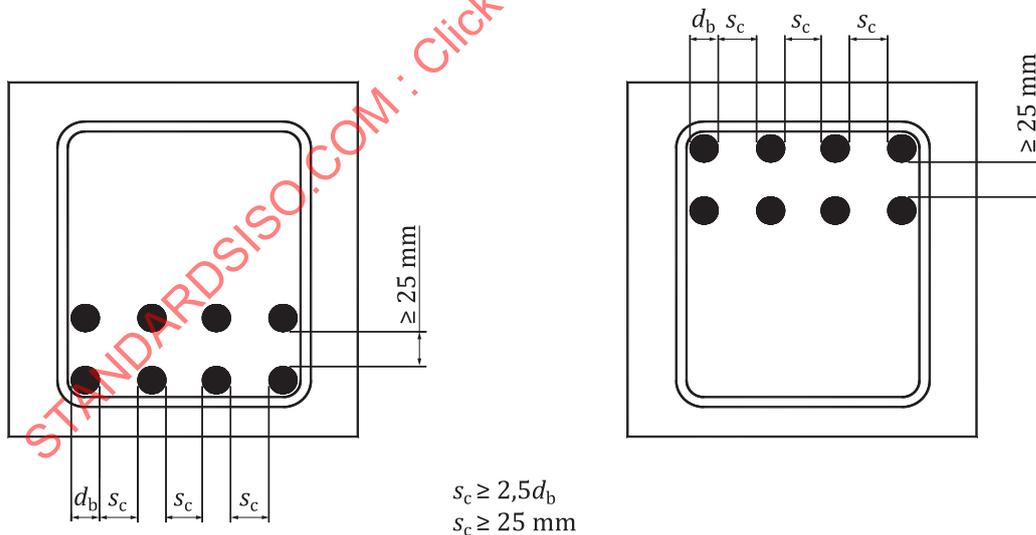


Figure 17 — Minimum clear spacing between parallel bars in a layer, and clear distance between parallel layers of reinforcement

**10.8.5 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.

### 10.8.6 Maximum flexural reinforcement spacing in solid slabs

In solid slabs, primary flexural reinforcement should be spaced no farther apart than two times the slab thickness, nor more than 300 mm (see [Figure 18](#)).

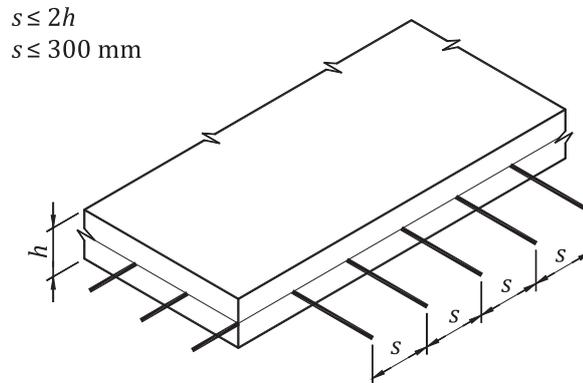


Figure 18 — Spacing between flexural reinforcement in solid slabs

### 10.8.7 Maximum shrinkage and temperature reinforcement spacing in solid slabs

In slabs, shrinkage and temperature reinforcement should be spaced no farther apart than three times the slab thickness, nor more than 300 mm (see [Figure 19](#)).

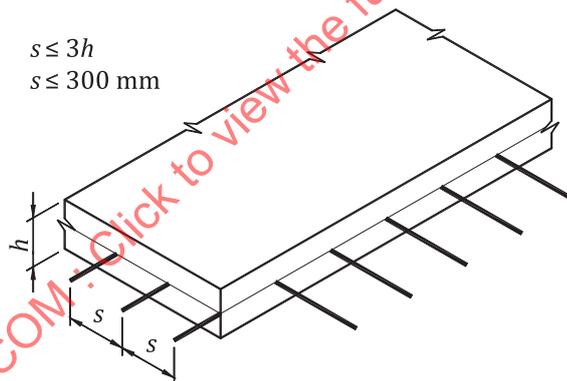


Figure 19 — Spacing between shrinkage and temperature reinforcement in slabs

### 10.8.8 Maximum reinforcement spacing in structural concrete walls

#### 10.8.8.1 Vertical and horizontal reinforcement

In structural concrete walls, vertical and horizontal reinforcement should be spaced no farther apart than three times the structural concrete wall thickness, nor more than 300 mm (see [Figure 20](#)).

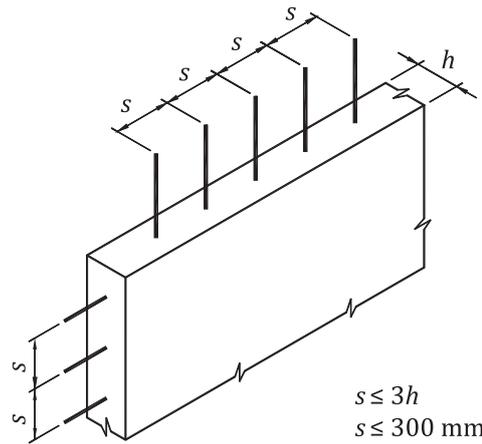


Figure 20 — Spacing between reinforcement in structural concrete walls

**10.8.8.2 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 30 mm.

**10.8.8.3 Special details per element type**

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

**10.8.9 Minimum spacing of prestressing tendons and ducts**

**10.8.9.1 Pretensioning strand**

The distance between pretensioning strands, including shielded ones, at each end of a member within the transfer length, as specified in 10.9.1.3, 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 15.

Table 15 — Centre-to-centre spacings

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

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.

#### 10.8.9.2 Post-tensioning ducts not curved in the horizontal plane

Unless otherwise specified, 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.

#### 10.8.9.3 Curved post-tensioning ducts

The minimum clear distance between curved ducts shall be as required for tendon confinement as specified in [13.1](#). The spacing for curved ducts shall not be less than that required for straight ducts.

#### 10.8.10 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.

#### 10.8.11 Couplers in post-tensioning tendons

The contract documents shall specify that not more than 50 % of the longitudinal post-tensioning tendons be coupled at one section and that the spacing between adjacent coupler locations be 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.

### 10.9 Development length, lap splicing and anchorage of reinforcement

#### 10.9.1 Development length

##### 10.9.1.1 Reinforcing bars

ISO 28842 shall apply.

##### 10.9.1.2 Welded-wire fabric

ISO 28842 shall apply.

### 10.9.1.3 Prestressing strand

#### 10.9.1.3.1 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 [10.9.1.3.2](#).

The effects of debonding shall be considered as specified in [10.9.1.3.3](#).

#### 10.9.1.3.2 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 \(8\)](#):

$$l_d \geq \kappa(0,15 f_{ps} - 0,097 f_{pe}) d_b \quad (8)$$

where:

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

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

#### 10.9.1.3.3 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 \(8\)](#) 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.

### 10.9.2 Lap splice dimensions

ISO 28842 shall apply.

## 10.10 Limits for longitudinal reinforcement

### 10.10.1 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 [10.10](#). 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 permitted by [10.10](#).

### 10.10.2 Shrinkage and temperature reinforcement

Reinforcement for shrinkage and temperature stresses shall be provided near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. Temperature and shrinkage reinforcement shall ensure that the total reinforcement on exposed surfaces is not less than that specified herein.

Reinforcement for shrinkage and temperature may be in the form of bars, welded-wire reinforcement, or prestressing tendons.

For bars or welded-wire reinforcement, the area of reinforcement per mm, on each face and in each direction, shall satisfy [Formula \(9\)](#):

$$A_{s,st} \geq \frac{0,75bh}{2(b+h)f_y} \quad (9)$$

except that:  $0,233 \leq A_{s,st} \leq 1,27$

where

$A_{s,st}$  is the area of reinforcement in each direction and each face ( $\text{mm}^2/\text{mm}$ );

$b$  is the least width of component section (mm);

$f_y$  is the specified minimum yield strength of reinforcement  $\leq 520$  MPa;

$h$  is the least thickness of component section (mm).

Where the least dimension varies along the length of wall, footing, or other component, multiple sections should be examined to represent the average condition at each section. Spacing shall not exceed the following:

- 300 mm for walls and footings greater than 450 mm thick;
- 300 mm for other component greater than 900 mm thick; and
- for all other situations, 3 times the component thickness but not less than 450 mm.

For components 150 mm or less in thickness the minimum steel specified may be placed in a single layer. Shrinkage and temperature steel shall not be required for:

- end face of walls of buried footings 450 mm or less in thickness;
- side faces of buried footings 900 mm or less in thickness; and
- faces of all other components, with smaller dimension less than or equal to 450 mm.

If prestressing tendons are used as steel for shrinkage and temperature reinforcement, the tendons shall provide a minimum average compressive stress of 0,75 MPa on the gross concrete area through which a crack plane may extend, based on the effective prestress after losses. Spacing of tendons should not exceed either 1 800 mm or the distance specified in 10.8.8. Where the spacing is greater than 1 400 mm, bonded reinforcement shall be provided between tendons, for a distance equal to the tendon spacing.

### 10.10.3 Minimum area of tension flexural reinforcement

Minimum reinforcement provisions are intended to reduce the probability of brittle failure by providing flexural capacity greater than the cracking moment.

Appropriate values for  $M_{dnc}$  and  $S_{nc}$  shall be used for any intermediate composite sections. Where the beam is designed for the monolithic or non-composite section to resist all loads,  $S_{nc}$  shall be substituted for  $S_c$  in Formula (10) for the calculation of  $M_{cr}$ .

Unless otherwise specified, at any section of a non-compression-controlled flexural component, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance,  $M_r$ , greater than or equal to the lesser of the following:

- 1,33 times the factored moment required by the applicable strength load combination specified in Table 9; and
- 1,2 times the cracking moment,  $M_{cr}$ , determined on the basis of elastic stress distribution and the modulus of rupture,  $f_r$ , of the concrete, where  $M_{cr}$  may be taken as Formula (10):

$$M_{cr} = S_c (f_r + f_{cpe}) - M_{dnc} \left( \frac{S_c}{S_{nc}} - 1 \right) \geq S_c f_r \quad (10)$$

where

$f_{cpe}$  is the compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fibre of section where tensile stress is caused by externally applied loads (MPa);

$f_r$  is the modulus of rupture of concrete (MPa);

$M_{cr}$  is the cracking moment (N·mm);

$M_{dnc}$  is the total unfactored dead load moment acting on the monolithic or non-composite section (N·mm);

$S_c$  is the section modulus for the extreme fibre of the composite section where tensile stress is caused by externally applied loads (mm<sup>3</sup>); and

$S_{nc}$  is the section modulus for the extreme fibre of the monolithic or non-composite section where tensile stress is caused by externally applied loads (mm<sup>3</sup>).

## 11 Stress limitations

### 11.1 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 16; 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 10](#).

**Table 16 — Stress limits for prestressing tendons**

Condition	Tendon type		
	Stress-relieved strand and plain high-strength bars	Low relaxation strand	Deformed high-strength bars
<b>Pretensioning</b>			
Immediately prior to transfer ( $f_{pi}$ )	$0,70f_{pu}$	$0,75f_{pu}$	—
At serviceability limit state after all losses ( $f_{pe}$ )	$0,80f_{py}$	$0,80f_{py}$	$0,80f_{py}$
<b>Post-tensioning</b>			
Prior to seating: short-term $f_{pi}$ may be allowed	$0,90f_{py}$	$0,90f_{py}$	$0,90f_{py}$
At anchorages and coupler immediately after anchor set	$0,70f_{pu}$	$0,70f_{pu}$	$0,70f_{pu}$
Elsewhere along length of member away from anchorages and couplers immediately after anchor set	$0,70f_{pu}$	$0,74f_{pu}$	$0,70f_{pu}$
At serviceability limit state after losses ( $f_{pe}$ )	$0,80f_{py}$	$0,80f_{py}$	$0,80f_{py}$

## 11.2 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.

### 11.2.1 For temporary stresses before losses-fully prestressed components

#### 11.2.1.1 Compression stresses

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

#### 11.2.1.2 Tension stresses

The limits in [Table 17](#) shall apply for tensile stresses.

**Table 17 — 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 $0,5f_y$ , not to exceed 210 MPa.	$0,63\sqrt{f_{ci}'}$

Table 17 (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 $0,5f_y$ ; with internal tendons or external tendons	$0,25\sqrt{f_{ci}}$ maximum tension
	— Joints without the minimum bonded auxiliary reinforcement through the joints	No tension
	Transverse stresses through joints	
	— For any type of joint	$0,25\sqrt{f_{ci}}$
Stresses in other areas	— For areas without bonded nonprestressed reinforcement	No tension
	— 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 $0,5f_y$ , not to exceed 210 MPa.	$0,50\sqrt{f_{ci}}$
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	$0,289\sqrt{f_{ci}}$

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

11.2.2.1 Compression stresses

The limits in Table 18 shall apply.

Table 18 — 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	$0,60\sqrt{f_c}$

11.2.2.2 Tension stresses

The limits in Table 19 shall apply.

**Table 19 — 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 $0,5f_y$ ; 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 $0,5f_y$ , 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	$0,289\sqrt{f_c'}$

## 12 Loss of prestress

### 12.1 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 \(11\)](#) and [\(12\)](#):

In pretensioned members:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \quad (11)$$

In post-tensioned members:

$$\Delta f_{pT} = \Delta f_{pF} + \Delta f_{pA} + \Delta f_{pES} + \Delta f_{pLT} \quad (12)$$

## 12.2 Instantaneous losses

### 12.2.1 Anchorage set

The magnitude of the anchorage set shall be the greater of that required to control the stress in the prestressing steel at transfer or that recommended by the manufacturer of the anchorage. 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 \(13\)](#) can be used to obtain the anchorage set loss at anchorage, which is linearly reduced to 0 along the distance  $l_{\text{set}}$ , obtained by [Formula \(14\)](#), from the anchorage.

$$\Delta f_{\text{pA}} = \sqrt{\frac{4pE_p \Delta l}{A_{\text{ps}}}} \quad (13)$$

$$l_{\text{set}} = \sqrt{\frac{E_p A_{\text{ps}} \Delta l}{p}} \quad (14)$$

### 12.2.2 Friction

#### 12.2.2.1 Pretensioned construction

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

#### 12.2.2.2 Post-tensioned construction

Losses due to friction between the internal prestressing tendons and duct wall may be taken as per [Formula \(15\)](#):

$$\Delta f_{\text{pF}} = f_{\text{pj}} \left( 1 - e^{-(Kx + \mu\alpha)} \right) \quad (15)$$

Losses due to friction between the external tendon across a single deviator pipe may be taken as per [Formula \(16\)](#):

$$\Delta f_{\text{pF}} = f_{\text{pj}} \left( 1 - e^{-\mu(\alpha + 0,04)} \right) \quad (16)$$

Values of  $K$  and  $\mu$  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  $\mu$  as specified in [Table 20](#) may be used.

For tendons confined to a vertical plane,  $\alpha$  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  $\alpha$  shall be obtained by vectorially adding the total vertical angular change,  $\alpha_v$ , and the total horizontal angular change,  $\alpha_h$ .

**Table 20 — Friction coefficients for post-tensioning tendons**

Type of steel	Type of duct	$K$ /mm	$\mu$ /rad
Wire or strand	Rigid and semirigid galvanized metal sheathing	$[6,6 \times 10^{-7}]$	[0,15~0,25] (Recommended value: 0,20)
	Polyethylene	$[6,6 \times 10^{-7}]$	[0,23]
	Rigid steel pipe deviators for external tendons	$[6,6 \times 10^{-7}]$	[0,25]
High-strength bars	Galvanized metal sheathing	$[6,6 \times 10^{-7}]$	[0,30]

### 12.2.3 Elastic shortening

#### 12.2.3.1 Pretensioned members

The loss due to elastic shortening in pretensioned members shall be taken as per [Formula \(17\)](#):

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp} \quad (17)$$

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

#### 12.2.3.2 Post-tensioned members

The loss due to elastic shortening in post-tensioned members, other than slab systems, may be taken as per [Formula \(18\)](#):

$$\Delta f_{pES} = \frac{N-1}{2N} \frac{E_p}{E_{ci}} f_{cgp} \quad (18)$$

$f_{cgp}$  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.

#### 12.2.3.3 Combined pretensioning and post-tensioning

In applying the provisions of [12.2.3.1](#) and [12.2.3.2](#) 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.

### 12.3 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,  $\Delta f_{pLT}$ , due to creep of concrete, shrinkage of concrete, and relaxation of steel shall be estimated using [Formulae \(19\)](#) to [\(21\)](#):

$$\Delta f_{pLT} = 10 \frac{f_{pi} A_{ps}}{A_g} \gamma_h \gamma_{st} + 83 \gamma_h \gamma_{st} + \Delta f_{pR} \quad (19)$$

where:

$$\gamma_h = 1,7 - 0,01 H_r \quad (20)$$

$$\gamma_{st} = \frac{35}{(7 + f_{ci})} \quad (21)$$

### 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 \(22\)](#):

$$\Delta f_{pLT} = \frac{\varepsilon_{cs} E_p + 0,8 \Delta f_{pR} + \frac{E_p}{E_c} \varphi(t, t_0) f_{c,QP}}{1 + \frac{E_p}{E_c} \cdot \frac{A_{ps}}{A_c} \left(1 + \frac{A_c}{I_c} z_{cp}^2\right) [1 + 0,8 \varphi(t, t_0)]} \quad (22)$$

where

- $A_c$  is the area of the concrete section;
- $A_{ps}$  is the area of all the prestressing tendons at the location  $x$ ;
- $E_c$  is the modulus of elasticity for the concrete;
- $E_p$  is the modulus of elasticity for the prestressing steel;
- $f_{c,QP}$  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;
- $\Delta f_{pLT}$  is the absolute value of the variation of stress in the tendons due to creep, shrinkage and relaxation at location  $x$ , at time  $t$ ;

$\Delta f_{pR}$  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;

$\varepsilon_{cs}$  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 \(22\)](#) should be used with a positive sign. [Formula \(22\)](#) 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.  $\varepsilon_{cs}$  after the end of initial wet curing, e.g. after 7 days for moist cured concrete and after 1 to 3 days for steam cured concrete, can be obtained by [Formulae \(23\)](#) and [\(24\)](#), respectively.

$$\varepsilon_{cs} = \frac{t}{35+t} \varepsilon_{csu} \quad (23)$$

$$\varepsilon_{cs} = \frac{t}{55+t} \varepsilon_{csu} \quad (24)$$

where

$t$  is the time in days;

$\varepsilon_{csu} = [780] \times 10^{-6}$  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 to 3 days for steam cured concrete can be obtained by [Formula \(25\)](#).

$$\varphi(t, t_0) = \frac{t^{0,6}}{10+t^{0,6}} \varphi_u \quad (25)$$

where

$t$  is the time in days after prestressing;

$\varphi_u = [2,35]$ .

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

$\Delta f_{pR}$  can be obtained by [Formulae \(26\)](#) and [\(27\)](#) for post-tensioned members and pretensioned members, respectively.

$$\Delta f_{pR} = f_{pi} \frac{\log_{10} t}{10} \left( \frac{f_{pi}}{f_{py}} - 0,55 \right) \quad (26)$$

$$\Delta f_{pR} = f_{pi} \frac{\log_{10} t - \log_{10} t_r}{10} \left( \frac{f_{pi}}{f_{py}} - 0,55 \right) \quad (27)$$

where

$t$  is the time in hours;

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

## 13 Details of tendon

### 13.1 Tendon confinement

#### 13.1.1 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 segments, nominal confinement reinforcing shall be provided around the duct at each segment face. The reinforcement shall not be less than two rows of 13 mm diameter hairpin bars at both sides of each duct with vertical dimension equal to the slab thickness, less than top and bottom cover dimensions.

The effects of grouting pressure in the ducts shall be considered.

#### 13.1.2 Effects of curved tendons

##### 13.1.2.1 General

Reinforcement shall be used to confine curved tendons. The reinforcement shall be proportioned to ensure that the steel stress at serviceability limit state does not exceed  $0,6f_y$ , and the assumed value of  $f_y$  shall not exceed 420 MPa. Spacing of the confinement reinforcement shall not exceed either 3,0 times the outside diameter of the duct or 600 mm.

Where tendons are located in curved webs or flanges or are curved around and close to re-entrant corners or internal voids, additional concrete cover and/or confinement reinforcement shall be provided. The distance between a re-entrant corner or void and the near edge of the duct shall not be less than 1,5 times duct diameters. When a tendon curves in two planes, the in-plane and out-of-plane forces shall be added together vectorially.

##### 13.1.2.2 In-plane force effects

In-plane deviation force effects due to the change in direction of tendons shall be taken as per [Formula \(28\)](#):

$$F_{u-in} = \frac{P_t}{R} \quad (28)$$

The maximum deviation force shall be determined on the basis that all the tendons, including provisional tendons, are stressed.

The shear resistance per unit length of the concrete cover against pull-out by deviation forces,  $V_r$ , shall be taken as per [Formula \(29\)](#):

$$V_r = \phi V_{ni} \quad (29)$$

where

$$V_{ni} = 0,33d_{ce}\sqrt{f_{ci}'};$$

$$\phi = [0,75].$$

If the factored in-plane deviation force exceeds the factored shear resistance of the concrete cover, as specified in [Formula \(29\)](#), fully anchored tie-backs to resist the in-plane deviation force shall be provided in the form of either non-prestressed or prestressed reinforcement.

Where stacked ducts are used in curved girders, the moment resistance of the concrete cover, acting in flexure, shall be investigated.

For curved girders, the global flexural effects of out-of-plane forces shall be investigated.

Where curved ducts for tendons other than those crossing at approximately 90° are located so that the direction of the radial force from one tendon is toward another, confinement of the ducts shall be provided by:

- a) spacing the ducts to ensure adequate nominal shear resistance, as specified in [Formula \(29\)](#);
- b) providing confinement reinforcement to resist the radial force; or
- c) specifying that each inner duct be grouted before the adjacent outer duct is stressed.

### 13.1.2.3 Out-of-plane force effects

Out-of-plane force effects due to the wedging action of strands against the duct wall may be estimated as per [Formula \(31\)](#):

$$F_{u-out} = \frac{P_t}{\pi R} \quad (31)$$

If the factored shear resistance given by [Formula \(29\)](#) is not adequate, local confining reinforcement shall be provided throughout the curved tendon segments to resist all of the out-of-plane forces, preferably in the form of spiral reinforcement.

## 13.2 External tendon supports

The unsupported length of external tendons shall not exceed 7 500 mm.

## 13.3 Post-tensioned anchorage zones

### 13.3.1 General

For anchorage zones at the end of a component or segment, the transverse dimensions may be taken as the depth and width of the section but not larger than the longitudinal dimension of the component or segment. The longitudinal extent of the anchorage zone in the direction of the tendon shall not be less than the greater of the transverse dimensions of the anchorage zone and shall not be taken as more than one and one-half times that dimension.

For intermediate anchorages, the anchorage zone shall be considered to extend in the direction opposite to the anchorage force for a distance not less than the larger of the transverse dimensions of the anchorage zone.

### 13.3.2 General zone and local zone

#### 13.3.2.1 General zone

The extent of the general zone shall be taken as identical to that of the overall anchorage zone including the local zone, defined in [13.3.1](#).

Design of general zones shall comply with the requirements of [13.3.3](#).

#### 13.3.2.2 Local zone

Design of local zones shall comply with the requirements of [13.3.4](#).

For design of the local zone, the effects of high bearing pressure and the application of confining reinforcement shall be considered.

Anchorage devices based on the acceptance test shall be referred to as special anchorage devices.

### 13.3.3 Design of general zone

#### 13.3.3.1 Limitations of application

Concrete compressive stresses ahead of the anchorage device, location and magnitude of the bursting force, and edge tension forces may be estimated using [Formula \(35\)](#) through [Formula \(39\)](#), provided that:

- the member has a rectangular cross-section and its longitudinal extent is not less than the larger transverse dimension of the cross-section;
- the member has no discontinuities within or ahead of the anchorage zone;
- the minimum edge distance of the anchorage in the main plane of the member is not less than 1,5 times the corresponding lateral dimension,  $a$ , of the anchorage device;
- only one anchorage device or one group of closely spaced anchorage devices is located in the anchorage zone; and
- the angle of inclination of the tendon, as specified in [Formulae \(36\)](#) and [\(37\)](#), is between  $-5,0^\circ$  and  $+20,0^\circ$ .

#### 13.3.3.2 Compressive stresses

The concrete compressive stress ahead of the anchorage devices,  $f_{ca}$ , calculated using [Formulae \(32\)](#) to [\(34\)](#), shall not exceed  $0,7\phi f_{ci}$ :

$$f_{ca} = \frac{0,6P_t \kappa}{A_b \left( 1 + l_c \left( \frac{1}{b_{eff}} - \frac{1}{h} \right) \right)} \quad (32)$$

where

if  $s_a < 2a_{eff}$  then:

$$\kappa = 1 + \left( 2 - \frac{s_a}{a_{eff}} \right) \left( 0,3 + \frac{n}{15} \right) \quad (33)$$

if  $s_a \geq 2a_{eff}$  then:

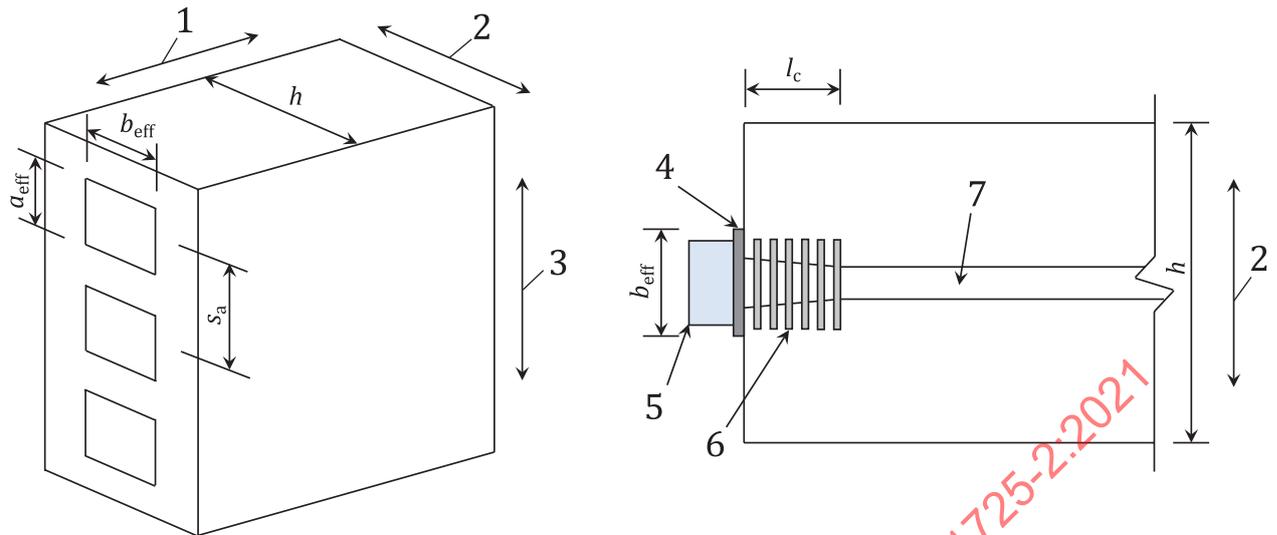
$$\kappa = 1 \quad (34)$$

where  $l_c$  should not be more than the larger of  $1,15a_{eff}$  or  $1,15b_{eff}$  (mm). Refer to [Figure 21](#).

The effective bearing area,  $A_b$ , in [Formula \(32\)](#) shall be taken as the larger of the anchor bearing plate area,  $A_{plate}$ , or the bearing area of the confined concrete in the local zone,  $A_{conf}$ , with the following limitations:

- if  $A_{plate}$  controls,  $A_{plate}$  shall not be taken larger than  $(4/\pi)A_{conf}$ ;
- if  $A_{conf}$  controls, the maximum dimension of  $A_{conf}$  shall not be more than twice the maximum dimension of  $A_{plate}$  or three times the minimum dimension of  $A_{plate}$ . If any of these limits is violated, the effective bearing area,  $A_b$ , shall be based on  $A_{plate}$ ; and
- deductions shall be made for the area of the duct in the determination of  $A_b$ .

If a group of anchorages is closely spaced in two directions, the product of the correction factors,  $\kappa$ , for each direction shall be used, as specified in [Formula \(32\)](#).


**Key**

- 1 longitudinal direction
- 2 transverse direction
- 3 vertical direction
- 4 anchor plate (bearing plate)
- 5 anchor head (wedge plate)
- 6 confining reinforcement (bursting reinforcement)
- 7 duct (sheath)

**Figure 21 — Anchorage of a PSC girder**
**13.3.3.3 Bursting forces**

The bursting forces in anchorage zones,  $T_{burst}$ , may be taken as per [Formula \(35\)](#):

$$T_{burst} = 0,25 \sum P_t \left( 1 - \frac{a_t}{h} \right) + 0,5 \left| \sum (P_t \sin \alpha_t) \right| \quad (35)$$

The location of the bursting force,  $d_{burst}$ , may be taken as per [Formula \(36\)](#):

$$d_{burst} = 0,5(h - 2e_a) + 5e_a \sin \alpha_t \quad (36)$$

If a special anchorage shape is used, the reinforcements provided for bursting forces can be arranged differently from usual practice by approval of a chief engineer.

**13.3.3.4 Edge tension forces**

The longitudinal edge tension force may be determined from an analysis of a section located at one half the depth of the section away from the loaded surface taken as a beam subjected to combined flexure and axial load. The spalling force may be taken as equal to the longitudinal edge tension force but not less than 2 % of the total factored tendon force.

**13.3.4 Design of local zone**

**13.3.4.1 Dimensions of local zone**

Where either:

- the manufacturer has not provided edge distance recommendations; or
- edge distances have been recommended by the manufacturer, but they have not been independently verified;

The transverse dimensions of the local zone in each direction shall be taken as the greater of:

- the corresponding bearing plate size, plus twice the minimum concrete cover required for the particular application and environment; and
- the outer dimension of any required confining reinforcement, plus the required concrete cover over the confining reinforcing steel for the particular application and environment.

Where the manufacturer has recommendations for minimum cover, spacing and edge distances for a particular anchorage device, and where these dimensions have been independently verified, the transverse dimensions of the local zone in each direction shall be taken as the lesser of:

- twice the edge distance specified by the anchorage device supplier; and
- the centre-to-centre spacing of anchorages specified by the anchorage device supplier.

Recommendations for spacing and edge distance of anchorages provided by the manufacturer shall be taken as minimum values.

The length of the local zone along the tendon axis shall not be taken to be less than:

- the maximum width of the local zone;
- the length of the anchorage device confining reinforcement; or
- for anchorage devices with multiple bearing surfaces, the distance from the loaded concrete surface to the bottom of each bearing surface, plus the maximum dimension of that bearing surface.

The length of the local zone shall not be taken as greater than 1,5 times the width of the local zone.

**13.3.4.2 Bearing resistance**

Normal anchorage devices shall comply with the requirements specified herein.

When general zone reinforcement satisfying [13.3.3](#) is provided, and the extent of the concrete along the tendon axis ahead of the anchorage device is at least twice the length of the local zone as defined in [13.3.4.1](#), the factored bearing resistance of anchorages shall be taken as per [Formulae \(37\)](#) to [\(39\)](#):

$$P_r = \phi f_n A_b \tag{37}$$

where

$f_n$  is the lesser of:

$$f_n = 0,7 f_{ci} ' \sqrt{\frac{A}{A_g}} \tag{38}$$

$$f_n = 2,25 f_{ci} ' \tag{39}$$

$$\phi = [0,70].$$

The full bearing plate area may be used for  $A_g$  and the calculation of  $A_b$  if the plate material does not yield at the factored tendon force and the slenderness of the bearing plate,  $n_b/t_b$ , shall satisfy [Formula \(40\)](#):

$$n_b / t_b \leq 0,08 \left( \frac{E_b}{f_b} \right)^{0,33} \quad (40)$$

For anchorages with separate wedge plates,  $n_b$  may be taken as the largest distance from the outer edge of the wedge plate to the outer edge of the bearing plate. For rectangular bearing plates, this distance shall be measured parallel to the edges of the bearing plate. If the anchorage has no separate wedge plate,  $n_b$  may be taken as the projection beyond the outer perimeter of the group of holes in the direction under consideration.

For bearing plates that do not meet the slenderness requirement specified herein, the effective gross bearing area,  $A_g$ , shall be taken as:

- for anchorages with separate wedge plates: the area geometrically similar to the wedge plate, with dimensions increased by twice the bearing plate thickness; and
- for anchorages without separate wedge plates: the area geometrically similar to the outer perimeter of the wedge holes, with dimension increased by twice the bearing plate thickness.

## 14 Superstructure

### 14.1 Strength of members subjected to flexural moments

#### 14.1.1 General

The superstructure system employed by a bridge designed according to this document should be one of the systems covered or their permitted variations. The selection of an appropriate system should be performed by studying several alternatives.

In flexural design, the section shall be designed as under reinforced section to achieve the ductile failure. Calculation of the design strength of member sections subjected to flexural moments should be performed employing the requirements of [14.1](#). If the factored axial compressive load on the member,  $P_u$ , exceeds  $0,1f_cA_g$ , the calculation of the design strength should be performed by employing the analysis of combined axial load and flexural moment.

#### 14.1.2 Factored flexural moment at section

The factored flexural moment at section,  $M_u$ , caused by the factored loads applied to the structure should be determined, for the particular element type, from the following requirements.

#### 14.1.3 Minimum design flexural moment strength

The design flexural moment strength of the section,  $\phi M_n$ , should be greater than or equal to the factored flexural moment at that section,  $M_u$ , as shown in [Formula \(41\)](#).

$$\phi M_n \geq M_u \quad (41)$$

where  $\phi = [1,0]$ .

#### 14.1.4 Nominal moment strength of PSC box girder

The stress of prestressing steel at flexural failure,  $f_{ps}$ , may be assumed to be  $f_{py}$  for simplification. Box girder section can be idealized as an I-girder section to accommodate simplified calculations. [Figure 22](#)