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Guidelines for the simplified design of reinforced concrete bridges

Lignes directrices pour la conception simplifiée des ponts en béton armé

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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. 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. 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.

The committee responsible for this document is ISO/TC 71, *Concrete, reinforced concrete and pre-stressed concrete*, Subcommittee SC 5, *Simplified design standard for concrete structures*.

Introduction

The aim of this International Standard is to provide rules for the design and construction of relatively short span concrete bridges. This International Standard 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 International Standard shall 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 models. This International Standard 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 International Standard are intended to account for undesirable side effects that will 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 upon 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 International Standard contains guidelines that can be modified by the national standards body due to local design and construction requirements and practices. These guidelines that can be modified are included using ["boxed values"]. The authorities in each member country are expected to review the "boxed values" and may substitute alternative definitive values for these elements for use in the national application of the document.

A great effort was made to include self-explanatory tables, graphics, and design aids to simplify the use of this International Standard and provide foolproof procedures. Notwithstanding, the economic implications of the conservatism inherent in approximate procedures as a substitute for sound and experienced engineering should be a matter of concern to the designer that employs the document, and to the owner that hires him.

Guidelines for the simplified design of reinforced concrete bridges

1 Scope

This International Standard can be used as an alternative to the development of a National Concrete Bridge Design and Construction Code, or equivalent document in countries where no national design codes are available by themselves, or as an alternative to the National Concrete Bridge Design and Construction Code in countries where specifically considered and accepted by the national standards body or other appropriate regulatory organization, and applies to the planning, design and construction of structural concrete bridges to be used in new bridges of restricted span length, height of piers, and type.

The purpose of these guidelines is to provide sufficient information to perform the design of the structural concrete bridge that complies with the limitations established in 6.1. The rules of design as set forth in this International Standard are simplifications of more elaborate requirements.

Although the guidelines contained in this International Standard were drawn to produce, when properly employed, a structural concrete structure with an appropriate margin of safety, these guidelines are not a replacement for sound and experienced engineering. In order for the resulting structure designed employing these guidelines to attain the intended margin of safety, this International Standard must be used as a whole and alternative procedures should be employed only when explicitly permitted by the guidelines. The minimum dimensioning guides as prescribed in this International Standard replace, in most cases, more elaborate procedures such as those prescribed in the National Code, and the possible economic impact is compensated for by the simplicity of the procedures prescribed here.

The professional performing the structural design under these guidelines should meet the legal requirements for structural designers in the country of adoption and have training and a minimum appropriate knowledge of structural mechanics, statics, strength of materials, structural analysis, and reinforced concrete design and construction.

Designs and details for new bridges should 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;

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

2 Normative references

The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

ISO 679, *Cement — Test methods — Determination of strength*

ISO 863, *Cement — Test methods — Pozzolanicity test for pozzolanic cements*

ISO 3010, *Basis for design of structures — Seismic actions on structures*

ISO 4354, *Wind actions on structures*

ISO 6274, *Concrete — Sieve analysis of aggregates*

ISO 6782, *Aggregates for concrete — Determination of bulk density*

ISO 6783, *Coarse aggregates for concrete — Determination of particle density and water absorption — Hydrostatic balance method*

ISO 6934-1, *Steel for the prestressing of concrete — Part 1: General requirements*

ISO 6934-3, *Steel for the prestressing of concrete — Part 3: Quenched and tempered wire*

ISO 6934-4, *Steel for the prestressing of concrete — Part 4: Strand*

ISO 6934-5, *Steel for the prestressing of concrete — Part 5: Hot-rolled steel bars with or without subsequent processing*

ISO 6935-1, *Steel for the reinforcement of concrete — Part 1: Plain bars*

ISO 6935-2, *Steel for the reinforcement of concrete — Part 2: Ribbed bars*

ISO 6935-3, *Steel for the reinforcement of concrete — Part 3: Welded fabric*

ISO 7033, *Fine and coarse aggregates for concrete — Determination of the particle mass-per-volume and water absorption — Pycnometer method*

ISO 9194, *Bases for design of structures — Actions due to the self-weight of structures, non-structural elements and stored materials — Density*

ISO 9597, *Cement — Test methods — Determination of setting time and soundness*

ISO 10144, *Certification scheme for steel bars and wires for the reinforcement of concrete structures*

ISO 3766:2003, *Construction drawings — Simplified representation of concrete reinforcement*

3 Terms and definitions

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

3.1
abutment
end support of a bridge superstructure

NOTE Abutments are used to transmit the reaction of superstructure to the foundations, to retain the earth filling and to connect the superstructure to the approach roads.

3.2
acceleration of gravity, g
acceleration produced by gravity at the surface of the earth

NOTE For the application of these guidelines its value can be approximated to $g \approx$ [10] m/s^2.

3.3
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

3.4
aggregate
granular material, such as sand, gravel, crushed stone, and iron blast-furnace slag, used in conjunction with a cementing medium to form a hydraulic cement concrete or mortar

3.5
anchorage
device used to anchor a non-structural element to the structural framing

3.6
backfill
material used for refilling any hole that has been excavated

3.7
bar diameter, nominal
approximate diameter of a steel reinforcing bar, often used as a class designation

NOTE For deformed bars, it is common practice to use the diameter of a plain bar having the same area.

3.8
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

3.9
bearing capacity of the soil
maximum permissible stress on the foundation soil that provides adequate safety against bearing failure of the soil, or settlement of the foundation of such magnitude as to impair the structure

NOTE Its value is defined at the working stress level.

3.10
bearing – elastomeric
device constructed partially or wholly from elastomer to transmit loads and accommodate movements between a bridge and its supporting structure

3.11
bending moment
product of a force and the distance to a particular axis, producing bending effects in a structural element

3.12
boundary elements
portions along wall edges strengthened by longitudinal and transverse reinforcement

NOTE Boundary elements do not necessarily require an increase in thickness of the wall.

3.13
bridge
structure carrying a road, path or railway over an obstacle

3.14
caisson
foundation pile of large diameter, built partly or totally above ground and sunk below ground usually by digging out the soil inside

3.15
cantilever
element that extends beyond its support and is supported on one end only

3.16
cement
material as specified in the corresponding referenced ISO standards, which, when mixed with water, has hardening properties

NOTE Used either in concrete or by itself.

3.17
clearance
distance by which one thing clears another; the space between them

3.18
column
vertical member used primarily to support axial compressive loads

3.19
collector elements
elements that serve to transmit the inertia forces within the diaphragm to members of the lateral-force resisting system

3.20
combined footing
footing that transmits to the supporting soil the load carried by several columns or structural concrete walls

3.21
compression reinforcement
reinforcement provided to resist compression stresses induced by flexural moments acting on the member section

3.22
concrete
mixture of portland cement and any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixtures

3.23
concrete mix design
choice and proportioning of the ingredients of concrete

3.24
concrete specified compressive strength, $f_c^{'}$
compressive cylinder strength of concrete used in design and evaluated in accordance with the appropriate ISO standard, expressed in megapascals (MPa)

NOTE 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).

3.25
confinement hook
hook on a stirrup, hoop, or crosstie having a bend not less than 135° with a six-diameter (but not less than 75 mm) extension that engages the longitudinal reinforcement and projects into the interior of the stirrup or hoop

3.26
confinement stirrup or tie
closed stirrup, tie or continuously wound spiral

NOTE A closed stirrup or tie can be made up of several reinforcement elements each having confinement hooks at both ends. A continuously wound spiral should have a confinement hook at both ends.

3.27
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

3.28
cover, concrete
thickness of concrete between surface of any reinforcing bar and the nearest face of the concrete member

3.29
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 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.

3.30
crown
highest point of a convex structure, such as an arch or a vault

3.31
curb
edge where a raised pavement/sidewalk/footpath, road median, or road shoulder meets an unraised street or other roadway

3.32
curing
keeping the concrete damp for a period of time, usually several days, starting from the moment it is cast, in order for the cement to be provided with enough water to harden and attain the intended strength

NOTE Appropriate curing will greatly reduce shrinkage, increase strength of concrete, and should reduce surface cracking. Curing time will depend on temperature and relative humidity of surrounding air, the amount of wind, the direct sunlight exposure, the type of concrete mix employed, and other factors.

3.33
deformed reinforcement
steel reinforcement that has deformations in its surface to increase its bond to the concrete.

NOTE The following steel reinforcement should be considered deformed reinforcement under these guidelines: deformed reinforcing bars, deformed wire, welded plain wire fabric, and welded deformed wire fabric conforming to the appropriate ISO standards.

3.34
depth of member, h
vertical size of a cross section of a horizontal structural element

3.35
design load combinations
combinations of factored loads and forces as specified in these guidelines

3.36
design strength
product of the nominal strength multiplied by a strength reduction factor

3.37
development length
length of embedded reinforcement required to develop the design strength of reinforcement at a critical section

3.38
development length for a bar with a standard hook
shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent to the outer edge of the 90° or 180° hook

3.39
differential settlement
when the foundation of different parts of a structure settle different amounts

3.40
drainage
natural or artificial removal of surface and sub-surface water from a given area

NOTE Drainage is a system that carries water or sewage away from a place.

3.41
durability
characteristic of a structure to resist gradual degradation of its serviceability in a given environment for the design service life

3.42
effective depth of section, d
distance measured from extreme compression fiber to centroid of tension reinforcement

3.43
embedment length
length of embedded reinforcement provided beyond a critical section

3.44
factored loads and forces
specified nominal loads and forces multiplied by the load factors prescribed in these guidelines

3.45
fire protection of reinforcement
amount of concrete cover necessary for protection of the reinforcement against the effects of the high temperatures produced by fire

NOTE The concrete cover is a function of the number of hours of exposure to the fire.

3.46
flange
top or bottom part of an I or T shaped section separated by the web

3.47
flexural
pertaining to the flexure bending moment

3.48
flexural reinforcement
reinforcement provided to resist the tensile stresses induced by flexural moments acting on the member section

3.49
footing
that portion of the foundation which transmits loads directly to the soil

NOTE May be the widening part of a column, a structural concrete wall or several columns, in a combined footing.

3.50
formwork
temporary construction to contain concrete in a plastic state while it is cast and setting, and that forms the final shape of the element as the concrete hardens

3.51
foundation
any part of the structure that serves to transmit loads to the underlying soil, or to contain it

3.52
foundation beam
beam that rests on the foundation soil and spans between footings, used either to support walls or to limit differential settlement of the foundation

3.53
foundation mat
continuous slab laid over the ground as part of the foundation and that transmits to the underlying soil the loads from the structure

3.54
girder
main horizontal support beam, usually supporting other beams

3.55
gravity loads
loads that act downward and are caused by the acceleration of gravity, g, acting on the mass of the elements that cause the dead and live loads

3.56
hydrostatic pressure
pressure at a specific elevation exerted by a body of water at rest or, in the case of groundwater, the pressure at a specific elevation due to the weight of water at higher levels in the same zone of saturation

NOTE Hydrostatic pressure increases linearly with depth.

3.57
hydrodynamic pressure
pressure at a specific elevation exerted by a body of water at rest or, in the case of groundwater, during an earthquake and that results from the dynamic response of the water itself

3.58
hook
bend at the end of a reinforcing bar

NOTE They are defined by the angle that the bend forms with the bar as either 90°, 180° or 135° hooks.

3.59
Interface Friction Angle
angle of friction at the interface (or the skin friction) between cohesionless soil and another material

3.60
Internal Friction Angle
angle measured between the normal force and resultant force that is attained when failure just occurs in response to a shearing stress

NOTE 1 It is a measure of the ability of a unit of rock or soil to withstand a shear stress.

NOTE 2 Its value is determined experimentally.

3.61
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

3.62
lap splice
splice between two reinforcing bars obtained by overlapping them for a specified length

3.63
lateral-force resisting system
portion of the structure composed of members proportioned to resist forces related to earthquake effects

3.64
lightweight aggregate concrete
concrete made with coarse granular material that weighs less than the granular material used in normal weight concrete

NOTE This type of concrete is not covered by these guidelines.

3.65
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)

3.66
live loads
static and dynamic effects, in terms of forces applied on the structure, produced by the use of the bridge by pedestrians and/or vehicles and and not including construction or environmental loads

3.67
load effects
forces and deformations produced in structural members by the applied loads

3.68
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

3.69
loads
forces or other actions that result from the weight of all bridge materials, pedestrians, vehicles, environmental effects, differential movement, and restrained dimensional changes

3.70
longitudinal reinforcement
reinforcement that is laid parallel to the longitudinal axis of the element, generally to account for flexural effects

3.71
mass
quantity of matter in a body

3.72
mesh wire
welded-wire fabric reinforcement

3.73
modulus of elasticity
ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material

3.74
negative moment
flexural moment that produces tension stresses at the upper part of the section of a horizontal or nearly horizontal element, and that requires placing negative flexural reinforcement in the upper part of the element section

3.75
negative reinforcement
in horizontal or nearly horizontal elements, the flexural reinforcement required for negative moment and that is placed in the upper part of the section of the element

3.76
nominal loads
magnitudes of the loads specified in these guidelines (dead, live, soil, wind, snow, rain, flood, and earthquake)

3.77
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 formulas set forth by these guidelines

NOTE 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.

3.78
pedestal
upright compression member with a ratio of unsupported height to average least lateral dimension of less than 3

3.79
permanent loads
loads in which variations over time are rare or of small magnitude

NOTE All other loads are variable loads (see also nominal loads).

3.80
pile
slender timber, concrete or structural steel element embedded in the ground to support loads

3.81
plain reinforcement
smooth surfaced steel reinforcement, or reinforcement that does not conform to the definition of deformed reinforcement

3.82
positive moment
flexural moment that produces tension stresses at the lower part of the section of a horizontal or nearly horizontal element, and that requires placing positive flexural reinforcement in the lower part of the element section

3.83
positive reinforcement
in horizontal or nearly horizontal elements, the flexural reinforcement required for positive moment and that is placed in the lower part of the section of the element

3.84
railing
structure made of rails and upright members that is used as a guard or barrier or for support

3.85
reaction
resistance to a force or load, or upward resistance of a support such as a structural concrete wall or column against the downward pressure of a loaded member such as a beam

3.86
reinforcement
steel bars, wire, or mesh wire, used for reinforcing the concrete where tensile stresses are expected, due either to the applied loads, or to environmental effects such as variation of temperature

3.87
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 these guidelines

3.88
retaining wall
wall built to hold back earth

3.89
selfweight
weight of the structural element, caused by the material that composes the element

3.90
service load
load specified by these guidelines (without load factors)

3.91
settlement
downward movement of the supporting soil

3.92
shear
internal force acting tangential to the plane where it acts

NOTE Also called diagonal tension.

3.93
shear reinforcement
reinforcement designed to resist shear

3.94
shores
vertical or inclined support members designed to carry the weight of the formwork, concrete, and construction loads above

3.95
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

3.96
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

3.97
slab
upper flat part of a reinforced concrete deck carried by supporting joists or beams or girders

3.98
solid slab
slab of uniform thickness that does not have voids to make it lighter or that has voids with size less than [0,6] times the slab thickness

3.99
span length
horizontal distance between supports of a horizontal structural element

NOTE Such as a slab, joist, beam, or girder.

3.100
specifications
written document describing in detail the scope of work, materials to be used, method of installation, and quality of workmanship

3.101
specified lateral earthquake forces
lateral forces corresponding to the appropriate distribution of the design base shear force prescribed by these guidelines, for earthquake-resistant design

3.102
specified wind forces
nominal pressure of wind to be used in design performed in accordance with these guidelines

3.103
spiral reinforcement
continuously wound reinforcement in the form of a cylindrical helix

3.104
spread footing
isolated footing that transmits to the supporting soil the load carried by a single column

3.105
stirrup
reinforcement used to resist shear and torsion stresses in a structural member

NOTE 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 tie.

3.106
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 Including the probability of understrength members due to variations in material strengths and dimensions, approximations in the design equations, 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.

3.107
stress
intensity of force per unit area

3.108
structural concrete
all concrete used for structural purposes including plain, reinforced and prestressed concrete

3.109
structural concrete walls
walls proportioned to resist combinations of shear, moments, and axial forces

NOTE A "shearwall" is a "structural wall".

3.110
structural diaphragms
structural members, such as deck slabs, which transmit inertia induced by earthquake motions

3.111
support
structural element that provides support to another structural element

3.112
tie
loop of reinforcing bar or wire enclosing longitudinal reinforcement

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

3.113
tie elements
elements which serve to transmit inertia forces and prevent spacing of bridge components such as footings and walls

3.114
transverse reinforcement
reinforcement located perpendicular to the longitudinal axis of the element, comprising stirrups, ties, spiral reinforcement, among others

3.115
wall
member, usually vertical and whose length and height are much larger than its thickness

3.116
water table
groundwater level
point under surface of the ground where the ground is completely saturated with water

3.117
web
thin vertical portion of an I or T shaped section that connects the flanges

3.118
weight
vertical downward force exerted by a mass when subjected to the acceleration of gravity

NOTE The weight is equal to the value of the mass multiplied by the acceleration of gravity, g.

3.119
wire
reinforcing bar of small diameter

3.120
working stress
allowable stress to be used with unfactored loads

3.121
yield strength, f_y
specified minimum yield strength or yield point of reinforcement

NOTE 1 The yield strength is expressed in units of megapascals (MPa).

NOTE 2 Applicable International Standards specify that the yield strength or yield point be determined in tension.

4 Symbols and abbreviated terms

The following symbols and abbreviated terms are used in these guidelines.

Symbol	Explanation	Unit
a	depth of equivalent uniform compressive stress block	mm
A_b	area of an individual reinforcement bar or wire	mm^2
A_c	loaded area of bearing on concrete or the area of the confined column core, in a column with spiral reinforcement, measured center to center of the spiral	mm^2
A_g	gross area of section of element	mm^2
A_j	effective cross-sectional area within a joint for shear evaluation or area of additional hanger reinforcement, where beams are supported by girders or other beams	mm^2
A_s	area of longitudinal tension reinforcement	mm^2
A'_s	area of longitudinal compression reinforcement	mm^2
$A_{s,min}$	minimum area of longitudinal tension reinforcement	mm^2
A_{se}	total extreme steel area in a column or structural concrete wall for computation of the balanced moment strength	mm^2
A_{ss}	total side steel area in a column or structural concrete wall for computation of the balanced moment strength	mm^2
A_{st}	total area of longitudinal reinforcement	mm^2
A_{su}	wind exposed surface area	m^2
A_v	area of shear reinforcement within a distance s	mm^2
α	coefficient of thermal expansion for concrete	
αA	inclination of the critical failure surface for minimum active earth pressure conditions	
αP	inclination of the critical failure surface for maximum passive earth pressure conditions	
αA	inclination of the critical failure surface for minimum total active earth pressure conditions	
αP	inclination of the critical failure surface for maximum total passive earth pressure conditions	
ah	amplitude of the harmonic base acceleration	
b	width of compression face of member, or width of the section of the member	mm
b_c	width of the column section, or largest plan dimension of capital or drop panel, for punching shear evaluation	mm
b_{col}	dimension of column section in the direction perpendicular to the girder span	m
b_f	effective width of the compression flange in a T shaped section	mm
b_w	web width in a T-shaped section, or web width of girders, beams or joists, or thickness of the web in a structural concrete wall	mm

Symbol	Explanation	Unit
b_0	perimeter of critical section for punching shear in slabs	mm
d	effective depth, should be taken as the distance from extreme compression fiber to centroid of tension reinforcement	mm
d'	distance from extreme compression fiber to centroid of compression reinforcement	mm
d_b	nominal diameter of reinforcing bar or wire	mm
d_c	distance from extreme tension fiber to centroid of tension reinforcement or diameter of the confined core of column with spiral reinforcement	mm
d_P	diameter of pile, at footing base	mm
D	dead loads, or related internal moments and forces	
δ_T	longitude change due to thermal expansion	
Δ_T	temperature variation	
δ	interface friction angle	
Δ PAE	dynamic component of the total active thrust on retaining walls	
Δ PPE	dynamic component of the total passive thrust on retaining walls	
Δ Peq	dynamic thrust on non yielding walls	
Δ Meq	dynamic overturning moment on non yielding walls	
E	load effects of earthquake, or related internal moments and forces	
E_c	modulus of elasticity of concrete	MPa
ϕ	internal friction angle	
f'_c	specified compressive strength of concrete	MPa
$\sqrt{f'_c}$	postivie square root of specified compressive strength of concrete. The result should have units of	MPa
f_{cd}	compressive strength of concrete reduced by the material factor	MPa
f_{cu}	extreme fiber factored compressive stress at edges of structural walls	MPa
f_y	specified yield strength of reinforcement	MPa
f_{yd}	yield strength of reinforcement reduced by the material factor	MPa
f_{ypr}	probable specified maximum strength of reinforcement	MPa (f_{ypr} = 1,25 · f_y).
f_{ys}	specified yield strength of transverse or spiral reinforcement	MPa
f_{ysd}	yield strength of transverse or spiral reinforcement reinforcement reduced by the material factor	MPa
F	loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces	
F_i, F_x	design wind or seismic force applied at level i or x, respectively	kN
F_{iu}, F_{xu}	factored design lateral force applied to the wall at level i or x, respectively	N
Fp	dynamic thrust factors for non yielding walls	
Fm	dynamic overturning moment factors for non yielding walls	
h	depth or thickness of structural element or overall thickness of member	mm
h_b	vertical distance measured from the bottom of the supporting girder to the bottom of the supported beam	mm
h_{col}	dimension of column section in the direction parallel to the girder span	m
h_c	height of the column section	mm

Symbol	Explanation	Unit
h_f	slab thickness	mm
h_i, h_x	height above the base to level i or x, respectively	m
h_n	clear vertical distance between lateral supports of columns and walls	mm
h_{pi}	story height of deck I, measured from deck finish of the story to deck finish of the story immediately below	mm
h_s	total height of the supporting girder	mm
h_w	height of entire structural concrete wall from base to top	mm
H	loads due to the weight and pressure of soil, water in soil, or other materials, or related internal moments and forces	
H_P	height of the support which forces are being evaluated	
I_c	moment of inertia of the column section	m^4
I_P	about an axis on the bridge deck perpendicular to the bridge length, of the pier	m^4
K_A	active earth pressure coefficient	
K_P	passive earth pressure coefficient	
K_{AE}	total active earth pressure coefficient	
K_{PE}	total passive earth pressure coefficient	
kv	vertical pseudostsatic force on yielding walls	
kh	horizontal pseudostsatic force on yielding walls	
l	span of structural element or length of span measured center-to-center of beams or other supports	
l_a	length of clear span in the short direction of two-way slabs, measured face-to-face of beams or other supports	m
l_b	length of clear span in the long direction of two-way slabs, measured face-to-face of beams or other supports	m
l_d	development length for reinforcing bar	mm
l_m	length of clear span in the direction that moments, shears and reinforcement are being determined, measured face-to-face of supports	m
l_j	clear spacing between joists	m
l_n	length of clear span in the long direction of two-way construction, measured face-to-face of supports in slabs without beams, and face-to-face of beams or other supports in other cases or length of clear span, measured face-to-face of supports in slabs without beams, and face-to-face of beams or other supports in other cases	mm
l_w	horizontal length of structural concrete wall	mm
l_o	column confinement length	
L	live loads, or related internal moments and forces	
L_r	sloping roof live load, or related internal moments and forces	
max.	maximum	
min.	minimum	
M_{bn}	nominal flexural moment strength at section at balanced conditions	N · mm
M_{br}	flexural moment strength at section at balanced conditions	N ··mm
M_{iu}, M_{xu}	factored story moment caused by lateral loads at story i or x, respectively	N
M_n	nominal flexural moment strength at section	N · mm
M_r	flexural moment strength at section	N · mm

Symbol	Explanation	Unit
M_{pr}	probable flexural moment strength of the element at the joint face computed using f_{ypr} and = 1	N · m
M_u	factored flexural moment at section	N · m
M_u^-	factored negative flexural moment at section	N · m
M_u^+	factored positive flexural moment at section	N · m
ΣM_c	sum of lowest flexural strengths ($\phi \cdot M_n$) of columns framing into a joint	N · m
ΣM_g	sum of flexural strengths ($\phi \cdot M_n$) of girders framing into a joint	N · m
ΔM_u	factored unbalanced moment at a column-girder joint or factored unbalanced moment at a wall-girder joint	N · m
P_d	non factored dead load axial force at section or non factored concentrated dead load applied directly to the element	N
P_{bn}	nominal compression axial load strength at section at balanced conditions	N
P_{br}	nominal compression axial load strength at section at balanced conditions	N
P_{cu}	factored compression load on wall boundary element, including earthquake effects	
P_l	non factored live load axial force at section or non factored concentrated live load applied directly to the element	N
P_n	nominal axial load strength at section	N
$P_{n(max)}$	maximum compression nominal axial load strength at section	N
P_{tn}	axial tension strength at section	N
P_{tu}	factored tension force on wall boundary element, including earthquake effects	
P_u	factored axial load at section or factored concentrated design load applied directly to the element or factored axial load on column or wall	N
P_{0n}	axial compressive strength at section	N
ΣP_u	sum of all factored concentrated design loads within the span	N
P_A	active thrust on a retaining wall	
P_P	passive thrust on a retaining wall	
P_{AE}	total Active thrust on a retaining wall including seismic effects	
P_{PE}	total Passive thrust on a retaining wall including seismic effects	
P_w	hydrodynamic thrust on retaining walls	
q_d	non-factored dead load per unit area	N/m^2
q_l	non-factored live load per unit area	N/m^2
q_u	factored load per unit area	N/m^2
r_u	factored uniformly distributed reaction from the slab on the supporting girder, beam or structural concrete wall	N/m
r_w	pore pressure ratio	
R_a	rain load, or related internal moments and forces	
R_u	total factored concentrated reaction from a supported structural element	N
ΣR_u	sum of all factored reactions from supported structural elements at the same story	N
s	center-to-center spacing of transverse reinforcement measured along the axis of the element or spacing between stirrups or vertical spacing between bars of skin reinforcement or spacing of longitudinal or transverse reinforcement or clear distance between webs	mm
S	snow load, or related internal moments and forces	

Symbol	Explanation	Unit
Ss	5 % damped spectral response accelerations	
T	cumulative effect of temperature, creep, shrinkage, or differential settlement, or related internal moments and forces	
T_u	factored torsional moment at section	N · mm
U	required factored strength to resist factored loads or related internal moments and forces	
V_c	contribution of the concrete to the nominal shear strength at section	N
ΔV_e	factored design shear force from the development of the probable flexural capacity of the element at the faces of the joints	N
V_{iu}, V_{xu}	factored story shear caused by lateral loads at story i or x, respectively	N
V_n	nominal shear strength at section	N
V_s	contribution of the horizontal reinforcement to the nominal shear strength at section	N
V_u	factored shear force at section	N
VT	shear force caused by thermal expansion	N
V	soil Poisson's Ratio	
w_d	non-factored uniformly distributed dead load per unit element length applied directly to the element	N/m
w_l	non-factored uniformly distributed live load per unit element length applied directly to the element	N/m
w_u	factored uniformly distributed design load per unit element length applied directly to the element	N/m
W	wind loads, or related internal moments and forces	
W_u	total factored uniformly distributed design load per unit element length	kN/m
α_a	fraction of the load that travels in the short direction in two-way slabs-on-girders	
α_b	fraction of the load that travels in the long direction in two-way slabs-on-girders	
α_s	constant used to compute nominal punching shear strength in slabs	
β	ratio of clear spans in long to short direction of two-way slabs	
ϕ	strength reduction factor	
ρ	ratio of longitudinal tension reinforcement.	$\frac{A_s}{b \cdot d}$
ρ'	ratio of longitudinal compression reinforcement	
ρ_h	ratio of horizontal reinforcement in structural concrete walls	
ρ_{max}	maximum permissible ratio of longitudinal flexural tension reinforcement	
ρ_{min}	minimum permissible ratio of longitudinal flexural tension reinforcement	
ρ_s	ratio of spiral reinforcement	
ρ_t	ratio of total longitudinal reinforcement area to gross concrete section area,.	$\frac{A_{st}}{b \cdot d}$
ρ_v	ratio of vertical reinforcement in structural concrete walls.	
f_{cd}	compressive strength of concrete reduced by the material factor	MPa
f_{yd}	yield strength of reinforcement reduced by the material factor	MPa
γ_{mc}	material factor for concrete	
γ_{ms}	material factor for steel	

5 Design and construction procedure

5.1 Procedure

The design procedure comprises the following steps. See Figure 1.

5.1.1 Step A

Definition of the layout in plan and height of the structure, following the guides of chapter 7. Verification that the limitations of 6.1 are met.

5.1.2 Step B

Calculation of all gravity loads that act on the structure using the guides of chapter 8, excluding the self weight of the structural elements.

5.1.3 Step C

Definition of an appropriate superstructure system, depending on the span lengths and the magnitude of the gravity loads, according to the guides of chapter 10.

5.1.4 Step D

Trial dimensions for the slab of the superstructure system. Calculation of the self weight of the system, and design of the elements than comprise it, correcting the dimension as required by the ultimate and serviceability limit states, complying with the guides of 10.4 for slab systems with beams.

5.1.5 Step E

Trial dimensions for the beams and girders and calculation of their self weight. Flexural and shear design of the beams and girders, correcting the dimension as required by the ultimate and serviceability limit states, complying with the guides include in 10.5.

5.1.6 Step F

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, complying with the guides of 11.5.

5.1.7 Step G

If lateral loads such as earthquake, wind, or lateral earth pressure exist, their magnitude is established using the guides in Chapter 8; otherwise the designer should proceed to Step I.

5.1.8 Step H

The loads at the foundation level are determined, and a definition of the foundation system is performed employing the guides of chapter 12. The structural elements of the foundation are designed.

5.1.9 Step I

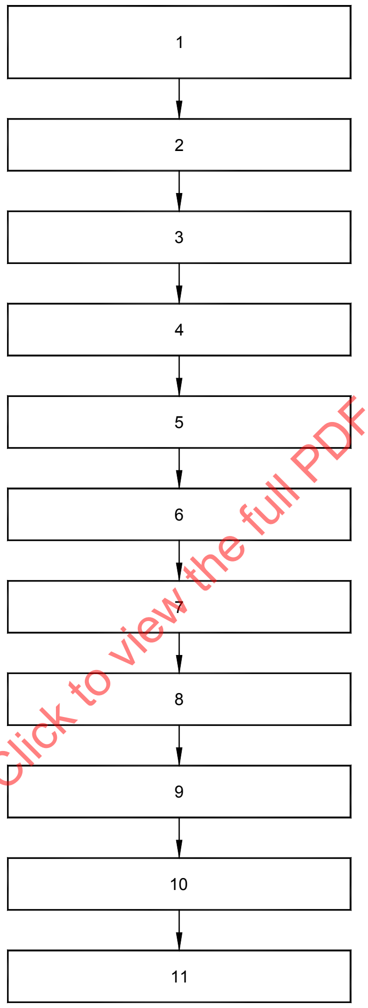
Preliminary location and trial dimensions for structural concrete walls capable of resisting the lateral loads established, using the guides of 13 for earthquake forces, the influence of their self weight is evaluated, and flexure and shear design of the structural concrete walls is performed, complying with the guides of 11.6.

5.1.10 Step J

Production of structural drawings.

5.1.11 Step K

Construction of the structure should be performed complying with the local construction and practice.



Key

1 Step A: preliminary structure design
2 Step B: loads definition
3 Step C: superstructure system definition
4 Step D: slab design
5 Step E: beams and girders design
6 Step F: substructure design
7 Step G: lateral forces definition
8 Step H: foundations design
9 Step I: structural walls design
10 Step J: structural drawings
11 Step K: structure construction

Figure 1 — Design and construction procedure

5.2 Design documentation

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

5.2.1 Calculation memoir

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

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

b) A description of the structural system employed.

c) Loads employed.

d) Grade, strength and fabrication standards for all structural materials.

e) Presentation of all design computations.

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

5.2.2 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, and all other information required in 12.

5.2.3 Structural drawings

All the drawings required for construction of the bridge.

5.2.4 Specifications

The construction specifications required.

6 General Guides

6.1 Limitations

These guidelines should be employed only when the bridge being designed complies with all the limitations set forth in 6.1.1 to 6.1.12.

6.1.1 Permitted use

Bridges of mixed use should be permitted to be designed using these guidelines, but restricted to pedestrian and vehicular traffic. Bridges for trains are out of the scope of these guidelines.

6.1.2 Maximum number of spans

The maximum number of spans for a bridge designed using these guidelines should be as per Table 1.

Table 1 — Maximum allowed number of spans

	Number of spans
Bridge over water	[1]
Bridge for road overpass	[3]

6.1.3 Maximum span length

The maximum span length allowed is [30] m for each span.

6.1.4 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. %
[1]	—
[2]	20
[3]	10

6.1.5 Maximum cantilever length

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

Continuous deck slab cantilevers over intermediate beam supports, may have lengths up to 80 % of the length of the adjacent span.

6.1.6 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. At any rate, 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]

6.1.7 Maximum number of lanes

The maximum number of lanes for a vehicular bridge designed using these guidelines should be [2]. Up to [2] sidewalks may be considered in addition to the vehicle traffic lanes.

6.1.8 Width limitations

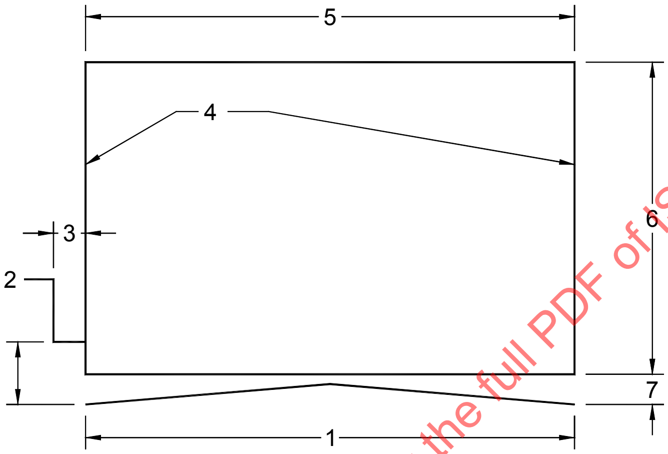
Pedestrian bridges should not have widths of less than [1.2] m.

Vehicular bridges should not have roadways with widths, excluding sidewalks, of less than [3] m or in excess of [8] m. Sidewalk should comply with a minimum width of [1] m, and neither should exceed half of the maximum lane width.

6.1.9 Clearances

The horizontal clearance shall be the clear width, and vertical clearance the clear height for the passage of vehicular traffic as shown in Figure 2.

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.



Key

1 roadway width

2 curb

3 brush or sidewalk if warranted (max. 2,75 m)

4 face of curb or barrier

5 horizontal clearance

6 vertical clearance

7 crown

Figure 2 — Clearance diagram for bridges

6.1.9.1 General Clearances

6.1.9.1.1 Vertical Clearance

Vertical clearance shall not be less than [5.50] m over the entire roadway width with an allowance of [0.3] m for resurfacing.

6.1.9.1.2 Horizontal clearance

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

6.1.9.2 Clearances for Underpasses

6.1.9.2.1 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 [0.70] m from the face of pier or abutment.

The face of the guardrail or other device shall be at least [0.70] m outside the normal shoulder line.

6.1.9.2.2 Vertical Clearance

A vertical clearance of not less than [5.50] m shall be provided between curbs, or if curbs are not used, over the entire width that is available for traffic.

6.1.9.3 Clearances for Depressed Roadways

6.1.9.3.1 Clearance between walls

The minimum width between walls for depressed roadways carrying two lanes of traffic shall be [9.00] m.

6.1.9.3.2 Curbs

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

6.1.10 Maximum skew angle

Bridges designed using these guidelines should have a low skew angle, not exceeding [15 °] for girder and slab bridges, and [30 °] for box girder bridges.

6.1.11 Maximum bridge horizontal curvature

Bridges designed using these guidelines should have a maximum length to horizontal curvature radius of [4 %].

6.1.12 Cross section variation

Bridges designed using these guidelines should have a constant cross section along the continuous portions of the bridge, except in cantilever sections.

6.1.13 Interaction between superstructure and substructure

No framing effect is permitted along the longitudinal axis of the bridge. No direct transmition 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 be able to move freely in the deck longitudinal direction.

6.2 Limit states

The design approach of the present guidelines 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 guidelines.

6.2.1 Deflection Serviceability Verification

6.2.1.1 Vehicular Bridges

The deflection of the floor structure must be less than L/700

6.2.1.2 Pedestrian bridges

A simplified design criterion for the resonance response is given by

$$\frac{a_p}{g} = \frac{P_0 \, exp(-0.35\, f_n)}{\beta W} \leq \frac{a_0}{g}$$

Where:

$\frac{a_p}{g}$: estimated peak acceleration (in units of g)

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

f_n: natural frequency of floor structure

f_0: constant force equal to 0.29 kN for floors and 0.41 kN for footbridges

W: effective weight of the floor

β: modal damping ratio The natural frequency of the floor structure must be greater than 3 Hz.

The natural period is given by

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

The natural frequency is the inverse of the natural period,

$$f_n = \frac{1}{T} = \frac{1}{2\pi}\sqrt{\frac{k}{m}}$$

Taking into account that $k = F / \Delta$ and $m = F / g$, then:

$$f_n = \frac{1}{2\pi}\sqrt{\frac{Fg}{F\Delta}} \cong 0.16\sqrt{\frac{9.81}{\Delta}}$$

Δ may be calculated as per annex B

Table 4 — Maximum allowable support height

1 Span	2 and 3 continuous spans
Δ	(0.40-0.50)Δ

6.3 Ultimate limit state design format

6.3.1 General

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

The basic requirement for ultimate limit state should be:

Resistances ≥ Load effects Equation (1)

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

$$R = f\left(\frac{f_{cd}}{\gamma_{mc}}, \frac{f_{yd}}{\gamma_{ms}}\right) \geq \gamma_1 S_1 + \gamma_2 S_2 + \gamma_3 S_3 + \cdots \qquad \text{Equation (2)}$$

R stands for strength and S stands for load effects based on the nominal loads prescribed by these guidelines. Therefore, the ultimate limit state design format requires that:

Design Strength ≥ Required Factored Strength Equation (3)

or

$$R = f\left(\frac{f_{cd}}{\gamma_{mc}}, \frac{f_{yd}}{\gamma_{ms}}\right) \geq U \qquad \text{Equation (4)}$$

Where the required factored strength is $U = \gamma_1 \cdot S_1 + \gamma_2 \cdot S_2 + \cdots$

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 8.9.1.

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 these guidelines, based on the limit stress reduced according to each corresponding material as per Table 5:

Table 5 — Material factor

MATERIAL	
Concrete, γ_{mc}	[1.5]
Steel, γ_{ms}	[1.15]

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. The compliance with the serviceability limit state under these guidelines, should be obtained indirectly thorough 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 or fatigue that is sufficient to cause significant reduction in the strength of the structure or in its service life.

7 Structural systems and layout

7.1 Description of the components of the structure

For the purposes of these guidelines, the bridge structure should be divided in the following components:

7.1.1 Superstructure system

The superstructure or deck system consists of the structural elements that comprise the portion of the bridge that receive directly the live load. In section 10, the different superstructure systems covered by these guidelines are described. The superstructure system includes the girders, beams, and joists (if employed), and the slab that spans between them, or the slab, when it is directly supported on piers, columns, or walls. The superstructure should also act as a diaphragm that carries through its plane the lateral loads from their point of application to the vertical elements of the lateral load resisting system.

7.1.2 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. Under these guidelines the main vertical elements of the substructure system should be cantilever piers, or frames or structural concrete walls, and are described in section 11.

7.1.3 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. Foundation systems are described in section 12. Deep foundations are beyond the scope of these guidelines, and are not covered by it.

7.2 General program

7.2.1 Architectural program

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 program 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 program 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.

Environmental and visual impact.

7.2.2 General structural guides for the project

Based in the general architectural program information, the structural designer should define the general structural guides for the structure being designed under these guidelines. 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.

Availability of qualified workmanship.

7.3 Structural layout

7.3.1 General structural layout

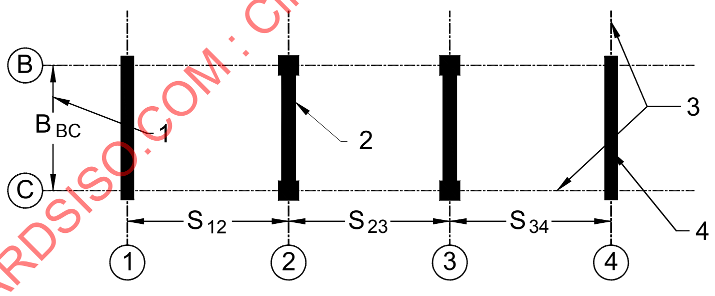
The structural designer should define a general structural layout in plan. See Figure 3. The general structural layout in plan should include:

Dimensioned grid for axes, or centerlines, 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.

Horizontal distance between centerlines, S, which corresponds to the center-to-center span lengths, and horizontal distance B, which corresponds to the center-to-center breadth, of the superstructure system.



Key

1 deck width

2 pier, frame or wall

3 centerlines grid

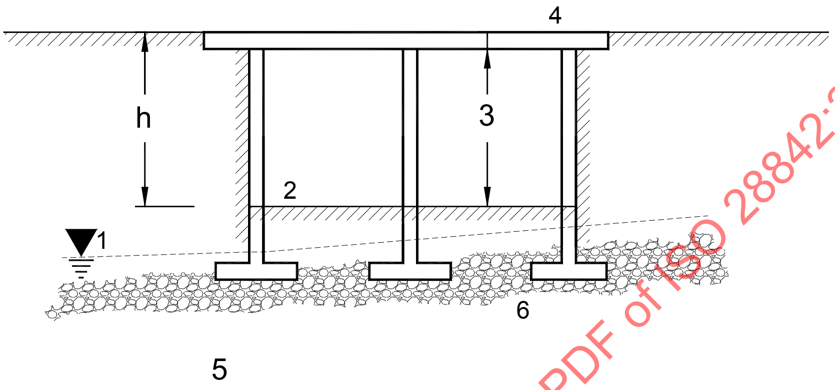
4 abutment

Figure 3 — General structural layout in plan

7.3.2 Vertical layout

The structural designer should define a general structural vertical layout. See Figure 4. 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 these guidelines or required by local highway specifications, whichever is larger.
- 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 4 — 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 under these guidelines. The compliance with the following limitations should be verified:

The use of the bridge should be within the accepted uses of 6.1.1.

The number of spans should not exceed the maximum permissible, given in 6.1.2.

The span lengths should be within maximum lengths prescribed in 6.1.3.

The difference between adjacent spans should not exceed the limit of 6.1.4.

Cantilever lengths should be within maximum lengths prescribed in 6.1.5.

The height of the tallest support, measured from ground to superstructure finish, should not exceed the maximum permissible height given in 6.1.6, nor the difference between supports heights should exceed the limits given there.

The number of lanes should not exceed the maximum permissible, given in 6.1.7.

Pedestrian bridge decks and vehicular roadways should comply with width limitations given in 6.1.8.

Bridge clearances must be specified according to 6.1.9.

Bridge skew angle for girders and deck should not exceed the limit given in 6.1.10.

Bridge length to horizontal curvature ratio should not exceed limit given in 6.1.11.

Cross section variation along bridge length must comply with 6.1.12.

8 Actions (Loads)

8.1 General

This clause provides minimum load guides for the design of bridges under these guidelines. Loads and the appropriate load combinations should be used together.

Loads and forces explicitly considered in bridge design according to these guidelines are:

Dead loads

Live loads (Static and dynamic effects)

Longitudinal forces

Earth pressure

Wind loads

Earthquake inertial forces

Loads and forces implicitly considered are:

Thermal forces

Shrinkage forces

Skew stress effects

Elastomeric bearings shear resistance

Settlement of the ground

8.2 Dead loads

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.

8.2.1 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 [2.5] Mg/m³. The use lower values for normal concrete density must 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.

8.2.2 Non structural elements

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

Table 6 — Density values for materials used in bridge construction

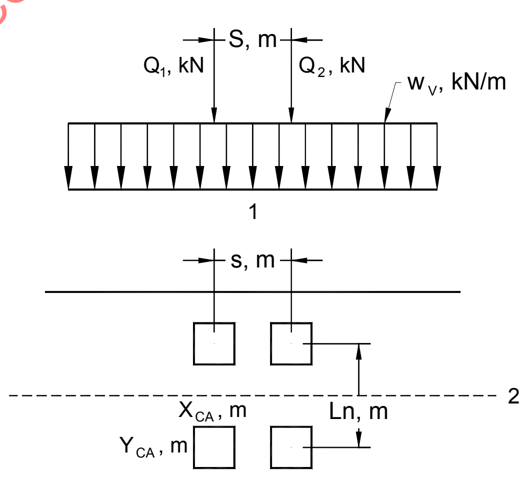
Material	Density, Mg/m^3
Steel	[7.9]
Timber	[0.8]
Reinforced concrete	[2.5]
Compacted filling soil	[1.9]
Loose filling soil	[1.6]
Stone masonry	[2.7]
Concrete masonry	[2.3]
Clay masonry	[1.4]
Asphalt	[1.8]

8.3 Live loads

Bridge live loads comprise the weights of all loads that might be applied to the superstructure according to the bridge use.

8.3.1 Vehicular bridges

Live load for vehicular bridges should be taken as a distributed load, w_V, and concentrated loads Q_1 and Q_2, distributed as in Figure 5.



Key

1 Vertical Load pattern

2 Plan Load distribution

Figure 5 — Vehicular live loads for a two-lane bridge

Vehicular live loads should be applied on a standard lane [3] m wide, regardless of the actual width of the bridge deck. Bridges with roadways widths exceeding [5.5] m should be considered two lane bridges and have dissimilar loads applied on each lane. Values for vehicular live loads are prescribed in Table 7. Data on overloads may be applied.

For a single span bridge, critical load occurs when Q1 is applied at the center of the span. For two and three span bridges, critical load may occur when Q1 and Q2 are not applied simultaneously on all spans. Positions of Q1 and Q2 shall be varied as to obtain critical condition.

Table 7 — Vehicular live loads values

Number of lanes	Lane	w_V (kN/m)	Q_1 (kN)	Q_2 (kN)	S (m)	X_{CA} (m)	Y_{CA} (m)	L_n (m)
1	1	[30]	[150]	[0]	[0]	[0.4]	[0.4]	[2]
2	1	[27]	[150]	[150]	[1.2]	[0.4]	[0.4]	[2]
	2	[8]	[100]	[100]	[1.2]	[0.4]	[0.4]	[2]

When light vehicular traffic is anticipated, the values given in Table 7 may be reduced as per Table 8. However, the application of these factors shall be accompanied with readily visible warning signs stating the restrictions for light trucks or for automobile only use. Appropriate physical barriers may be required as needed.

Table 8 — Vehicular live loads reduction factors

Restrictions	Light trucks	Only automobiles
Barriers to restrict heavier loads	[0.8]	[0.5]
No traffic physical restrictions	[0.8]	[0.8]

8.3.2 Pedestrian bridges

Pedestrian live loads of 5 kN/m² should be applied on the deck walkable area, as to cause the most unfavorable effects. Additionally, a truck load should be considered to account for maintenance equipment, unless the bridge width is less than 2 m or vehicle entrance is prevented by permanent barriers. The truck load, shown in Figure 6 and Table 9 should not be applied simultaneously with the distributed pedestrian load.

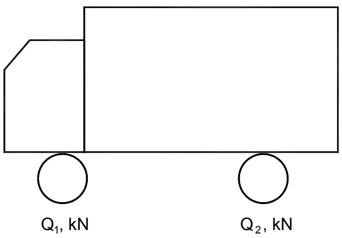


Figure 6 — Pedestrian bridge truck

Table 9 — Truck loads

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

8.3.3 Dynamic effect of live loads

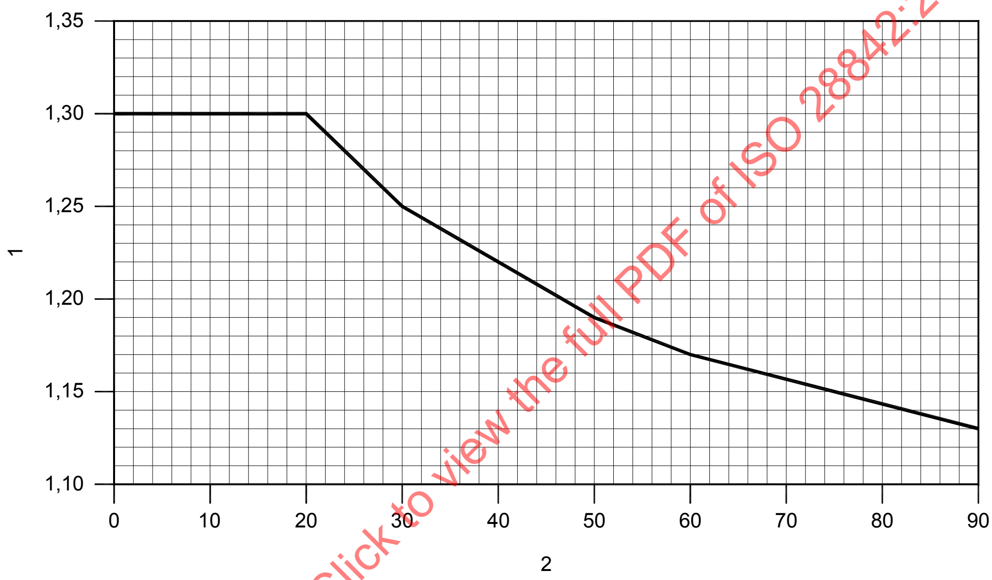
To account for the dynamic effects, such as impacts due to deck surface irregularities, vehicular live loads should be increased by the dimensionless factor given in Figure 7 according to the loaded area as to produce the most unfavorable effect on each element.

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 analysys (joists, slabs, etc.) increase the live load by 1.2.

Pedestrian live loads need not be increased.



Key

1 live load dynamic effect factor

2 loaded length, m

Figure 7 — 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.

Under no circumstance earth pressure should be taken as less than an equivalent fluid weight of [5] kN/m³.

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 10.

Table 10 — Wind loads for hurricane, cyclone or typhoon prone areas:

Load condition	Load direction	Load, kN/m^2
Load on structure*	Transverse	[2.5]
	Longitudinal	[0.6]
Load on live load*	Transverse	[1.5]
	Longitudinal	[0.4]

* 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 requirements of the National corresponding Standard should be met when calculating the mass of bridge building materials. When no National Standard is available, the requirements of ISO 9194 may be used. Table 6 of these guidelines may also be used to determine bridge masses.

For bridges which may be designed under these guidelines, 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 structuresite. 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 seconds, denoted as A_a, and usually expressed as a fraction of the acceleration of gravity, g (Acceleration of gravity may taken as 9.81 $^{m}/_{s^2}$).

For the purpose of the scope of these guidelines, the values for A_a must be taken from the National corresponding Standard having jurisdiction over the site of the considered existing structure. 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 seconds, 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 8.

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 percentage 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:

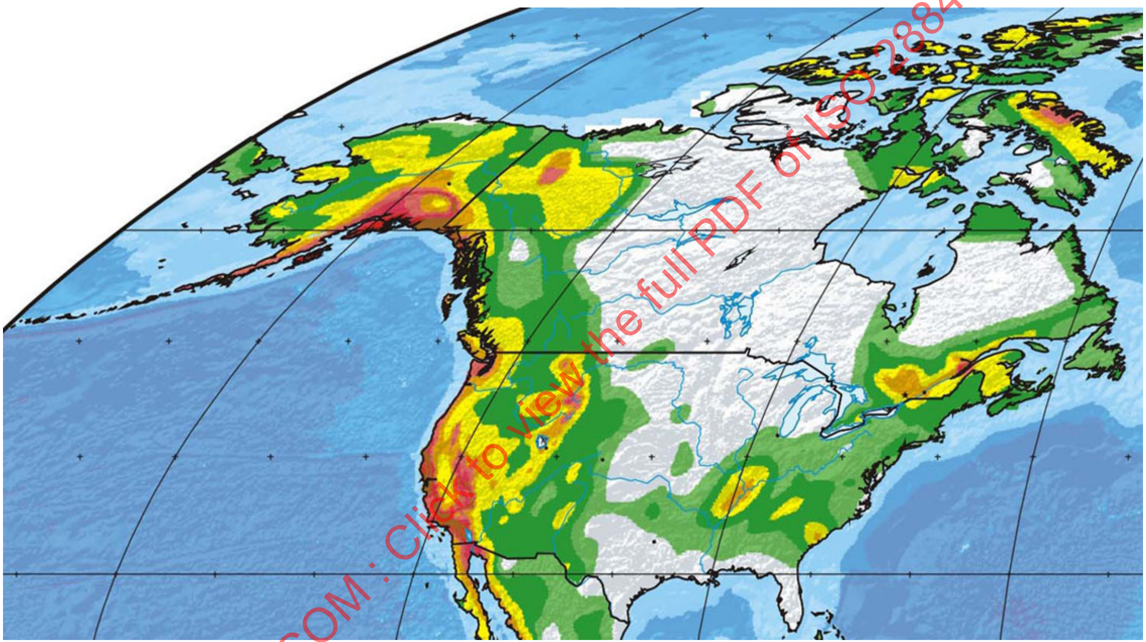
A zone where the value of A_a is estimated as more than [0.05] but less or equal to [0.10] may be deemed as a ***low seismic hazard*** zone.

8.7.5 Intermediate seismic hazard zones:

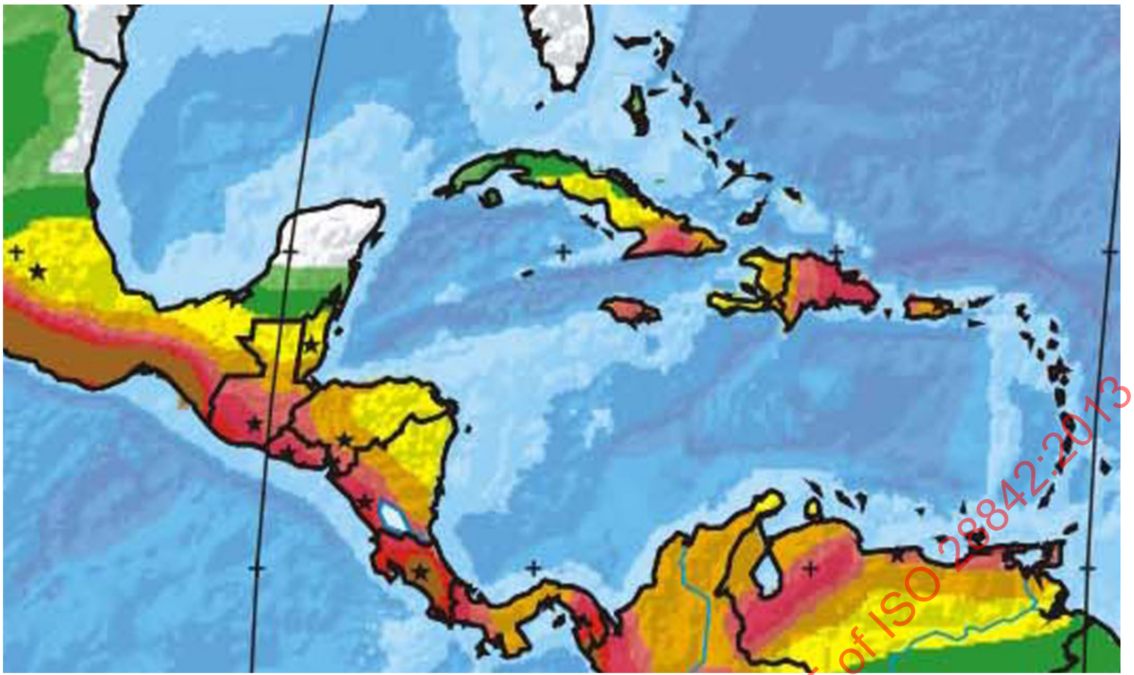
A zone where the value of A_a is estimated as more than [0.1] but less or equal to [0.20] may be deemed as a ***intermediate seismic hazard*** zone.

8.7.6 High seismic hazard zones:

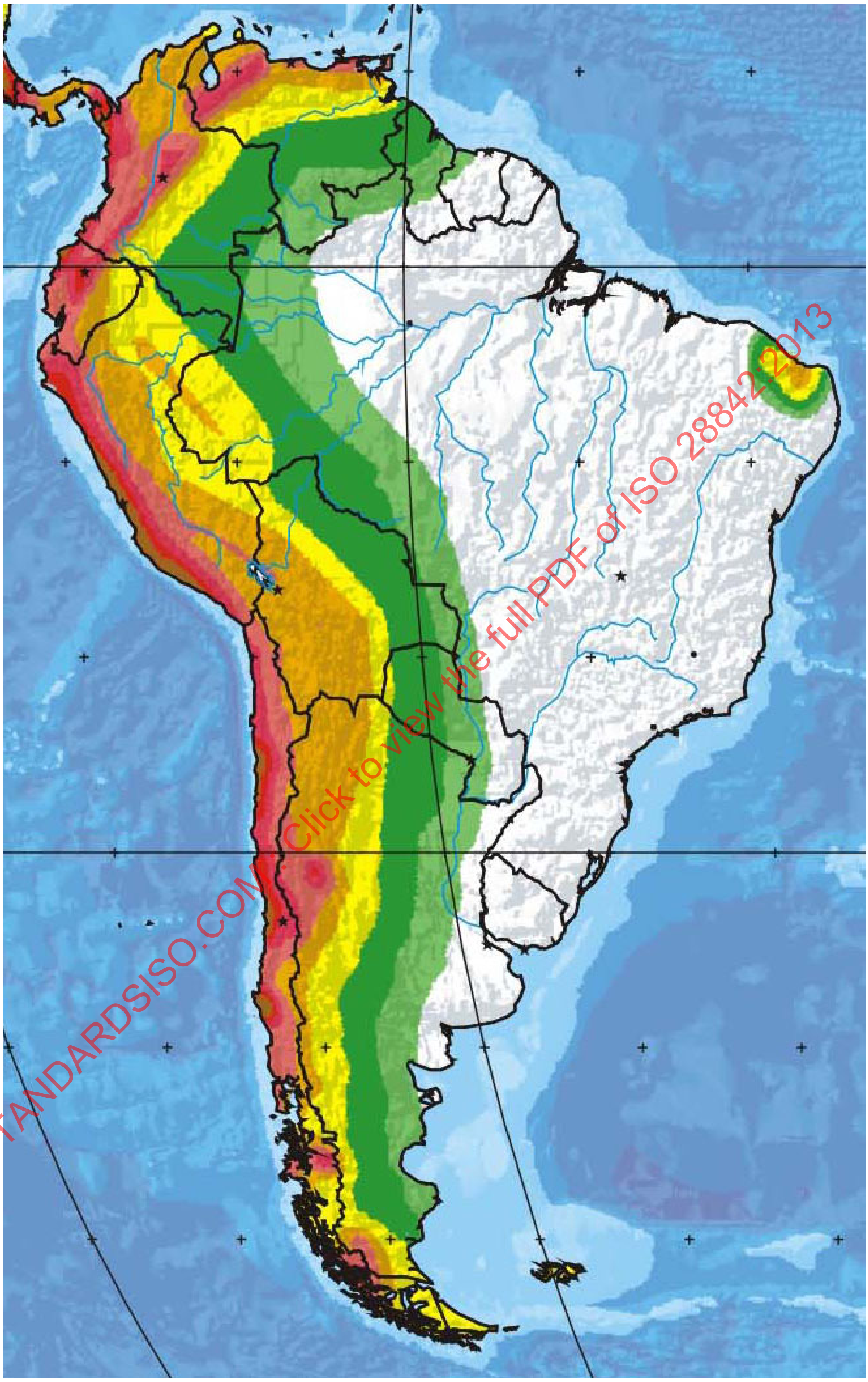
A zone where the estimated value of A_a exceeds [0.20] may be deemed as a ***high seismic hazard*** zone.



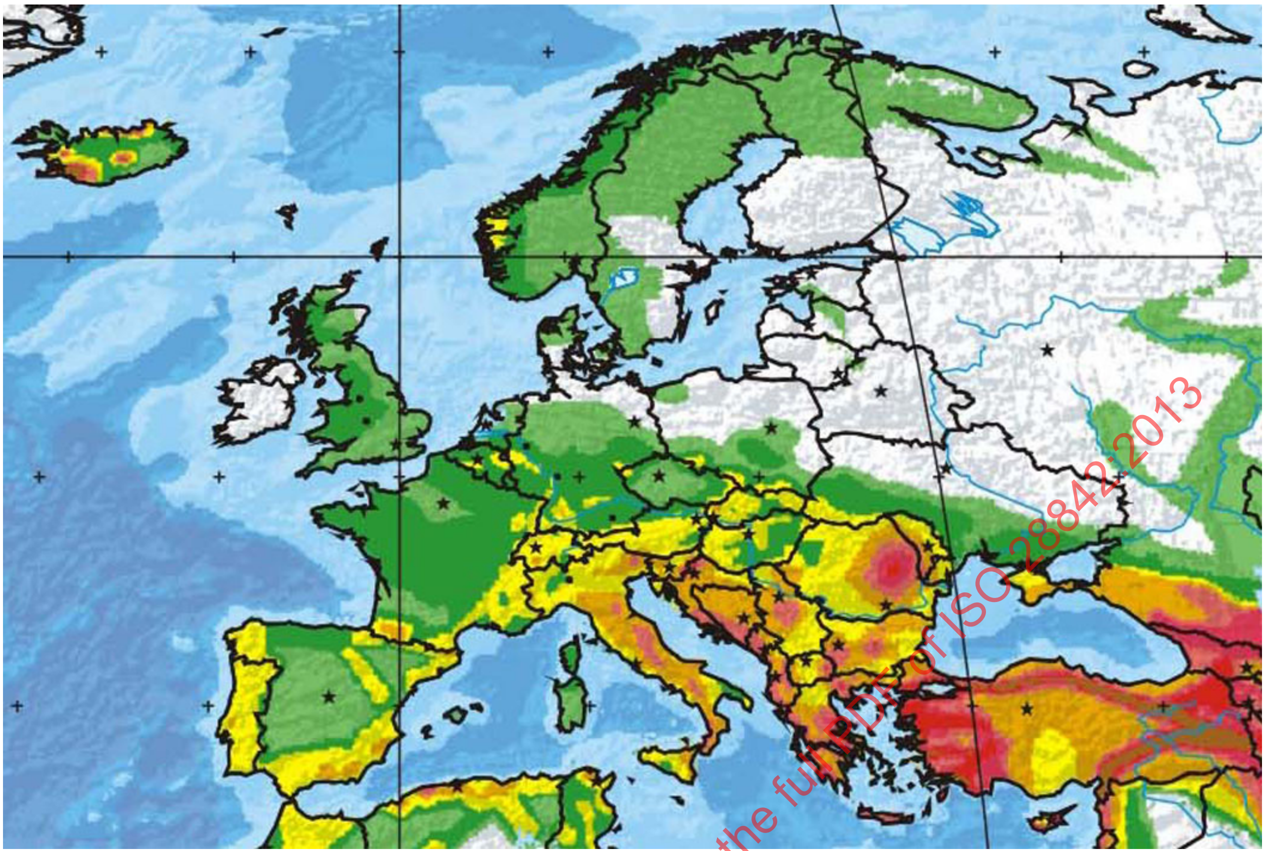
a) North America



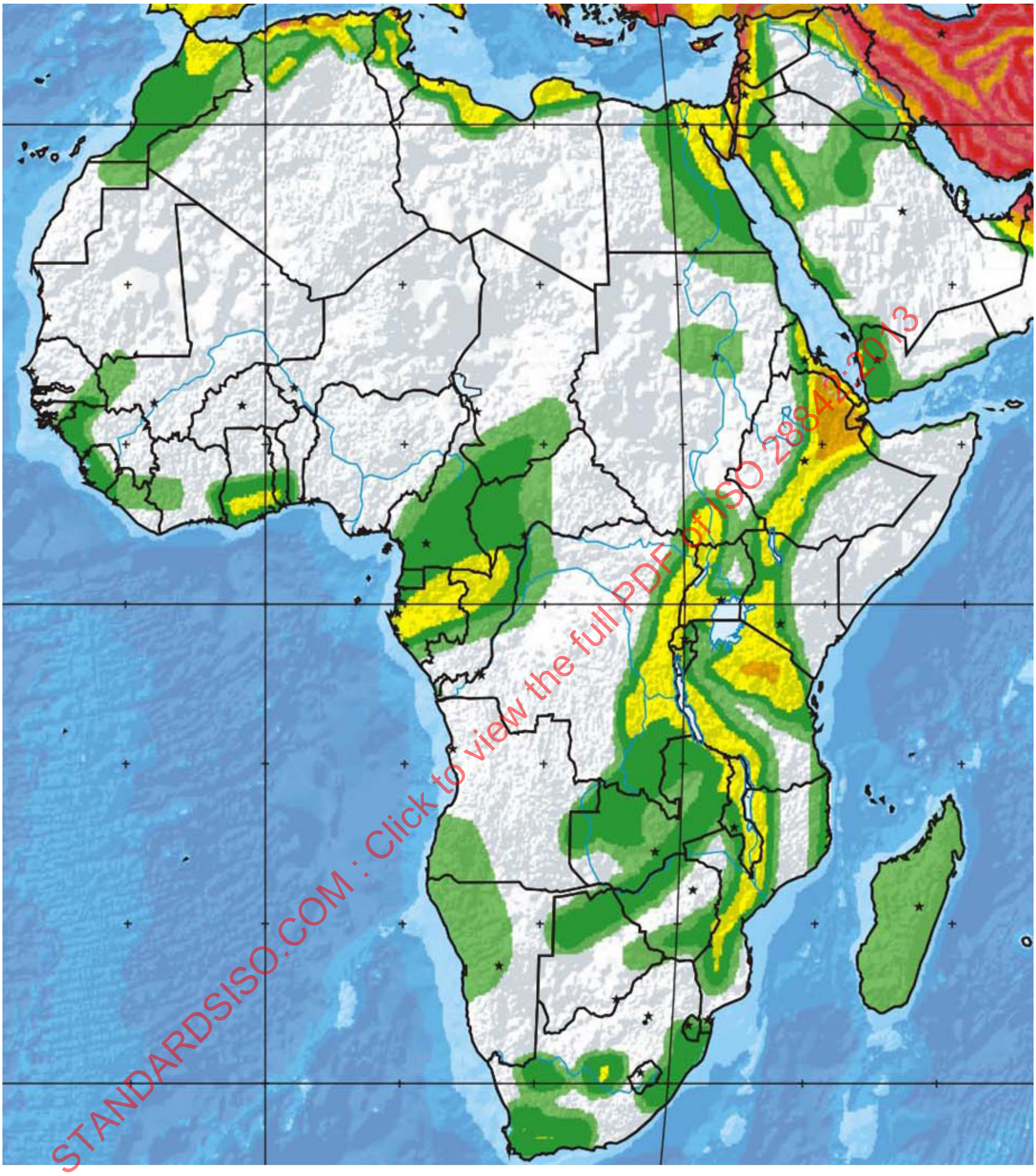
b) Central America and the Caribbean



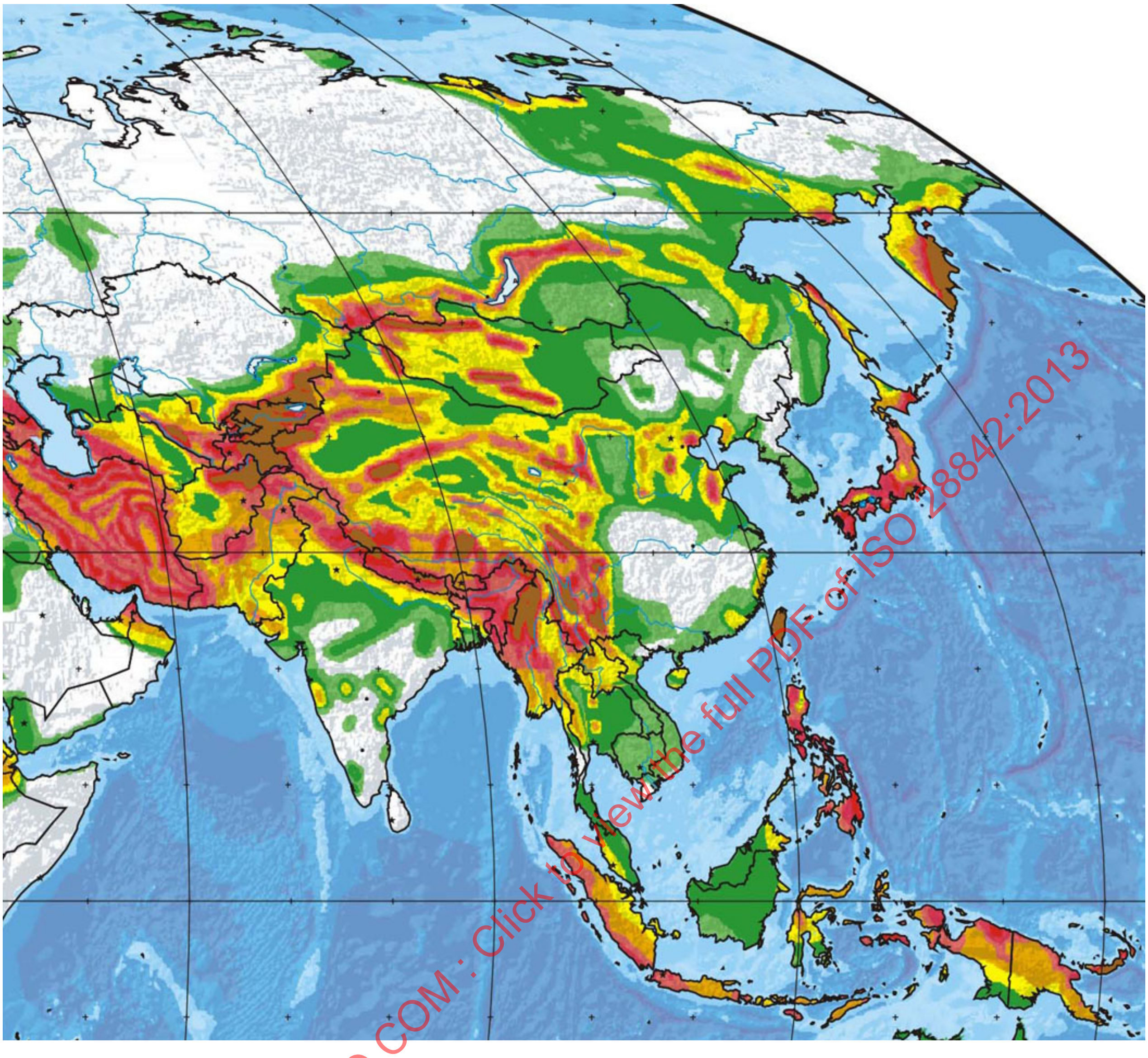
c) South America



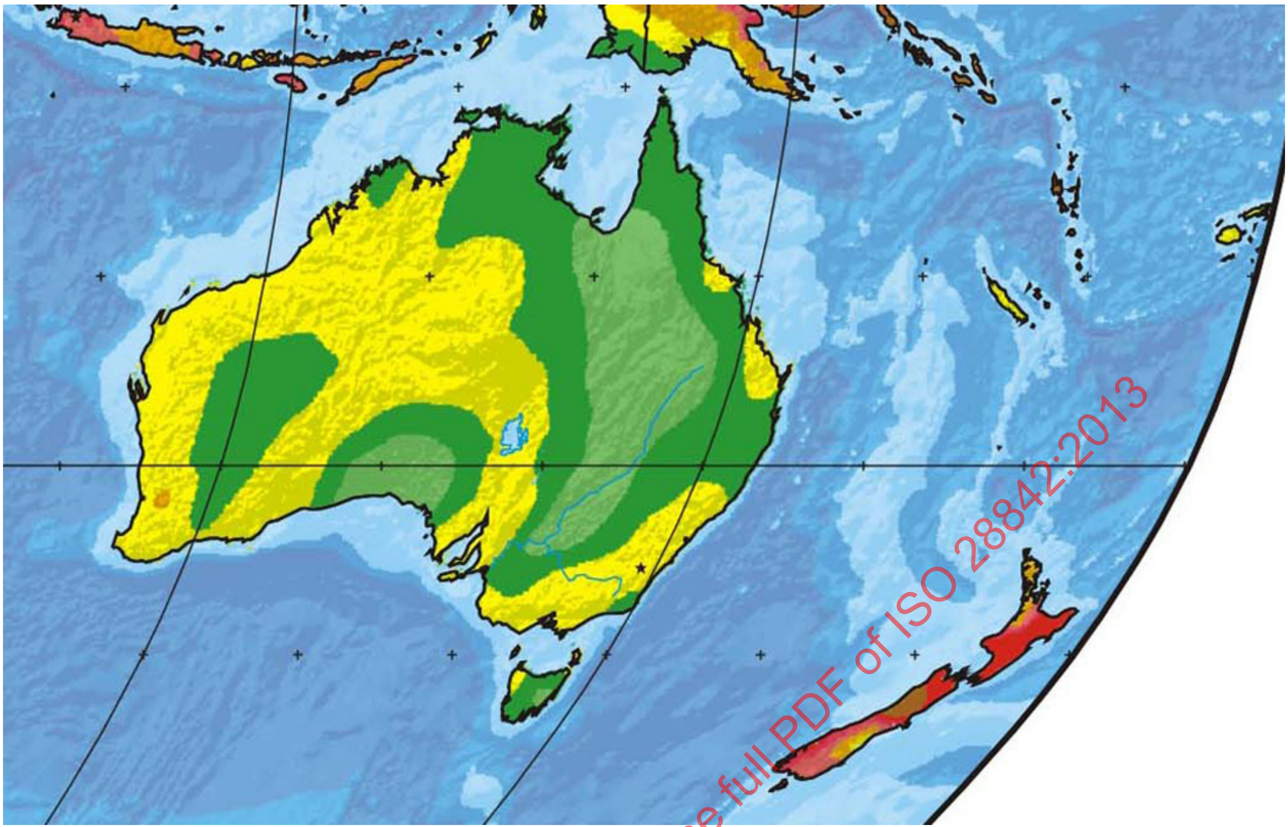
d) Europe



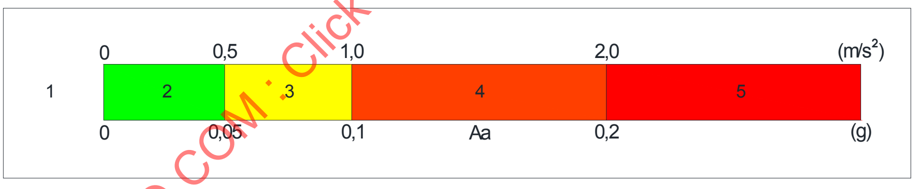
e) Africa



f) Asia



g) Oceania



Key

1 seismic hazard

2 no

3 low

4 intermediate

5 hight

Figure 8 — Global Seismic Hazard Map

8.7.7 Soil profile types

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

Soil Profile S_A: hard rock with a measured shear wave velocity v_s> 1 500 m/s;

Soil Profile S_B: rock with moderate fracturing and weathering with a measured shear wave velocity in the range (1500 m/s ≥ v_s> 750 m/s);

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

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

Soil Profile S_E: a soil profile where the measured shear wave velocity v_s≤ 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 these guidelines:

- 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) 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 classsify must 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 11 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 11.

Site effect of seismically vulnerable soils, as described in 8.7.7, are beyond the scope of these guidelines and designs should be made under the National Standard or other applicable standards.

Table 11 — Site coefficient, F_a.

Soil Profile	Site coefficoent, F_a, for short periods of vibreation				
	A_a < [0.1]	A_a < [0.2]	A_a < [0.3]	A_a < [0.4]	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]

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, expressed as a fraction of the acceleration of gravity, shall be calculated in the short periods of vibration range, using Equation 5:

$$S_a = 2.5 A_a F_a \qquad \text{Equation (5)}$$

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 Equation 6:

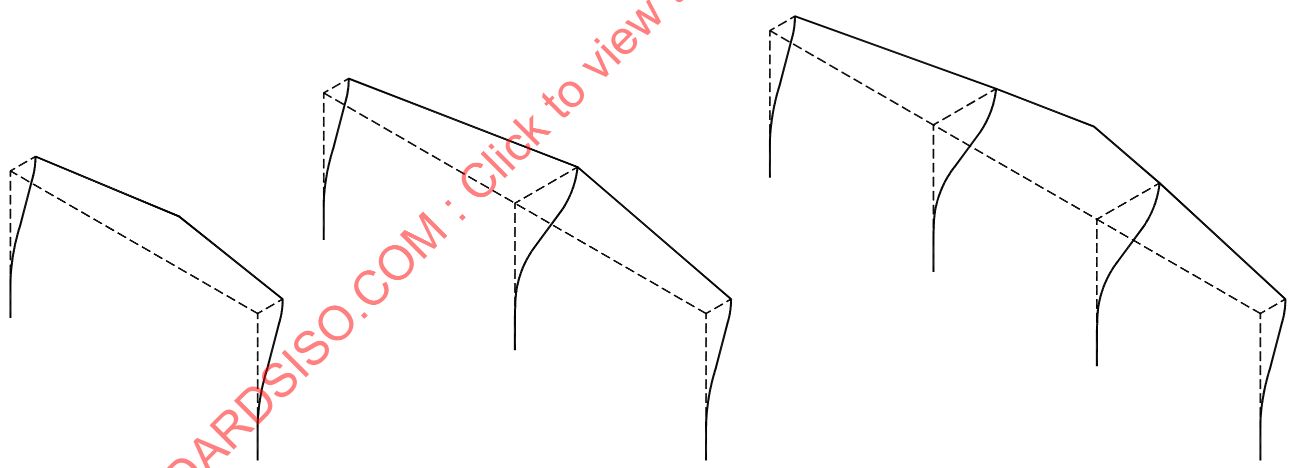
$$w_S = \frac{m_T g S_a}{L_T} \qquad \text{Equation (6)}$$

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.

In Equation 6, m_T stands for 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, g is the force of gravity, S_a is the design spectrum ordinate, and L_T is the total length of the bridge.

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 9.



a) One span b) Two spans c) Three spans

Figure 9 — Fundamental mode shape.

The fundamental shape for each case and for each span is described by a function u(x), as per Table 12.

Table 12 — Unitary deformation distribution.

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

In Table 12, x stands for any point along the considered span, L is the span's length in meters, H is the largest height of the bridge supports in meters, I_D is the moment of inertia of the deck section in m^4, for bending within its plane, due to horizontal forces, and I_P is the moment of inertia of the wall, frame or pier, in m^4, for bending due to horizontal forces.

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 Equation 7:

$$w_e = w_S u(x) \qquad \text{Equation (7)}$$

In Equation 7, w_s is a uniformly distributed load caused by the seismic ground motions, as specified in 8.7.10, and u(x) is the function describing the fundamental mode shape, as specified in 8.7.11.

8.8 Thermal Forces

Longitude change due to thermal expansion, δ_T, must be calculated for a continuous deck, as per Equation 8, at each vertical support.

$$\delta_T = \alpha \Delta_T L_C \qquad \text{Equation (8)}$$

The value for the coefficient of thermal expansion for concrete, α, depends mainly on the type of aggregate used; accepated values range from 10 x 10^{-6} $^{m/m}/_{°C}$ to 13.5 x 10^{-6} $^{m/m}/_{°C}$. A value of [11.5 x 10^{-6}] $^{m/m}/_{°C}$ may be used for α in Equation 8. ΔT is the maximum daily variation in temperature recorded at the site of the bridge. In lieu of this information, the designer may use the data provided in Table 13. Classification of temperature region for the bridge site should be made according to Figure 10.

Table 13 — Temperature variation according to world region.

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

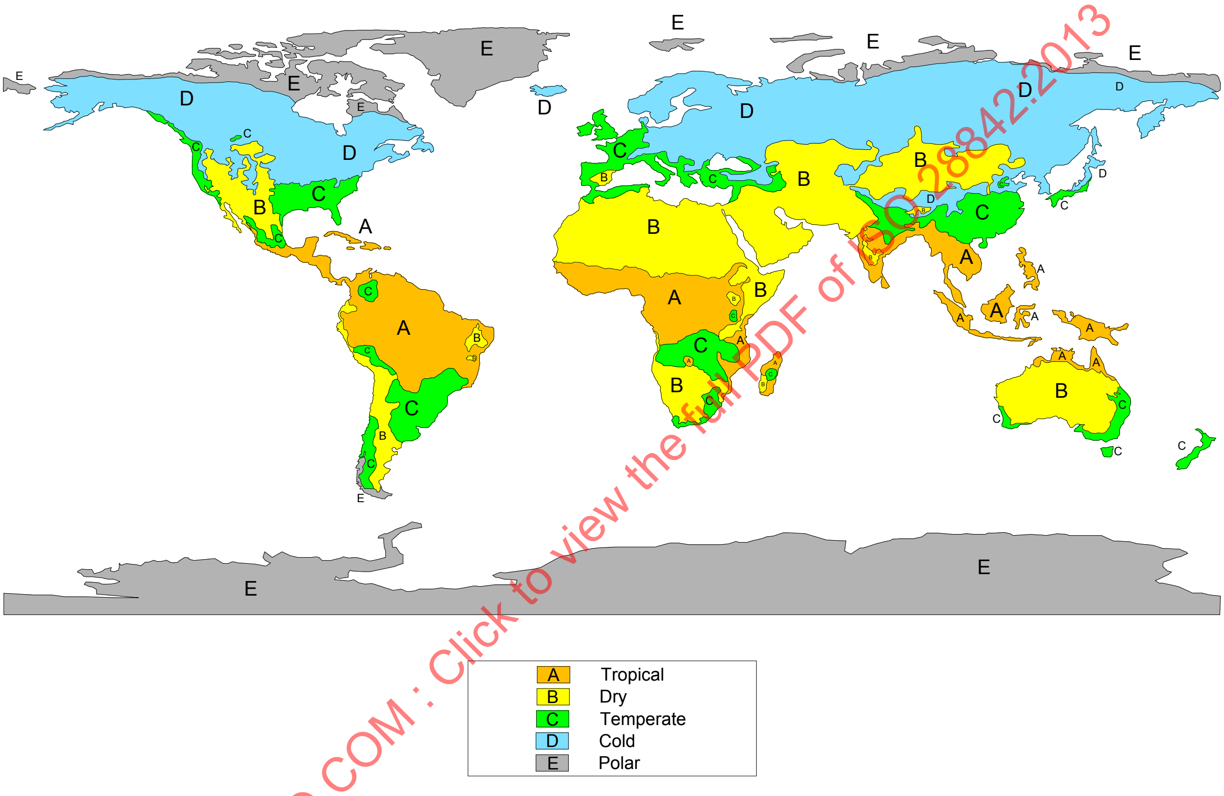


Figure 10 — Map of temperature regions of the world.

Shear force V_T caused by thermal expansion δ_T in the vertical supports should be calculated as per equation 9, where I_P is the moment of inertia, about an axis on the bridge deck perpendicular to the bridge length, of the pier, frame or wall serving as support for the superstructure and H_P is the height of the support which forces are being evaluated. Moment due to V_T should be also calculated.

$$V_T = 60 \times 10^6 \frac{I_P \delta_T}{H_P} \qquad \text{Equation (9)}$$

When elastomeric pads without lateral movements restrictions are used between the superstructure and its supports, thermal expansion of the bridge deck may not be fully transmitted to the infrastructure and, therefore, may be reduced by a factor of [0.4]. Elastomeric pads shall be designed as per 14.3.

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

8.9 Load combinations

8.9.1 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 through 8.8, increased by amplification factors, A, depending on the loads being combined in each group, as per Table 14.

Table 14 — Load amplification factors and load combinations

	AMPLIFICATION FACTORS, A				
GROUP	**D**	**L**	**LF**	**EP**	**EQ**
1	[1.35]	[1.35]	[1.00]	[1.50]	[0]
2	[1.35]	[1.5]	[0]	[1.50]	[0]
3	[1.35]	[1.7]	[0]	0.50	[0]
4	[1.35]	[1.9]	[0]	[0]	[0]
5	[1.00]	[0]	[0]	[1.50]	[0]
6	[1.2]	[0]	[0]	[1.50]	[1.00]
7	[1.00]	[0]	[0]	[0]	[1.00]
8	[1.00]	[1.3]	[0]	[0]	[1.00]
9	[0.9]	[0]	[0]	[0]	[1.00]

NOTES:

D = Dead loads

L = Live loads, including dynamic effects

LF = Longitudinal forces

EP = Earth pressure

EQ = Earthquake inertial forces

8.9.2 Service loads

For service loads the same load combinations shown in Table 14 should be used, except all factors for other than earthquake forces should be taken as 1.00. Earthquake loads factors should be taken as 0.75.

9 Design requirements

9.1 Scope

The present subclause contains the guides that are common to the structural concrete elements covered by these guidelines. They include: guides for materials, concrete cover of reinforcement, details and limits on the amount of reinforcement, and the procedures for defining the design strength of members subjected to flexural moments, axial loads with or without flexure, and shear.

9.2 Additional requirements

The designer should comply with the additional requirements for each individual element type of these guidelines.

9.3 Materials for structural concrete

9.3.1 General

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

9.3.2 Cement

Cement should conform to the following ISO Standards, or corresponding national cement standards:

ISO 679 Cement -- Test methods -- Determination of strength

ISO 863 Cement -- Test methods -- Pozzolanicity test for pozzolanic cements

9.3.3 Aggregates

Aggregates should conform to the following ISO Standards, or corresponding national aggregate standards:

ISO 6274, *Concrete — Sieve analysis of aggregates*

ISO 6782, *Aggregates for concrete — Determination of bulk density*

ISO 6783, *Coarse aggregates for concrete — Determination of particle density and water absorption — Hydrostatic balance method*

ISO 7033, *Fine and coarse aggregates for concrete — Determination of the particle mass-per-volume and water absorption — Pycnometer method*

9.3.4 Water

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

9.3.5 Steel reinforcement

Steel reinforcement should be deformed reinforcement, with the exceptions noted in 9.3.5.3, and should conform to the following limitations, and comply to the corresponding ISO standards, especially ISO 10144. Welded-wire fabric should be considered deformed reinforcement, under the present guidelines.

9.3.5.1 Deformed reinforcement

The maximum specified yield strength for deformed reinforcement should be 400 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 the present guidelines the nominal diameter of deformed reinforcement bars is limited to 32 mm (see 9.3.8).

9.3.5.2 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. Under the present guidelines the nominal diameter of wire for welded-wire fabric is limited to 10 mm (see 9.3.8).

9.3.5.3 Plain reinforcement

Plain reinforcement should be permitted only for stirrups, ties, spirals, and when it is part of a welded-wire fabric. The maximum specified yield strength for plain reinforcement should be 300 MPa. Plain reinforcing bars should conform to ISO 6935-1 or corresponding national plain reinforcement standard. ISO 6935-1 covers grades PB 240 and PB 300 (240 MPa and 300 MPa characteristic upper yield stress, respectively) and nominal diameters of (6, 8, 10,12, 16 and 20) mm, although under the present guidelines the nominal diameter of plain reinforcement bars is limited to 16 mm (see 9.3.8).

9.3.6 Admixtures

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

9.3.7 Storage of materials

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

9.3.8 Minimum and maximum reinforcement bar diameter

Reinforcement employed in structures designed under these guidelines 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 15.

Table 15 — Minimum and maximum reinforcing bar diameters

Reinforcement	Minimum bar diameter d_b	Maximum bar diameter d_b
Deformed reinforcing bars (see 9.3.5.1)	[8] mm	[32] mm
Wire for welded-wire fabric (see 9.3.5.2)	[4] mm	[10] mm
For stirrups and ties	[6] mm	[16] mm
Plain reinforcing bars (see 9.3.5.3)	[6] mm	[16] mm
For non seismic areas	[4] mm	[32] mm

9.4 Concrete Mixture Proportioning

Concrete shall be proportioned to provide an average compressive strength, $f_c{'}_r$, that shall minimize the frequency of strengths below $f_c{'}$. The requirements for $f_c{'}$ shall be based on 28-day age tests on pairs of cylinders made and tested according to ISO standards prescribed in chapter 2. 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-cement ratios, shall be established based on field experience, trial mixtures, or both, with the materials to be used.

9.4.1 Durability requirements

9.4.1.1 General

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

9.4.1.2 Calculation of the water-cement ratio

The water-cement ratios shall be calculated using the weight of water in kg/m³ of concrete divided by the cement used in the mixture in kg/m³ of concrete. The use of fly ash, pozzolans, slag, and silica fume is beyond the scope of these guidelines and if used shall be in accordance with appropriate ISO standards.

9.4.1.3 Freezing and thawing exposures

9.4.1.4 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 ± 1.5 %.Requirements for special exposure conditions

Concrete maximum water/ cement ratios and minimum specified compresive strength should comply with specification of Table 16, according to conditions of exposure.

Table 16 — Requirements for special exposure conditions

Exposure condition	Maximum water-cement ratio by weight	Minimum fc′ (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

9.4.1.5 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-cement ratio less than or equal to 0.45 by weight and a minimum compressive strength, f_c' , of 31 MPa. Calcium chloride as an admixture shall not be used in concrete exposed to sulfates.

9.4.1.6 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 17.

Table 17 — 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	1.00
Other reinforced concrete construction	0.30

9.4.2 Required average compressive strength

Required average compressive strength, f'_{cr}, for concrete shall be 10.5 MPa greater than the specified concrete compressive strength f'_c.

9.4.3 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-cement ratios that comply with the durability requirements of 9.4.1 and the slump requirements from Table 18, and that encompass the required average strength f'_{cr}. The trial mixtures shall be designed to produce slumps within ± 20 mm of the maximum permitted.

Table 18 — Slumps for various types of construction

Member	Slump (cm)	
	Maximum	Minimum
Reinforced foundation walls, columns and footings	7.5	2.5
Plain footings, caissons, and substructure walls and columns	7.5	2.5
Beams and reinforced walls	10	2.5
Columns	10	2.5
Pavements and slabs	7.5	2.5
Mass concrete	5	2.5

9.4.4 Concrete cover of reinforcement

9.4.4.1 Minimum concrete cover

The following minimum concrete cover should be provided for reinforcement, even in nonseismic areas:

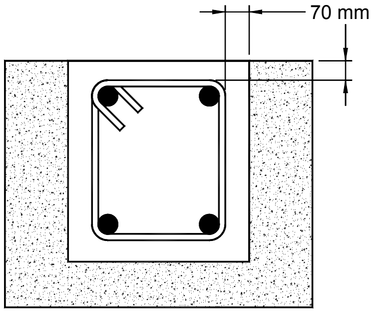


Figure 11 — Minimum concrete cover of 70 mm for all types of reinforcement of elements cast and permanently exposed to earth or water.

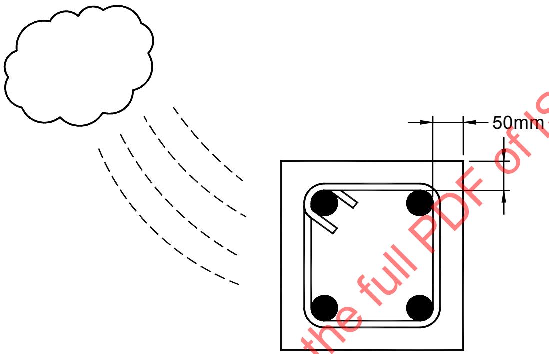


Figure 12 — All types of reinforcement of elements exposed to weather. Minimum concrete cover 50 mm

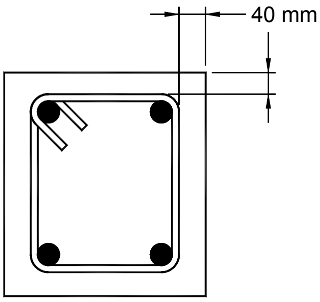


Figure 13 — All types of reinforcement of girders, beams, or columns, when not exposed to weather or in contact with ground. Minimum concrete cover 40 mm

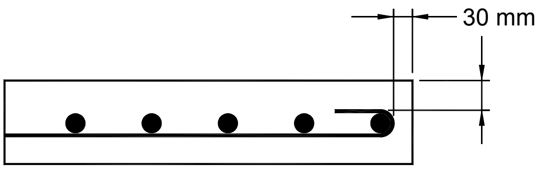


Figure 14 — All types of reinforcement of solid slabs, structural concrete walls or joists, when not exposed to weather or in contact with ground. Minimum concrete cover 30 mm

9.4.4.2 Special corrosion protection

In very aggressive environments special corrosion protection of the reinforcement should be employed, such as epoxy-coated bars, air-entrained concrete and other means. This type of protection is beyond the scope of these guidelines.

9.4.5 Minimum reinforcement bend diameter

Diameter of bend of the reinforcement, measured on the inside of the bar, should not be less than the following values:

	Diameter of bend
a) Deformed reinforcing bars	6 d_b
b) Plain reinforcing bars	6 d_b
c) For stirrups and ties	4 d_b

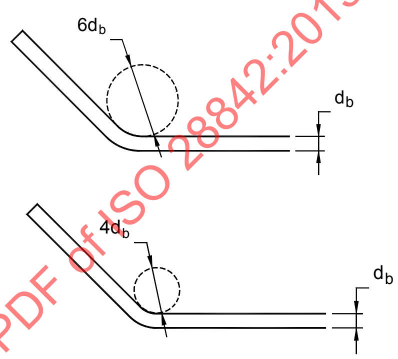


Figure 15 — Minimum reinforcement bend diameter

9.4.6 Standard hook dimensions

The term "standard hook" as used in these guidelines should mean one of the following:

a) 90° hook — a 90° bend plus a 12 d_b extension at free end of bar

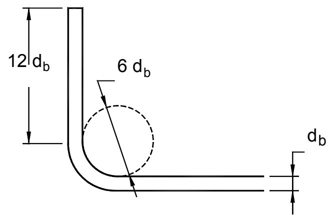


Figure 16 — 90° hook

b) 180° hook — a 180° bend plus 4 d_b extension at free end of bar

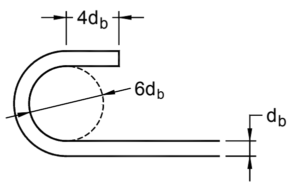


Figure 17 — 180° hook

c) For stirrup and tie hooks

- a 90° bend plus 6 d_b extension at free end of bar, or
- a 135° bend plus 6 d_b extension at free end of bar

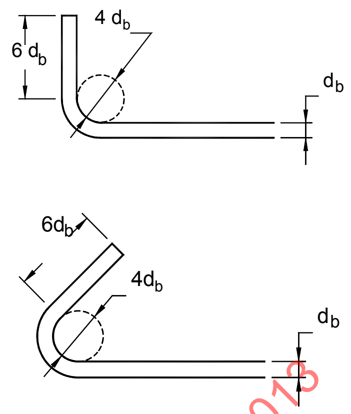


Figure 18 — For stirrup and tie hooks

d) For confinement stirrups and ties in seismic zones

a 135° bend plus 6 db extension at free end of bar, but not less than 75 mm

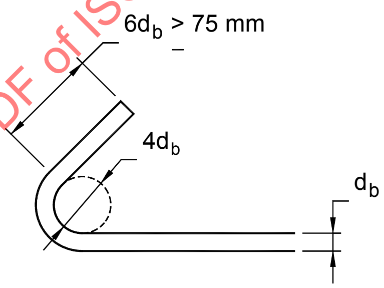


Figure 19 — For confinement stirrups and ties in seismic zones

e) For crossties in seismic zones

a 135° bend plus 6 db extension at free end of bar, but not less than 75 mm

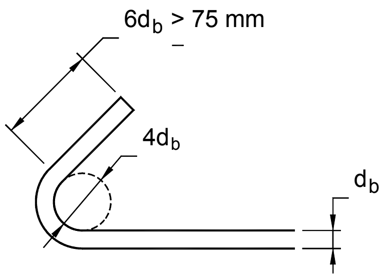


Figure 20 — For crossties in seismic zones

9.4.7 Bar spacingand maximum aggregate size

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

9.4.8 Maximum nominal coarse aggregate size

Maximum nominal coarse aggregate size, see Figure 21, 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.

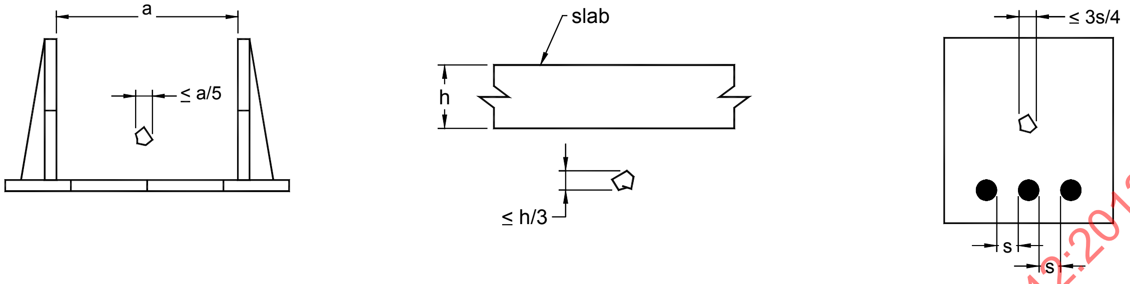


Figure 21 — Maximum nominal coarse aggregate size

9.4.9 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 22. These guides should apply also for the spacing between parallel stirrups or ties.

9.4.10 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 22.

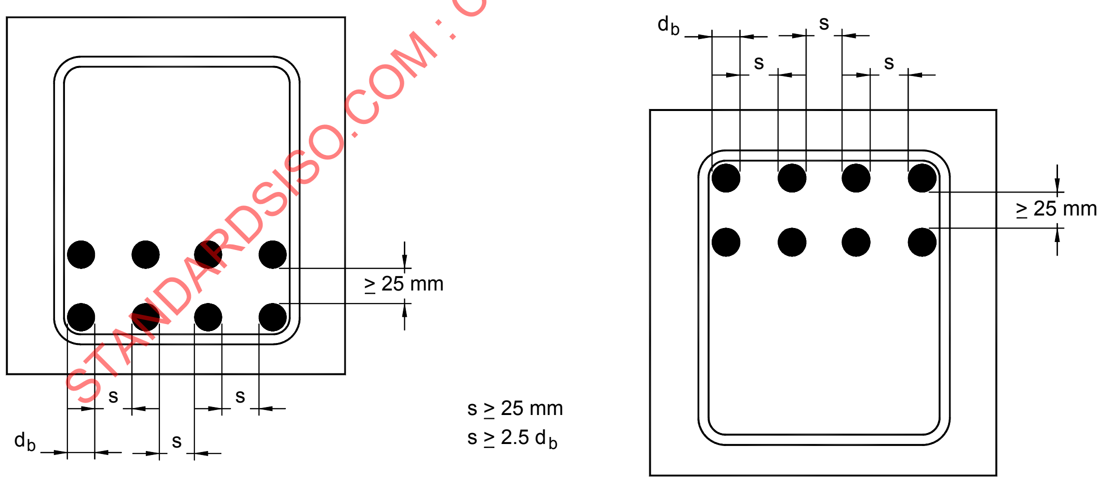


Figure 22 — Minimum clear spacing between parallel bars in a layer, and clear distance between parallel layers of reinforcement

9.4.11 Minimum clear spacing between longitudinal bars in columns

In columns, clear distance between longitudinal bars should not be less than 1,5 db or 40 mm. See Figure 23.

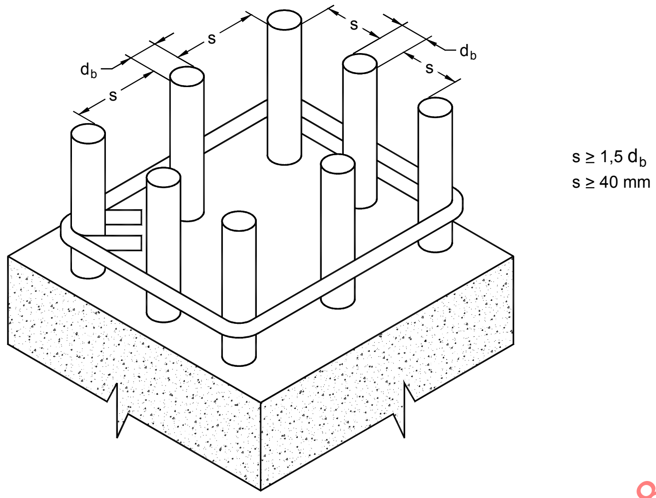


Figure 23 — Clear distance between longitudinal bars in columns

9.4.12 Clear spacing between parallel lap splices

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

9.4.13 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 24).

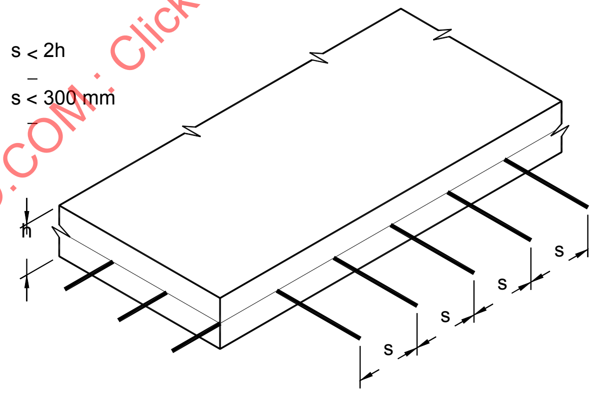


Figure 24 — Spacing between flexural reinforcement in solid slabs

9.4.14 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 25.

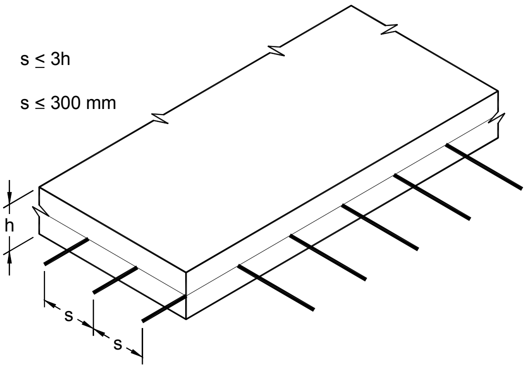


Figure 25 — Spacing between shrinkage and temperature reinforcement in slabs

9.4.15 Maximum reinforcement spacing in structural concrete walls

9.4.15.1 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 26.

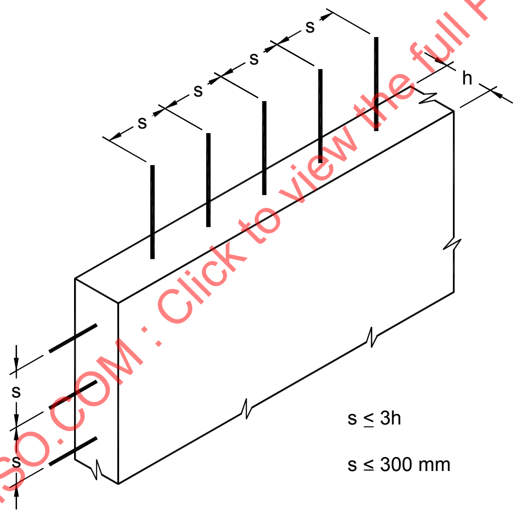


Figure 26 — Spacing between reinforcement in structural concrete walls

9.4.15.2 Number of layers of reinforcement

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

9.4.15.3 Special details per element type

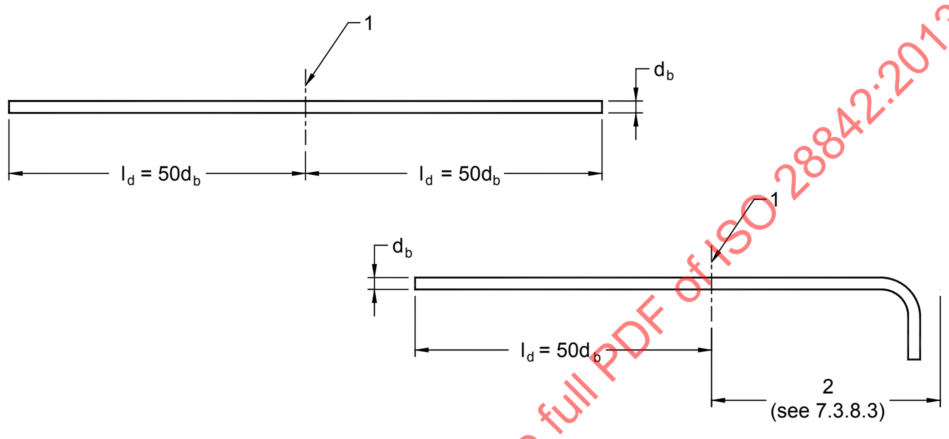
The designer should comply with the additional reinforcement detail required for each individual element type, as guide by 10 to 14 of these guidelines.

9.5 Development length, lap splicing and anchorage of reinforcement

9.5.1 Development length

9.5.1.1 Reinforcing bars

The minimum length of embedment, l_d, required on each side of a critical section, for a reinforcing bar to develop its full strength should be 50 d_b, for the bar diameters permitted by these guidelines in 9.3.8. It should be permitted to replace development length in one side of the critical section by a length of bar ending in a standard hook complying with the minimum anchorage distance of 9.5.3. See Figure 27.



Key

1 critical section

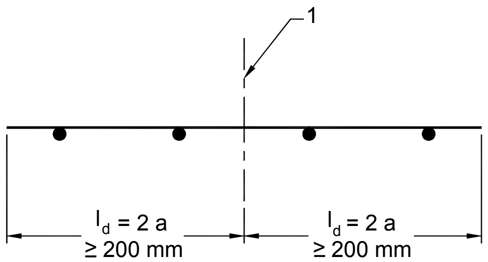
2 anchorage distance (see 7.3.8.3)

Figure 27 — Required development length for reinforcing bars

Whenever plain bars may be used instead of deformed bars, the development length specified here must be multiplied by 1.8.

9.5.1.2 Welded-wire fabric

The development length l_d, of welded-wire fabric measured on each side of the critical section to the end of wire should contain two cross-wires, but should not be less than 200 mm, for the wire diameters permitted by these guidelines in 9.3.8. See Figure 28.



Key

1 critical section

Figure 28 — Required development length for welded-wire fabric

9.5.2 Lap splice dimensions

9.5.2.1 Reinforcing bars

The minimum length of lap for splicing of reinforcing bars should be 50 db, for the bar diameters permitted by these guidelines in 9.3.8. See Figure 29.

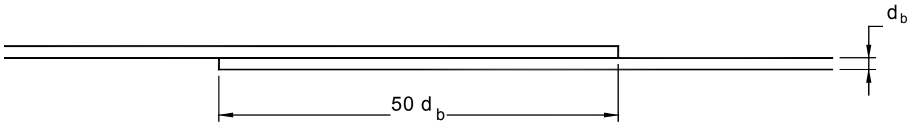


Figure 29 — Minimum lap splice length for reinforcing bars

9.5.2.2 Welded-wire fabric

Welded-wire fabric splicing should be attained by superimposing two cross-wires, but the distance between the edge cross-wires should not be less than 250 mm, for the wire diameters permitted by these guidelines in 9.3.8, See Figure 30.

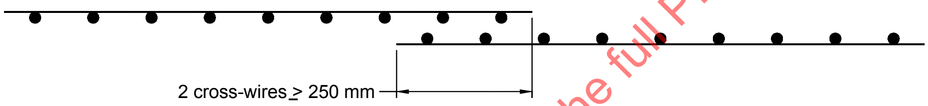
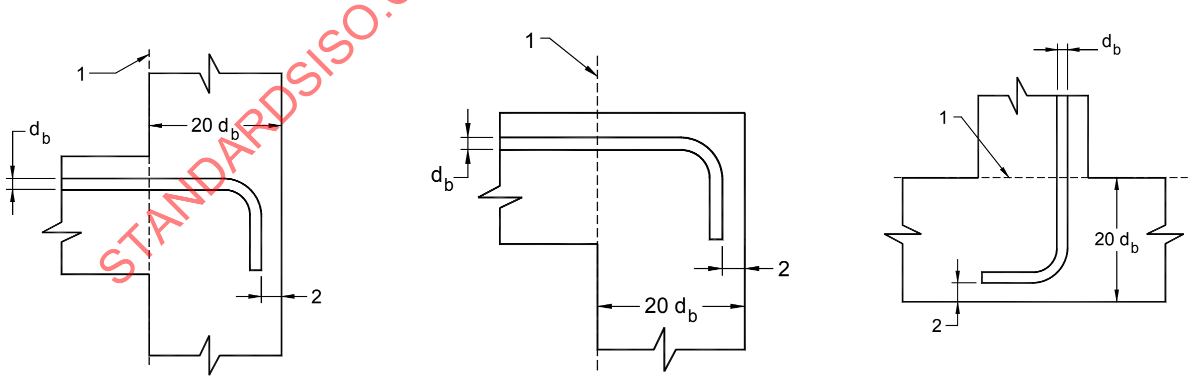


Figure 30 — Minimum lap splice length for welded-wire fabric

9.5.3 Minimum standard hook anchorage distance

The minimum distance between the outer face of concrete and the critical section where the hooked bar develops its full strength should not be less than 20 d_b. See figure 31.



Key

1 critical section

2 cover requirement

Figure 31 — Minimum standard hook anchorage distance

9.6 Limits for longitudinal reinforcement

9.6.1 General

Longitudinal reinforcement in reinforced concrete structural elements should be provided to resist axial tension, axial compression, flexural 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 these guidelines should be that required to resist the factored loads and forces, but should be not less than the minimum values given in 9.6. The dimensions of the structural element should be appropriately modified when the amount of calculated reinforcement required to resist the factored loads and forces exceed the maximum amounts permitted by 9.6.

9.6.2 Solid slabs and footings

9.6.2.1 Minimum area of shrinkage and temperature reinforcement

Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement should be provided in structural solid slabs and footings where flexural reinforcement extends in one direction only. See Figure 32. The maximum spacing for this reinforcement should comply with 9.4.14. The following minimum ratios of reinforcement area to gross concrete area, ρ_t, should be provided for shrinkage and temperature:

a. where deformed bars with f_y < 350 MPa are used $\rho_t \geq 0{,}0020$

b. where deformed bars or welded-wire fabric with $f_y \geq$ 350 MPa are used $\rho_t \geq 0{,}0018$

9.6.2.2 Minimum area of tension flexural reinforcement

The minimum area of tension flexural reinforcement, $A_{s,min}$, in structural solid slabs and footings should be greater or equal to the reinforcement area required for shrinkage and temperature stresses as required by 9.6.2.1, ($A_{s,min} \geq \rho_t \cdot b \cdot h$). See Figure 32. The maximum spacing of this reinforcement should comply with 9.4.13.

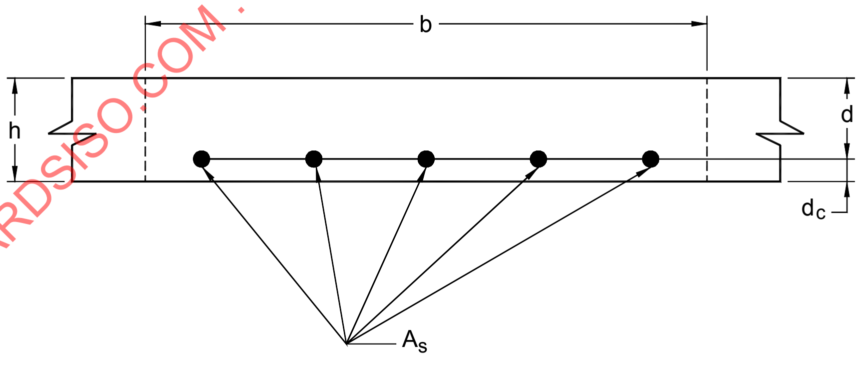


Figure 32 — Slab or footing section

9.6.2.3 Maximum area of tension flexural reinforcement

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

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

		f_y (MPa)		
		[240]	[300]	[400]
f'_c (MPa)	[20]	[0,0220]	[0,0160]	[0,0110]
	[25]	[0,0270]	[0,0200]	[0,0140]
	[30]	[0,0320]	[0,0240]	[0,0160]

NOTE It should be permitted to interpolate for different values of f_y and f'_c

9.6.3 Girders, beams and joists

9.6.3.1 Minimum area of tension flexural reinforcement

At every section of a girder, beam or joist, where tension flexural reinforcement is required by 10.1, the minimum area of tension flexural reinforcement, $A_{s,min}$, should be greater or equal to the following values, where ρ_{min}, is the value stipulated in Table 20:

For rectangular sections, and for T sections where the flange is in compression (See Figure 33):

$$A_{s,min} = \rho_{min} \cdot d \cdot b_w \qquad \text{Equation (10)}$$

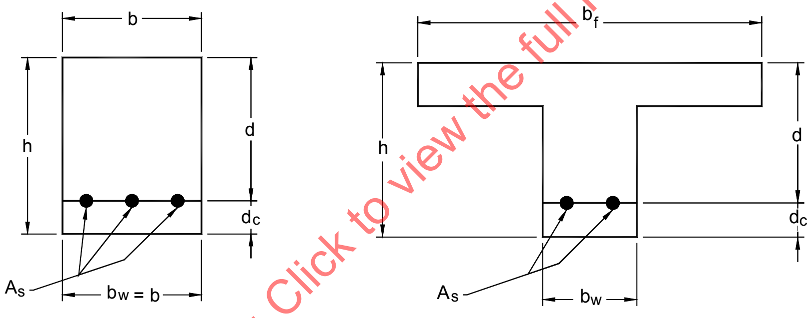


Figure 33 — Rectangular section and T-shaped section with flange in compression

For T sections where the flange is in tension (see Figure 34), should be greater or equal to the smaller value obtained from Equation 11or Equation 12:

$$A_{s,min} = 2 \cdot \rho_{min} \cdot d \cdot b_w \qquad \text{Equation (11)}$$

$$A_{s,min} = \rho_{min} \cdot d \cdot b_f \qquad \text{Equation (12)}$$

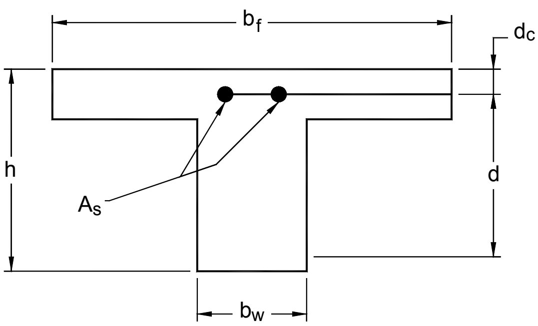


Figure 34 — T-shaped section with flange in tension

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

		f_y (MPa)		
		240	300	400
f'_c (MPa)	20	0,0047	0,0037	0,0028
	25	0,0052	0,0042	0,0031
	30	0,0057	0,0046	0,0034

NOTE It should be permitted to interpolate for different values of f_y and f'_c, or use the following equation:

$$\rho_{min} \geq 0{,}25 \frac{\sqrt{f'_c}}{\sqrt{f_y}} \geq \frac{1{,}4}{f_y}$$

9.6.3.2 Maximum flexural reinforcement ratios

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

In girders, beams and joists, having only tension flexural reinforcement:

$$\rho = \frac{A_s}{b \cdot d} \leq \rho_{max}$$ Equation (13)

In girders, beams and joists, having tension and compression flexural reinforcement (See Figure 35):

$$\rho - \rho' = \frac{A_s - A'_s}{b \cdot d} \leq \rho_{max}$$ Equation (14)

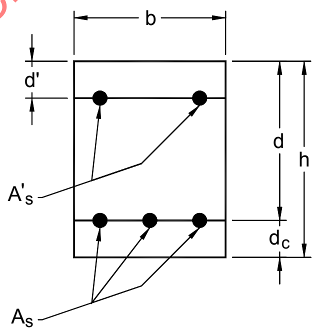


Figure 35 — Section with tension and compression reinforcement

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

		f_y (MPa)		
		240	300	400
f'_c (MPa)	15	0,0240	0,0180	0,0120
	20	0,0320	0,0240	0,0160
	25	0,0400	0,0300	0,0200
	30	0,0480	0,0360	0,0240

NOTE It should be permitted to interpolate for different values of fy and f_c' or use the following equation:

$$\rho_{máx} \geq 0{,}55 \frac{f_c'}{f_y} \cdot \frac{600}{600 + f_y}$$

9.6.4 Columns

9.6.4.1 Minimum and maximum area of longitudinal reinforcement

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

$$0{,}01 \leq \rho_t = \left[\frac{A_{st}}{A_g}\right] \leq 0{,}06 \qquad \text{Equation (15)}$$

9.6.4.2 Minimum diameter of longitudinal bars

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

9.6.4.3 Minimum number of longitudinal bars

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

9.6.4.4 Distribution of longitudinal bars

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

9.6.5 Structural concrete walls

9.6.5.1 Minimum area of vertical reinforcement

The minimum ratio, ρ_v, of vertical reinforcement area to gross concrete horizontal section area should be 0,0025.

9.6.5.2 Maximum area of vertical reinforcement

The maximum ratio, ρ_v, of vertical reinforcement area to gross structural concrete wall horizontal section area should be 0,06, but when the ratio, ρ_v, exceeds 0,01 the vertical reinforcement should be enclosed with ties as prescribed for columns in 9.7.4.1.

$$0{,}0025 \leq \rho_v = \left[\frac{A_{st}}{b_w \cdot l_w}\right] \leq 0{,}06 \qquad \text{Equation (16)}$$

9.7 Minimum amounts of transverse reinforcement

9.7.1 General

Transverse reinforcement in reinforced concrete structural elements should be provided to resist shear, diagonal tension, and torsion stresses. It should be provided also to counteract the tendency of compression loaded bars to buckle out of the concrete by bursting the thin outer concrete cover, and to prevent displacement of the longitudinal reinforcement during construction operations. In seismic zones it should be placed in special regions of the structural elements to provide confinement of concrete subjected to stresses in the non-linear range. The amount of transverse reinforcement employed in the structural elements covered by these guidelines should be that required to resist the factored loads, forces, and stresses, but should be not less than the minimum values given by 9.7.4. The dimensions of the structural element should be appropriately modified when the amount of calculated reinforcement required to resist the factored loads, forces and stresses, exceed the maximum amounts permitted by 9.7.4.

9.7.2 Slabs

The design procedures for slabs prescribed by these guidelines do not require the employment of transverse reinforcement in slabs. The procedures for design of transverse or shear reinforcement in slabs are beyond the scope of these guidelines.

9.7.3 Girders, beams and joists

9.7.3.1 Minimum transverse reinforcement

The minimum transverse reinforcement in girders, beams and joist should be the required for shear, as specified in 10.2.4.3and 10.2.4.4, with the exceptions noted in 9.7.3.2

9.7.3.2 Girders and beams in seismic zones

Girders and beams framing into columns and structural concrete walls located in seismic zones should be provided with confining transverse reinforcement as required in 13.

9.7.4 Columns

All columns should have transverse reinforcement in the form of either tie reinforcement or spiral reinforcement conforming to the guides of 9.7.4.1 or 9.7.4.2, respectively.

9.7.4.1 Ties

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

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

b) Ties should be arranged in such a manner that every corner and alternate longitudinal bar should have lateral support provided by the corner of a tie or a crosstie. See Figure 36.

c) No longitudinal bar should be farther than 150 mm clear on each side along the tie from a laterally supported longitudinal bar. See Figure 36.

d) The vertical spacing of ties, s, should not exceeda half of the effective depth of the column section. See Figure 37.

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

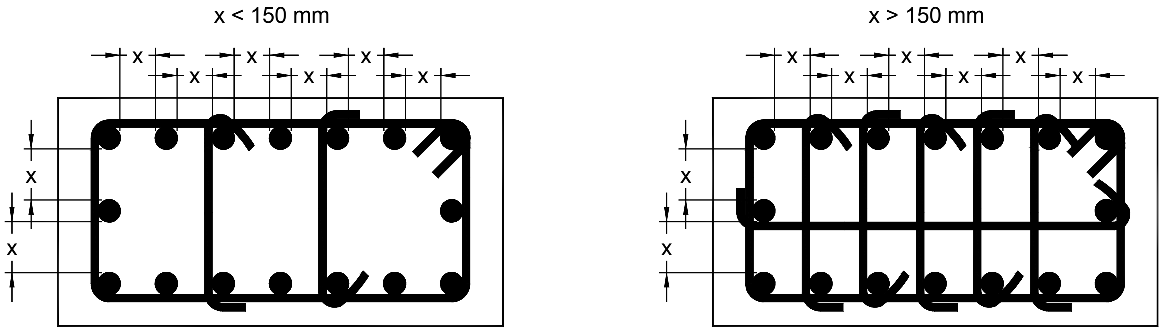
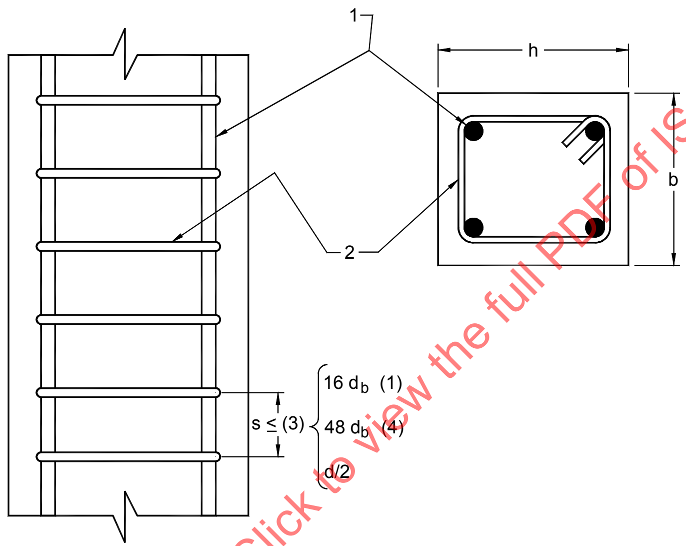


Figure 36 — Arrangement of ties in a tied column section



Key

1 longitudinal bars

2 tie

3 smaller of

4 tie bar

Figure 37 — Vertical spacing of ties in a tied column

9.7.4.2 Spirals

Columns with spiral reinforcement should comply with the following guides:

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

b) Clear spacing between spirals should not exceed 80 mm, nor be less than 25 mm, and should comply with the guides of 9.4.7.

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

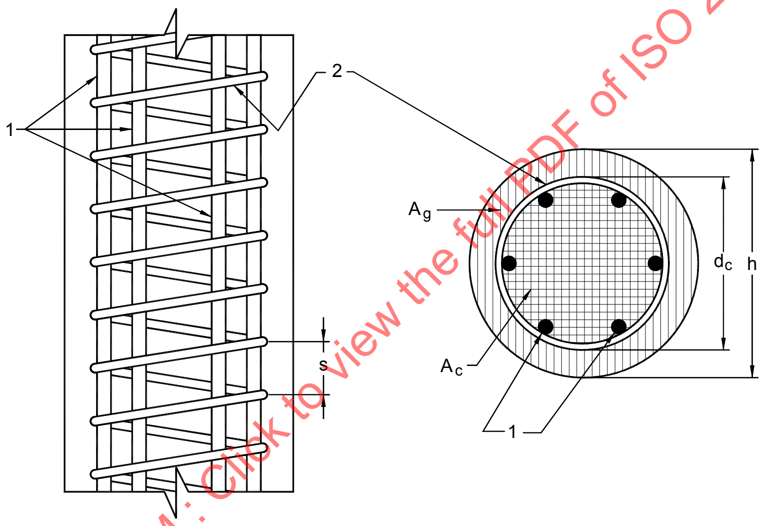
d) Splices in spiral reinforcement should comply with 9.5.2.

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

f) Ratio of spiral reinforcement, ρ_s, defined as ratio of the volume of reinforcement contained in one loop of the spiral to the volume of concrete in the core of the column confined by the same loop of spiral, should be not less than any of the values given by Equation 17. See Figure 38:

$$\rho_s = \frac{A_b \cdot \pi \cdot d_c}{A_c \cdot s} \geq \left[\begin{array}{c} 0{,}12 \cdot \dfrac{f'_c}{f_{ys}} \\ 0{,}45 \cdot \left[\dfrac{A_g}{A_c} - 1 \right] \cdot \dfrac{f'_c}{f_{ys}} \end{array} \right. \quad \text{Equation (17)}$$

Where A_b is the area of the bar of spiral, d_c is the center-to-center diameter of the spiral, s is the vertical spacing of the spiral, A_c is the area of the confined column core measured center to center of the spiral $\left(A_c = \dfrac{\pi \cdot d_c^2}{4} \right)$, A_g is the gross column section area, f'_c is the specified concrete strength of the column, and f_{ys} is the yield strength of the steel of the spiral.



Key

1 longitudinal bars

2 spiral

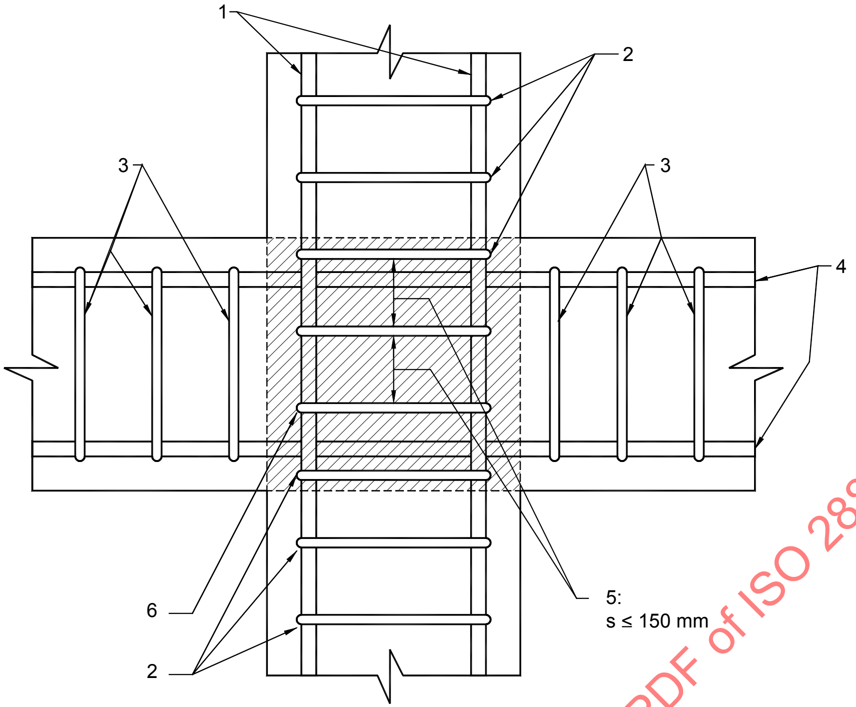
Figure 38 — Spiral reinforcement of column

9.7.4.3 Column-girder joints

At joints of frames where columns and girders meet, a minimum of three column ties, complying with 9.7.4.2 (a) to 9.7.4.2 (c), should be provided within the joint and the maximum vertical spacing between ties should be 150 mm. As many ties, as necessary to comply with the maximum spacing should be provided. See Figure 39.

9.7.5 Structural concrete walls

The minimum ratio, ρ_h, of horizontal reinforcement area to gross concrete vertical section area should be 0,0025.



Key

1 column longitudinal reinforcement

2 column ties

3 girder stirrups

4 girder longitudinal reinf.

5 joint ties

6 joint

Figure 39 — Column ties in column-girder joints

10 Superstructure

The superstructure system employed by a bridge designed under these guidelines should be one of the systems covered or their permitted variations. The selection of an appropriate system should be performed studying several alternatives.

10.1 Strength of members subjected to flexural moments

10.1.1 General

Calculation of the design strength of member sections subjected to flexural moments should be performed employing the requirements of 10.1. If the factored axial compressive load on the member, P_u, exceeds $\left(0{,}10 \cdot f_c' \cdot A_g\right)$, the calculation of the design strength should be performed employing the requirements of 10.2.

10.1.2 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 requirements of 10 to 14.

10.1.3 Minimum design flexural moment strength

The design flexural moment strength of the section, ($\phi \cdot M_n$), should be greater or equal than the factored flexural moment at that section, M_u, as shown in Equation 18.

$$\phi \cdot M_n \geq M_u \qquad \text{Equation (18)}$$

10.1.4 Design moment strength for rectangular sections with tension reinforcement only

10.1.4.1 Design moment strength

For a section with tension reinforcement only, the design moment strength at the section should be obtained using Equation19:

$$\phi \cdot M_n = \phi \cdot A_s \cdot f_y \left(d - \frac{a}{2} \right) \qquad \text{Equation (19)}$$

where the depth of the equivalent uniform stress block, a, should be (see Figure 40):

$$a = \frac{A_s \cdot f_y}{0{,}85 \cdot f_c' \cdot b} \qquad \text{Equation (20)}$$

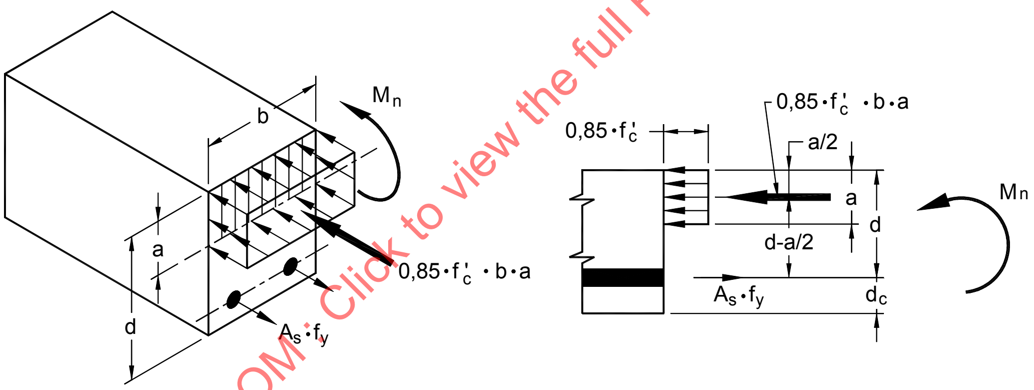


Figure 40 — Flexural nominal moment strength

It should be permitted to use Equation 21, where the value of a has been introduced in Equation 19, through Equation 21:

$$\phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot d \cdot \left(1 - 0{,}59 \cdot \frac{A_s \cdot f_y}{b \cdot d \cdot f_c'} \right) \qquad \text{Equation (21)}$$

For the purposes of these guidelines, it should be permitted to approximate the design moment strength in slabs, and also in girders, beams and joists where $\rho < \frac{\rho_{máx}}{2}$, with ρ_{max} from Table 21, as:

$$\phi \cdot M_n \approx \phi \cdot A_s \cdot f_y \cdot 0{,}85 \cdot d \qquad \text{Equation (22)}$$

10.1.4.2 Obtaining the flexural tension reinforcement area

The required ratio of flexural reinforcement, $\rho = \frac{A_s}{(b \cdot d)}$, should be obtained combining Equation 18 with Equation 19, and using the factored flexural moment, M_u, as:

$$\rho = \frac{A_s}{b \cdot d} = \alpha - \sqrt{\alpha^2 - \left(\frac{M_u}{\phi \cdot b \cdot d^2} \cdot \frac{2 \cdot \alpha}{f_y}\right)} \quad \text{where } \alpha = \frac{f_c'}{1{,}18 \cdot f_y} \qquad \text{Equation (23)}$$

or using the approximate Equation 22, in slabs where $\rho < \rho_{max}$, with ρ_{max} from Table 19, and in girders, beams and joists where $\rho < \frac{\rho_{máx}}{2}$, with ρ_{max} from Table 21, as:

$$\rho = \frac{A_s}{b \cdot d} \approx \frac{M_u}{\phi \cdot b \cdot d^2 \cdot 0{,}85 \cdot f_y} \qquad \text{Equation (24)}$$

In Equation 23 and Equation 24 ϕ = [0,90]. If the value obtained from Equation 23 or Equation 24 is smaller than ρ_{min} from 9.6.3.1 ρ should be increased to that value. For slabs, if the obtained value of ρ is greater than ρ_{max} from Table 19, the slab depth, h, should be increased, correcting the selfweight of the slab. For girders, beams, and joists, if the obtained value of ρ is greater than ρ_{max} from Table 21, the possibility of either using compression reinforcement (see 10.1.5), or changing dimensions, making the appropriate correction for the selfweight, should be investigated.

10.1.5 Use of compression reinforcement in girders, beams, and joists

10.1.5.1 Tension reinforcement less than maximum

If the ratio of tension reinforcement, ρ, is less than ρ_{max} as given in 9.6.3.2, the effect of reinforcement in the compression face of the element should be permitted to be disregarded.

10.1.5.2 Shallow doubly reinforced sections

If the ratio of $\frac{d'}{d}$ is greater than the values given in Table 22 the compression reinforcement should be considered not to be effective.

Table 22 — Maximum values of $\frac{d'}{d}$ for compression reinforcement to be effective

f_y (MPa)	240	300	400
$\frac{d'}{d}$	0,320	0,250	0,150

NOTE It should be permitted to interpolate for different values of fy

10.1.5.3 Design moment strength of sections with compression reinforcement

When the condition of $\frac{d'}{d}$ is met, the design moment strength at the section, should be (see Figure 41):

$$\phi \cdot M_n = \phi \cdot \left[\left(A_s - A_s'\right) \cdot f_y \left(d - \frac{a}{2}\right) + A_s' \cdot f_y \cdot \left(d - d'\right) \right] \qquad \text{Equation (25)}$$

where the depth of the equivalent uniform stress block, a, should be in this case:

$$a = \frac{\left(A_s - A_s'\right) \cdot f_y}{0{,}85 \cdot f_c' \cdot b}$$ Equation (26)

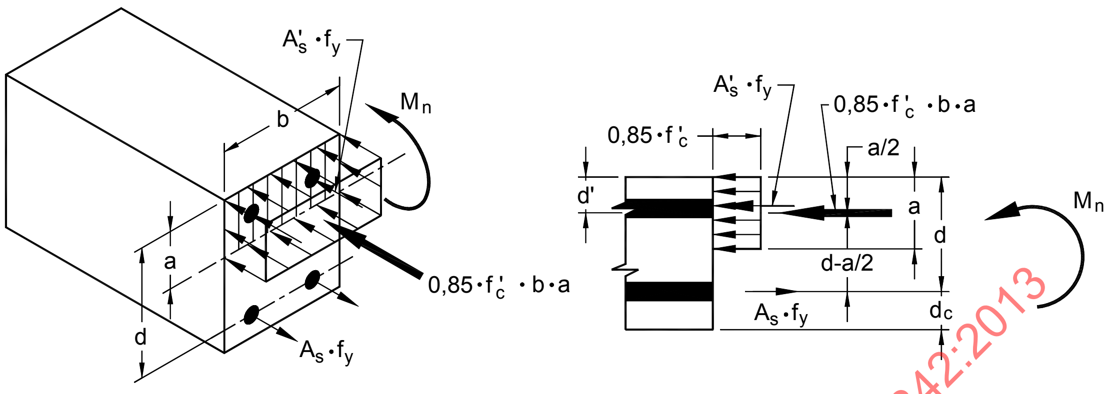


Figure 41 — Flexural nominal moment strength for doubly reinforced sections

10.1.5.4 Obtaining the flexural tension and compression reinforcement area

The required area of flexural tension reinforcement, A_s, and compression reinforcement, A_s', should be obtained combining Equation 18 with Equation 25, and using the factored flexural moment, M_u, as follows:

$$A_s' = \frac{M_u}{\phi \cdot f_y \cdot (d - d')} - \left[b \cdot d^2 \cdot \rho_{max} \cdot f_y \cdot 0{,}8\right]$$ Equation (27)

and

$$A_s = A_s' + \rho_{max} \cdot b \cdot d$$ Equation (28)

In Equation 27 and Equation 28, ϕ = [0,90]. The steel ratio, ρ_{max}, should be obtained from Table 21. This procedure should be used only when the condition of $\frac{d'}{d}$ of 10.1.5.2 is met. Compression reinforcement should be enclosed by ties as required by 9.7.3.2.

10.1.6 T-beam effect

In beams that are cast monolithically with a slab, and when subjected to flexural moments that induce compression stresses in the slab a portion of the slab should be permitted to act as a flange of the beam, and the flexural design should comply with the requirements of 10.1.6.1 to 10.1.6.5.

10.1.6.1 Effective flange width for beams with slab in both sides

The width of slab effective as a T-beam flange, b, should not exceed (see Figure 42):

a) one-quarter of the span length of the beam,

b) sixteen times the slab thickness h_f, plus the web thickness, b_w,

c) the clear distance between webs plus the web thickness, b_w.

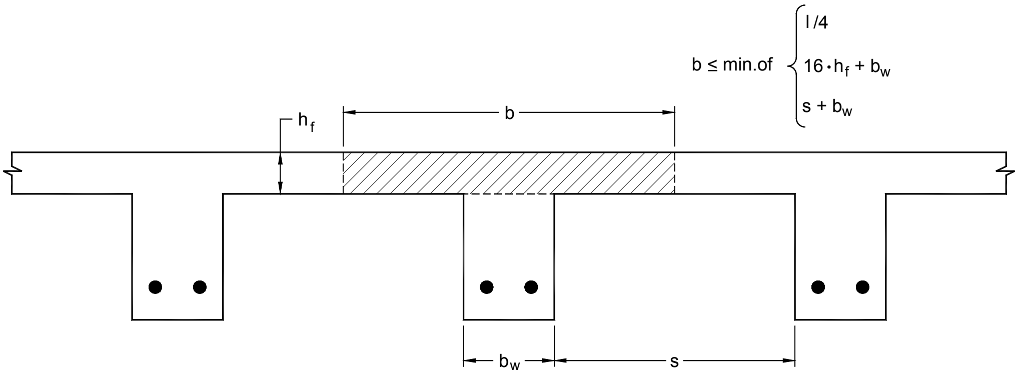


Figure 42 — Effective flange width for T-beams with slab in both sides

10.1.6.2 Effective flange width for beams with slab in one side only

The width of slab effective as a T-beam flange, b, should not exceed (see Figure 43):

one-twelfth of the span length of the beam plus the web thickness, b_w,

six times the slab thickness h_f, plus the web thickness, b_w,

one-half the clear distance to the next web plus the web thickness, b_w.

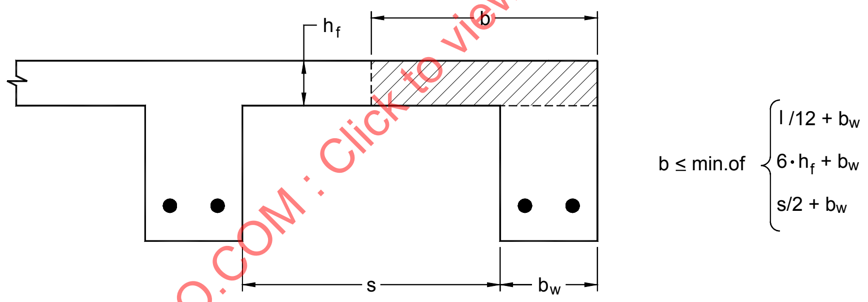


Figure 43 — Effective flange width for T-beams with slab in one side only

10.1.6.3 Isolated T-beams

The flange thickness h_f, in isolated T-beams should be at least one-half of the web thickness, b_w, and the effective flange width, b, should not exceed $4 \cdot b_w$ nor b_f (see Figure 44).

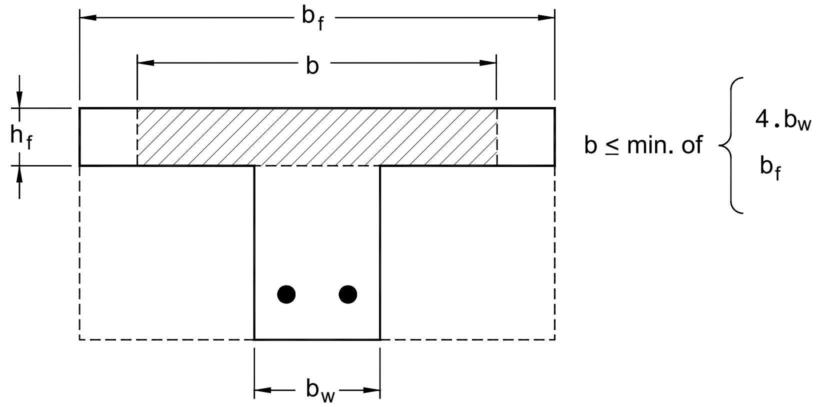


Figure 44 — Effective flange width for isolated T-beams

10.1.6.4 Design moment strength of T-beams

When the flange is in compression the moment strength should be calculated as for a rectangular beam using 10.1.4.1, as long as the depth of the equivalent uniform stress block, a, lies within the flange thickness, h_f. See Figure 45. The last condition should be verified using Equation 29.

$$h_f \geq a \quad \text{and} \quad a = \frac{A_s \cdot f_y}{0{,}85 \cdot f_c' \cdot b} \qquad \text{Equation (29)}$$

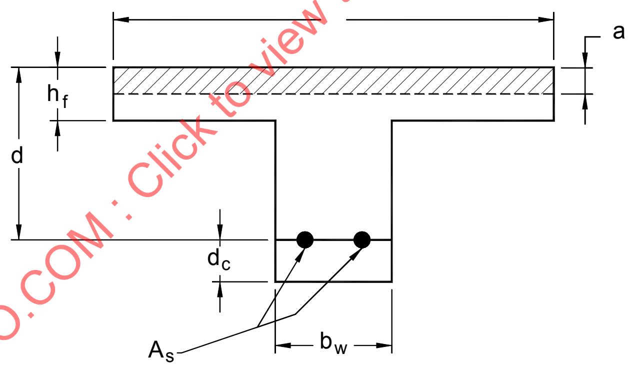


Figure 45 — Effective cross section for moment strength calculation of T-beams

10.1.6.5 Obtaining the flexural tension reinforcement area

The required ratio of flexural reinforcement, $\rho < \frac{A_s}{(b \cdot d)}$ for T-beams, should be obtained from Equation 23 or Equation 24, and the flexural reinforcement ratio, ρ, should not exceed the value given by Equation 30, in order for the depth of the equivalent uniform stress block, a, to lie within the flange thickness, h_f.

$$\rho \leq \frac{0{,}85 \cdot f_c' \cdot h_f}{f_y \cdot d} \qquad \text{Equation (30)}$$

If the value obtained from Equation 23 or Equation 24 is smaller than ρ_{min} from 9.6.3.1 ρ should be increased to that value. If the obtained value of ρ is greater than ρ_{max} from Table 20, the dimensions should be changed, making the appropriate correction for the selfweight.

10.2 Strength of members subjected to shear stresses

10.2.1 General

Calculation of the design strength of member sections subjected to diagonal tension or shear stresses should be performed employing the requirements of 10.2. Two type of shear stress effects are covered by these guidelines:

beam-action shear that accompany flexural moments and occurs in girders, beams, joists, solid slabs , in the vicinity of supports and concentrated loads, and

punching-shear or two-way action shear, that occurs in solid slabs, also in the vicinity of supports and concentrated loads.

Other types of diagonal tension effects, such as: special effects in deep flexural members, shear-friction employed in the design of brackets and corbels, and strut-and-tie models, are beyond the scope of these guidelines.

10.2.2 Factored shear

The factored shear, V_u, caused by the factored loads applied to the structure should be determined, for the particular element type, from the requirements of 10 to 14.

10.2.3 Design shear strength

The design shear strength at the section of the element, ($\phi \cdot V_n$), should be greater or equal than the factored shear, V_u, as shown in Equation 31.

$$\phi \cdot V_n \geq V_u \qquad \text{Equation (31)}$$

In Equation 31, ϕ = [0,85].

10.2.4 Beam-action shear

10.2.4.1 General

The guides in 10.2.4 should be applied to the design of members for beam-action shear. The following general guides should be employed:

a) where shear reinforcement is used the design shear strength, $\phi \cdot V_n$, should be computed using Equation 32.

$$\phi \cdot V_n = \phi \cdot (V_c + V_s) \qquad \text{Equation (32)}$$

In Equation 32, $\phi \cdot V_c$ is the contribution of the concrete to the design shear strength, and $\phi \cdot V_s$ is the contribution of the shear reinforcement, where employed, to the design shear strength. In Equation 32 ϕ = [0,85].

b) where support reaction, in direction of the applied shear, introduces compression into the end regions of the member, and no concentrated load occurs between the face of support and a distance from the support equal to d for girders, beams, joists, columns, slabs and footings, the sections in between should be permitted to be designed for the same factored shear, V_u, computed at d.

10.2.4.2 Contribution of concrete to beam-action design shear strength

At each critical location to be investigated, only the contribution of the concrete of the web of the beam should be taken into account, see Figure 46, and it should be computed using Equation 33 with ϕ = [0,85].

$$\phi \cdot V_c = \phi \cdot 2 \cdot \left[\frac{\sqrt{f'_c}}{6} \right] \cdot b_w \cdot d$$ Equation (33)

In Equation 33 for solid slabs and footings, b_w should be taken as the width of the section, b. See Figure 47.

10.2.4.3 Shear reinforcement

In girders, beams, and joists, the contribution to the design shear strength at the section of the shear reinforcement perpendicular to the axis of the element should be:

$$\phi \cdot V_s = \phi \cdot \left[\frac{A_v \cdot f_{ys} \cdot d}{s} \right]$$ Equation (34)

Where A_v corresponds to the area of shear reinforcement perpendicular to the axis of the element within a distance s, and f_{ys} is the yield strength of the steel of the shear reinforcement. In Equation 34 ϕ = [0,85].

The contribution of the shear reinforcement to the design shear strength should not be taken greater than:

$$\phi \cdot V_s \le \phi \cdot \left[\frac{2}{3} \cdot \sqrt{f'_c} \cdot b_w \cdot d \right] = 4 \cdot \phi \cdot V_c$$ Equation (35)

Shear reinforcement for solid slabs and footings is beyond the scope of these guidelines.

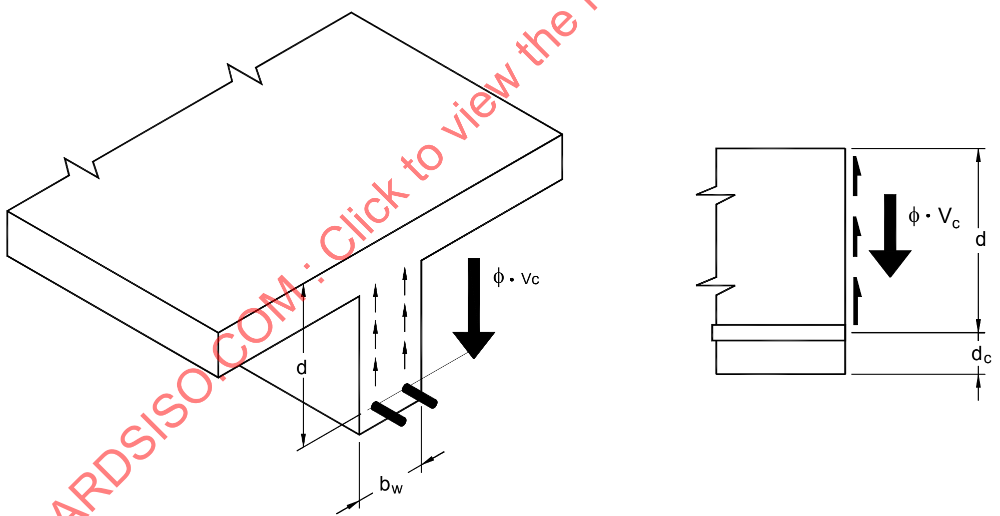


Figure 46 — Contribution of concrete to beam-action shear strength in girders, beams, and joists

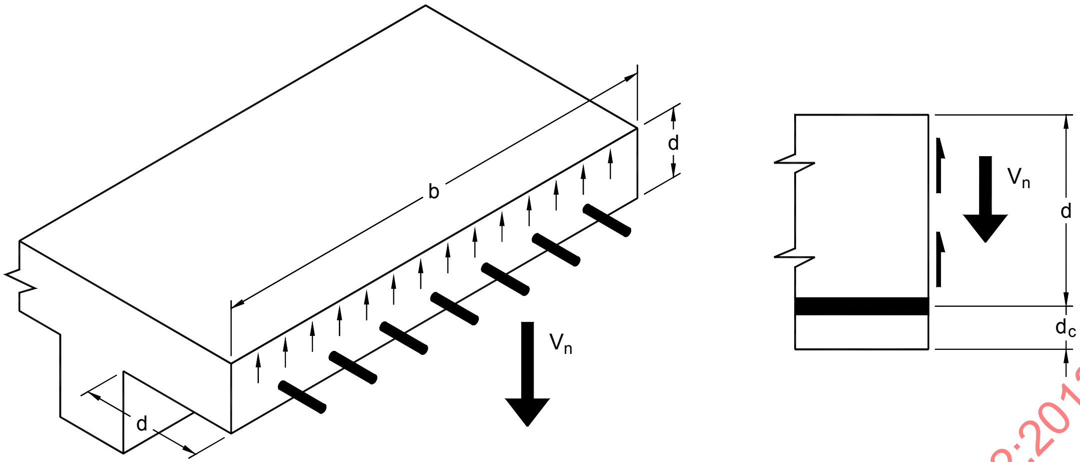


Figure 47 — Contribution of concrete to beam-action shear strength in solid slabs

10.2.4.4 Design of shear reinforcement

Shear reinforcement in girders, beams and joists, should be provided using stirrups perpendicular to the axis of the member with a maximum spacing s measured along the axis of the element:

a) where the factored shear V_u is less than one-half $\phi \cdot V_c$, it should be permitted to waive the use of shear reinforcement.

b) where the factored shear, V_u, exceeds one-half $\phi \cdot V_c$, and is less than $\phi \cdot V_c$, a minimum amount of shear reinforcement should be employed as specified by Equation 36. The maximum spacing s along the axis of the element should not exceed d/2, nor 600 mm. See Figure 48.

$$A_v = \frac{1}{16}\sqrt{f_c'}\,\frac{b_w \cdot s}{f_{ys}} \geq \frac{b_w \cdot s}{3 \cdot f_{ys}} \qquad \text{Equation (36)}$$

In Equation 36 A_v corresponds to the product of the area of the bar of the stirrup, A_b, multiplied by the number of vertical legs of the stirrup.

where the factored shear, V_u exceeds $\phi \cdot V_c$ the difference ($V_u - \phi \cdot V_c$) should be provided for by shear reinforcement, using Equation 32 , Equation 33 and Equation 34, and the following limitations should be employed (see Table 23):

The amount of shear reinforcement should not be less than that determined using Eq. (34).

If the value of $\phi \cdot V_s$, calculated using Equation 34 is less than ($2 \cdot \phi \cdot V_c$) the spacing limits of 10.2.4.4 b) should be employed.

If the value of $\phi \cdot V_s$, calculated using Equation 34 is greater than ($2 \cdot \phi \cdot V_c$) the spacing limits should be half of the values of 10.2.4.4 (b).

The value of $\phi \cdot V_s$, calculated using Equation 34 should not be taken greater than ($4 \cdot \phi \cdot V_c$).

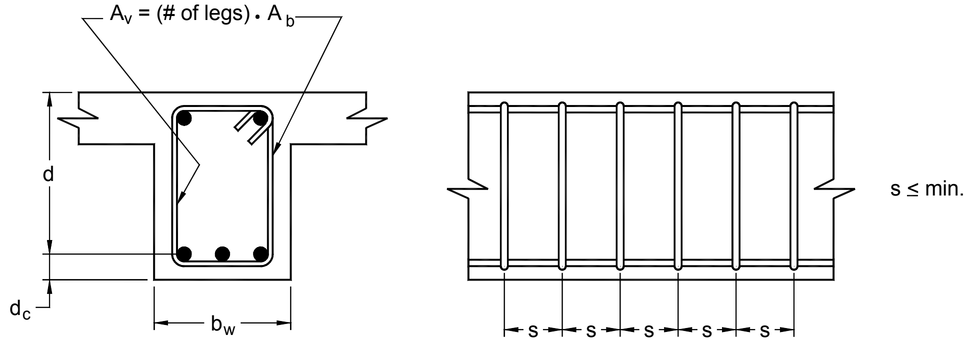


Figure 48 — Minimum shear reinforcement in girders, beams, and joists when ($\phi \cdot V_c/2 \le V_u < \phi \cdot V_c$)

Table 23 — Shear reinforcement in girders, beams, and joists, maximum spacing

Value of factored shear, V_u	Limiting value of ($\phi \cdot V_s$)	Required minimum area of shear reinforcement A_v within a distance s	Maximum spacing, s
$\frac{(\phi V_c)}{2} > V_u$		not required	
$(\phi V_c) > V_u \ge \frac{(\phi V_c)}{2}$		$A_v = \frac{1}{16}\sqrt{f'_c}\,\frac{b_w \cdot s}{f_{ys}} \ge \frac{b_w \cdot s}{3 \cdot f_{ys}}$	$s \le$ min. of $\begin{cases} d/2 \\ 600\ mm \end{cases}$
$V_u \ge (\phi V_c)$	$2 \cdot \phi \cdot V_c > \phi \cdot V_s$	$A_v = \frac{(V_u - \phi \cdot V_c) \cdot s}{\phi \cdot f_{ys} \cdot d}$	$s \le$ min. of $\begin{cases} d/2 \\ 600\ mm \\ 3 \cdot A_v \cdot f_{ys} / b_w \end{cases}$
	$4 \cdot \phi \cdot V_c > \phi \cdot V_s \ge 2 \cdot \phi \cdot V_c$	$A_v = \frac{(V_u - \phi \cdot V_c) \cdot s}{\phi \cdot f_{ys} \cdot d}$	$s \le$ min. of $\begin{cases} d/4 \\ 300\ mm \\ 3 \cdot A_v \cdot f_{ys} / b_w \end{cases}$
	$\phi \cdot V_s \ge 4 \cdot \phi \cdot V_c$	not permitted	

10.2.5 Two-way action shear (punching shear) in solid slabs and footings

10.2.5.1 General

The shear strength for two-way action shear, or punching-shear, should be investigated at edges of columns, concentrated loads, and supports, and at changes of thickness such as edges of capitals and drop panels.

10.2.5.2 Critical section definition for two-way action shear

The critical sections to be investigated should be located at a distance d/2 so that its perimeter b_0 is a minimum.

10.2.5.3 Two-way action shear design strength

The design shear strength should be the smallest of the values obtained from Equation 37, Equation 38 and Equation 39, with ϕ = [0,85]:

$$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[1 + \frac{2}{\beta_c}\right] \cdot \left[\frac{\sqrt{f'_c}}{6}\right] \cdot b_0 \cdot d$$ Equation (37)

where βc is the ratio of long side to short side of the column, concentrated load or reaction area,

$$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[2 + \frac{\alpha_s \cdot d}{b_0}\right] \cdot \left[\frac{\sqrt{f'_c}}{12}\right] \cdot b_0 \cdot d$$ Equation (38)

where αs is 40 for interior columns, 30 for edge columns, and 20 for corner columns, and:

$$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[\frac{\sqrt{f'_c}}{3}\right] \cdot b_0 \cdot d$$ Equation (39)

10.3 Decks

10.3.1 Types of deck systems

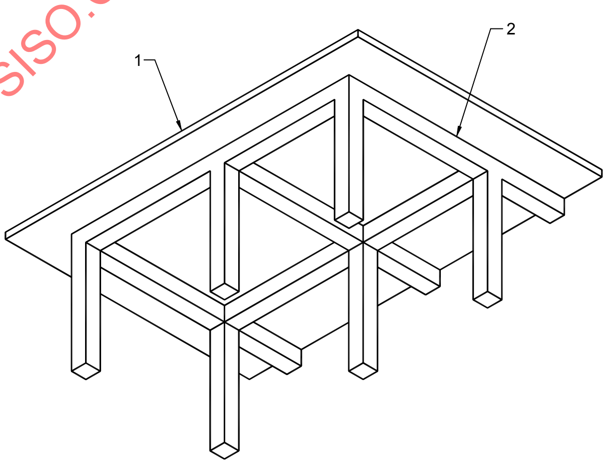
10.3.1.1 General

This clause describes the deck systems covered by the scope of these guidelines. The deck system employed by a bridge designed under these guidelines should be one of the systems covered or their permitted variations. The selection of an appropriate deck system should be performed studying several alternatives.

10.3.1.2 Slab-on-girder system

10.3.1.2.1 Description of the basic system

This system consists of a grid of girders in both main plan directions with a slab spanning the space between girders. These girders are located in the column lines or axis, spanning the distance between columns. A solid slab is supported by the girders. The slab can cantilever out of the edge beam. In this system the slab has a shallower depth than the girders. See Figure 49. For this system the guides for structural integrity of 10.3.3 should be complied with.



Key

1 slab

2 girder

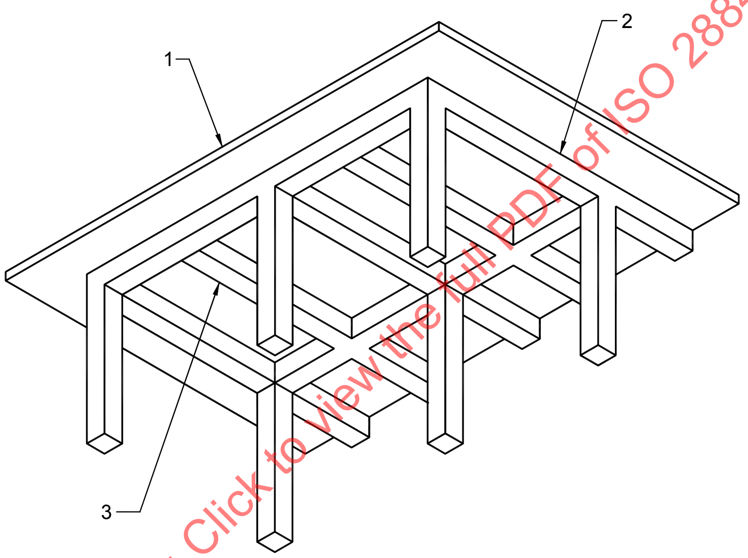
Figure 49 — Slab-on-girder deck system

10.3.1.2.2 Use of intermediate beams

One of the main variations of the system is the use of intermediate beams, supported on the girders. One or several beams can be employed per span. The intermediate beams can be of the same height of the girders, or shallower. These intermediate beams can be used in one direction, as shown in Figure 50, or in two directions, as shown in Figure 51. The use of too many intermediate beams will make the system gravitate to the joist system, described in 10.3.1.3.

10.3.1.2.3 Advantages of slab-on-girder system

For the slab-on-girder system each component has the appropriate minimum depth and width to comply with the strength or serviceability guides; therefore, having a relatively low selfweight. The system can accommodate spans of any size, can easily be adapted to any plan shape, and large perforations, ducts and shafts, can be located without major problems.



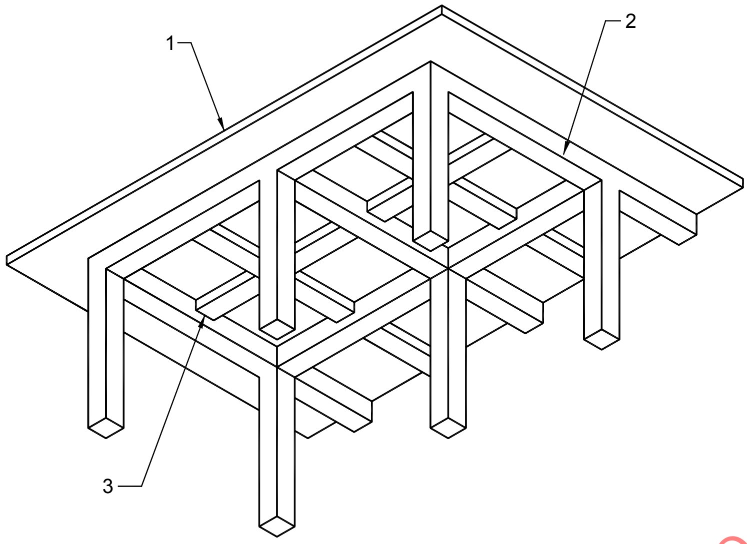
Key

1 slab

2 girder

3 intermediate one-direction beam

Figure 50 — Use of one-direction intermediate beams in the slab-on-girder deck system



Key

1 slab

2 girder

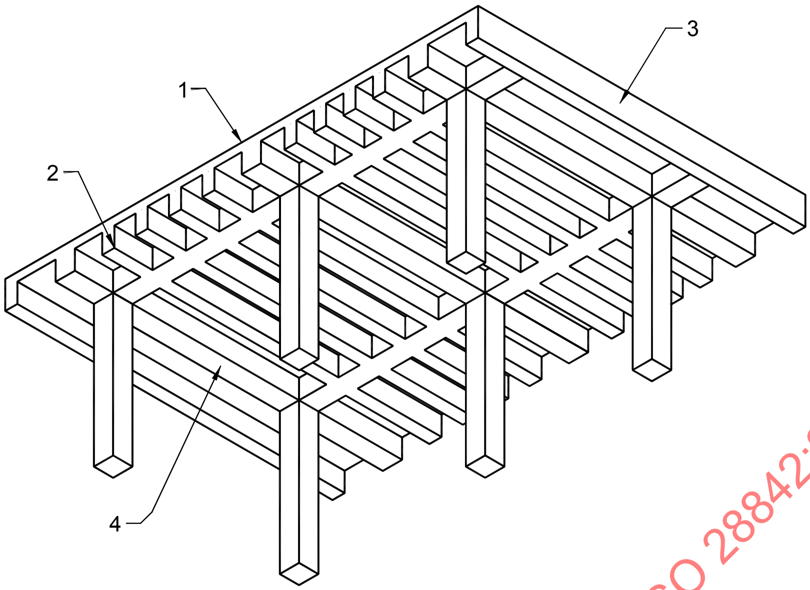
3 intermediate two-direction beam

Figure 51 — Use of two-direction intermediate beams in the slab-on-girder deck system

10.3.1.3 Joist systems

10.3.1.3.1 Description of the basic system

The joist system consists of a series of parallel ribs, or joists, supported by girders. The girders are located in the column lines or axis, spanning the distance between columns. A thin solid slab spans the space between joists. See Figure 52. For this system the guides for structural integrity of 10.3.3 should be complied with. The thin slab can not cantilever out of the edge joist. In this system the joists area usually of the same depth of the girders, but can have a shallower depth. The spacing between parallel joists, measured center-to-center of the joists, should not exceed [2,5] times the depth, h, of the joist, nor [1,2] m. The width of the web of the joist should be not less than [120] mm at the upper part. The minimum width should not be less than [100] mm. The clear depth of the joist should be not more than [5] times its average width. See Figure 53. The thin slab should comply with the minimum thickness guides of 10.3.5.2



Key

1 thin top slab

2 joist

3 edge joist

4 girder or beam

Figure 52 — Joist deck system

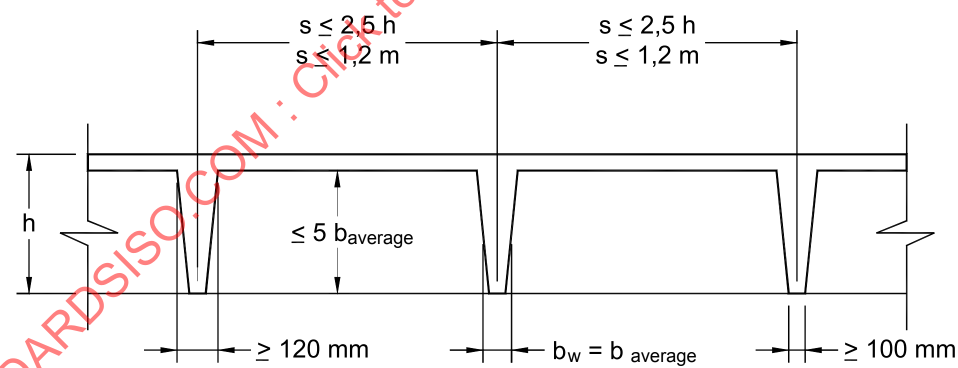


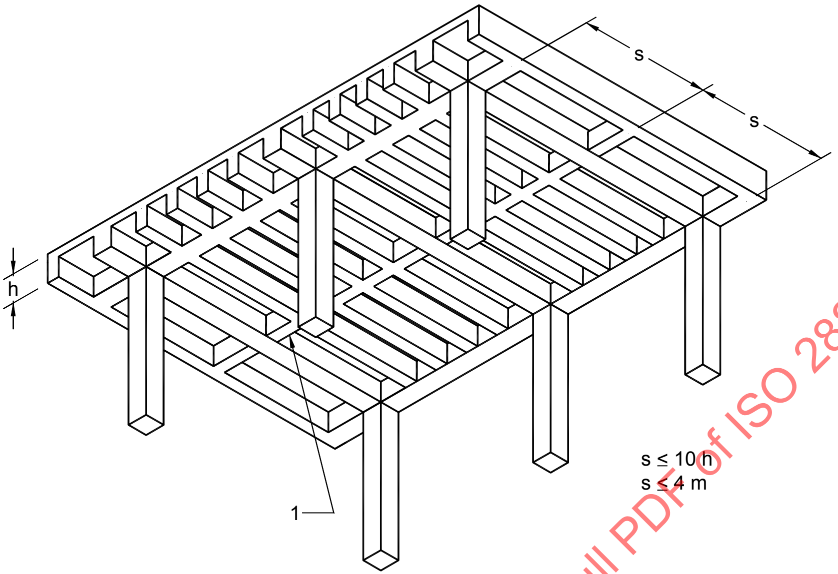
Figure 53 — Joist section dimension guides

10.3.1.3.2 Type of formwork

When the joists have the same depth of the girders, a flat formwork decking supported on shores, is employed. Joist shallower than the girders, may require more elaborate formwork. In order to create the voids, permanent and removable pans, or domes; of different shape and material are employed. Among those more popular are: permanent and removable wood pans, removable pans made out of metal, fiberglass, plastic or styrofoam, or permanent cement, cinder or clay filler blocks.

10.3.1.3.3 Distribution ribs

In joist systems that span in only one direction, in order to avoid that a concentrated load be carried by just one joist, transverse distribution ribs should be employed with spacings of no more than 10 times the total depth, h, of the joist, without exceeding 4 m. See Figure 54.



Key

1 distribution rib

Figure 54 — Distribution ribs

10.3.1.3.4 Advantages of joist systems

The joist system can accommodate medium to large spans, with relatively low selfweight. It is easy to locate small perforations, ducts and shafts. For heavy live loads or large permanent loads, the serviceability deflection guides can easily be accommodated because of the relatively high depth of the system. The clear spacing between joists is a tradeoff between a thinner top slab and requiring a larger amount of joists, thus allowing the designer great freedom in the choice of appropriate dimensions.

10.3.2 Criteria for the selection of the deck system

The structural designer should select a deck system from the systems covered by these guidelines, as presented in 10.3.1 Several alternatives should be studied and the final selection should be performed taking into account the merits of each of them in terms of:

a) the magnitude of the dead and live loads, and specially the selfweight of the system,

b) the geometry of the structural plan layout, specially the span lengths in both plan directions, and the ratio between them,

c) the presence of cantilevers, and their maximum span and direction,

d) the available material strengths, both for concrete and reinforcing steel,

e) the expected behavior of the slab system, and the adequacy to comply with the serviceability and deflection criteria,

f) the amount of materials -concrete, steel and formwork - required to build the deck system, taking into account that the deck system is probably responsible for the majority of the materials employed to build the structure,

g) local tradition in deck system construction plays an important role in the selection, and following it might simplify construction coordination,

h) workmanship training and proficiency should affect the selection, thus avoiding systems that require more training and proficiency than what the local workers can comply with, and

I) the relative cost of the alternatives, but the economical advantages should be pondered against the expected behavior and safety of the system.

10.3.3 Guides for structural integrity

10.3.3.1 General

The following should constitute minimum guides for improving the redundancy and ductility of the structure as a whole, in order for it to be able to be functional in the event of damage to a major supporting element or an abnormal loading event, by confining the damage to a relatively small area and maintaining overall stability.

10.3.3.2 Perimeter girders in slab-and-girder and joist systems

A ring of beams should be provided linking the perimeter columns and structural concrete walls of the structure, even when girders in slab-and-girder systems and joist systems are required for support of the slab or joists only in one direction in plan. These perimeter beams, or girders, should have a minimum area of continuous top and bottom longitudinal reinforcement, tied with closed stirrups. This reinforcement should always be lap-spliced using the minimum lap spliced length of 9.5.2.

10.3.3.3 Other beams and girders

All beams and girders, except the perimeter girders guide by 10.3.3.2 should have closed stirrups and a minimum area of continuous bottom longitudinal reinforcement, as required by 10.5.4.5 This reinforcement should always be lap-spliced, at or close to the supports, using the minimum lap splice length of 9.5.2.

10.3.3.4 Joists

In joists at least one bottom bar should be continuous over the support or should be spliced there using the minimum lap splice length of 9.5.2, and at not continuous supports should be terminated with a standard hook. See 10.5.4.5.

10.3.4 Slab one-way and two-way action and load path

10.3.4.1 General

The way the load is transported from the point of application to the supports in a slab system, depends on the geometrical plan dimensions of the slab panel, and on the stiffness of the supporting elements. For the purposes of this guideline, the way the loads are carried to the support should be classified into one-way and two-way action.

10.3.4.2 One-way action

A slab, solid or with joists, should be considered to work in one-way when:

a) has two opposing free edges, without vertical support, and has girders or beams along the full length of the edge, that provide vertical support in the other two opposing edges,

b) the slab panel has a rectangular plan shape, has girders, or beams, that provide vertical support in all edges, and the long slab span is greater than twice the short slab span, or

c) have joists, except the distribution ribs, in only one direction.

10.3.4.3 Two-way action

A slab, solid or with joists in both directions, should be considered to work in two-ways when the slab panel has a rectangular plan shape, and has girders, or beams along the full length of the edges, that provide vertical support in all edges, and the long slab span is less or equal than twice the short slab span.

10.3.4.4 Deck system load path

Based on the way the slab works, an approximate load path should be identified. The approximate load path should be used to assigning tributary load to all slab-supporting elements, and also in obtaining the preliminary dimensions of the slab and the supporting elements. The load path and the tributary load on the supporting elements should be verified, and corrected as needed, during the design and dimensioning stage of each of the structural elements.

10.3.5 Minimum allowable depth of the deck system elements

10.3.5.1 General

The following minimum allowable depth for elements of the deck system should be considered sufficient to meet the serviceability limit state, thus providing enough stiffness to the element to avoid undesirable deflections caused by the dead and live loads.

10.3.5.2 Solid one-way slabs supported by girders, beams or joists

The top thin slab should have a minimum thickness of l/20, (l = span length), but should not be less than [45] mm, when permanent concrete or clay filler blocks are employed, nor less than [50] mm in all other cases.

The slab minimum thickness, h, should not be less than the values given in Table 24, where the span length l, should be taken as the center-to-center distance between supports, except that when the span is less than 3 m, it should be permitted to take l as the clear span.

Table 24 — Minimum thickness, h, for one-way solid slabs

Continuity across the supports	Minimum thickness, h
Simply supported	$\frac{l}{20}$
One end continuous	$\frac{l}{24}$
Both ends continuous	$\frac{l}{28}$
Cantilever	$\frac{l}{10}$

10.3.5.3 Girders, beams and one-way joists supporting the slab

The girder, or beam, or one-way joist minimum thickness, h, should not be less than the values given in Table 25, where the span length l, should be taken as the center-to-center distance between supports, except that for joists when the span is less than 3 m, it should be permitted to take l as the clear span.

Table 25 — Minimum thickness, h, for girders, beams, and one-way joists

Continuity across the supports	Minimum thickness, h
Simply supported	$\frac{l}{16}$
One end continuous	$\frac{l}{18,5}$
Both ends continuous	$\frac{l}{21}$
Cantilever	$\frac{l}{8}$

10.3.5.4 Two-way slabs supported by girders or beams

The minimum allowable depth of two-way slabs, including two-way joist and waffle-on-beams systems, supported by girders, or beams in all edges of the panel, should be as given in Equation 40, and for solid slabs should be nor less than 120 mm for spans, l_n, greater than 3 m, and should not be less than 100 mm for spans, l_n, less or equal to 3 m.

$$h = \frac{l_n}{30 + 3 \cdot \beta} \qquad \text{Equation (40)}$$

Where the span length l_n, should be taken as the clear span in the long direction, measured face-to-face of the supporting beams, and β is the ratio of long clear span to short clear span of the slab panel. The procedure for design of two-way slabs-on-girders of this guideline guide that the supporting girders or beams should have a depth not less than three times the slab thickness (see 10.4.8.1).

10.3.6 Initial trial dimensions for the deck system

Initial trial dimensions should be defined for all the elements of the deck system. These initial trial dimensions should be assigned using the minimum depth or thickness, h, given in 10.3.5. For beam and girders the initial trial width, b_w, should be taken as one half of the depth, h, of the element but not less than 200 mm, and for joists should be defined using the minimum width dimensions given in 10.3.1.3.1.

These initial trial dimensions meet the serviceability limit state, and should be corrected as required by the strength limit state as the definite design proceeds. The selfweight calculated using the initial trial dimension should be corrected as modifications to the dimensions are introduced during the design process.

10.4 Solid slabs supported on girders, beams, or joists

10.4.1 General

The design of one-way and two-way solid slabs supported by girders, beams, or joists in their edges should be performed employing the guides of present 10.4. Guides for the top thin solid slab that span between joists are also included.

10.4.2 Design load definition

10.4.2.1 Loads to be included

The design load for solid slabs supported on girders, beams, or joists, should be established from the requirements of 8. The gravity loads that should be included in the design are:

Dead loads: selfweight of the structural element, flat non-structural elements, standing non-structural elements, and fixed equipment loads, if any.

Live loads.

Rain load and snow load, should be employed.

10.4.2.2 Dead load and live load

The values of q_d for dead load and q_l for live load should be in N/m^2. q_d should include the selfweight of the solid slab, at 25 N/m^2 per mm of thickness, and the weight of the flat and standing non-structural elements also in N/m^2. Live load should be determined as guide by 8.3. Snow loads should be included, if appropriate.

10.4.2.3 Factored design load

The value of the factored design load, q_u in N/m^2, should be the greater value obtained in the combinations 8.9.1.

10.4.3 Details of reinforcement

10.4.3.1 General

For the purposes of the present guidelines, the reinforcement of solid slabs-on-girders should be of the types described and should comply with the guides of 10.4.3.2 to 10.4.3.7.

10.4.3.2 Shrinkage and temperature reinforcement

10.4.3.2.1 Description

Reinforcement for shrinkage and temperature stresses normal to the flexural reinforcement of the slab should be provided in slabs-on-girders where the flexural reinforcement extends in one direction only.

10.4.3.2.2 Location

Shrinkage and temperature reinforcement should be located on top of the positive flexural reinforcement perpendicular to it, except in on roof slabs where it should be located under the negative flexural reinforcement perpendicular to it.

10.4.3.2.3 Minimum reinforcement area

Shrinkage and temperature reinforcement should comply with the minimum reinforcement steel ratio, ρ_t, of 9.6.2.1

10.4.3.2.4 Maximum and minimum reinforcement spacing

Shrinkage and temperature reinforcement should not be spaced further apart than guide by 9.4.14, nor should it be placed closer than guide by 9.4.9.

10.4.3.2.5 Reinforcement splicing

It should be permitted to lap-splice shrinkage and temperature reinforcement at any location. The splice length should comply with 9.5.2.

10.4.3.2.6 End anchorage of reinforcement

At edges of the slab, shrinkage and temperature reinforcement should end in a standard hook. It should be permitted to place the hook horizontally.

10.4.3.3 Positive flexural reinforcement

10.4.3.3.1 Description

Positive flexural reinforcement should be provided in the lower part of the slab section, as guide in present 10.4, and should comply with the general guides of 10.4.3.3, and the particular guides for each slab type as set forth in 10.4.4 to 10.4.8.

10.4.3.3.2 Location

Positive flexural reinforcement should be provided parallel to the short span in one-way solid slabs-on-girders, and in both directions in two-way-slabs. Positive flexural reinforcement should be located as close as concrete cover guides permit (see 9.4.4.1) to the bottom surface of the slab. In two-way systems the short span positive flexural reinforcement should be located under the long span positive flexural reinforcement. The amount of positive flexural reinforcement should be that required to resist the factored positive design moment at the section.

10.4.3.3.3 Minimum reinforcement area

Positive flexural reinforcement should have an area at least equal to the area guide by 9.6.2.2.

10.4.3.3.4 Maximum reinforcement area

Positive flexural reinforcement area should not exceed the values set forth in 9.6.2.3.

10.4.3.3.5 Maximum and minimum reinforcement spacing

Positive flexural reinforcement should not be spaced further apart than required by 9.4.13, nor should it be placed closer than permitted by 9.4.9.

10.4.3.3.6 Cut off points

It should be permitted to suspend at the locations indicated in 10.4.6 to 10.4.8 for each slab type, no more than one-half of the positive flexural reinforcement required to resist the corresponding factored design positive moment at mid-span.

10.4.3.3.7 Reinforcement splicing

It should be permitted to lap-splice the remaining positive flexural reinforcement between the cut-off point and the opposite face of the support.

10.4.3.3.8 Embedment at interior supports

Positive flexural reinforcement suspended at an interior support should be embedded by continuing it to the opposite face of the support.

10.4.3.3.9 End anchorage of reinforcement

Positive flexural reinforcement perpendicular to a discontinuous edge should extend to the edge of the slab and should end with a standard hook in the girder, beam, or structural concrete wall that provides support at the edge.

10.4.3.4 Negative flexural reinforcement

10.4.3.4.1 Description

Negative flexural reinforcement should be provided in the upper part of the slab section, at edges and supports, in the amounts and lengths required in present 10.4, and should comply with the general guides of 10.4.3.4, and the particular guides for each slab type as set forth in 10.4.4 to 10.4.8.

10.4.3.4.2 Location

Negative flexural reinforcement should be provided perpendicular to edge and intermediate supporting girders, beams, and structural concrete walls. Negative flexural reinforcement should be located as close as concrete cover guides permit (see 9.4.4.1) to the upper surface of the slab. In two-way systems the short span negative flexural reinforcement should be located above the long span negative flexural reinforcement. The amount of negative flexural reinforcement should be that required to resist the factored negative design moment at the section.

10.4.3.4.3 Minimum reinforcement area

Negative flexural reinforcement should have an area at least equal to the area guide by 9.6.2.2.

10.4.3.4.4 Maximum reinforcement area

Negative flexural reinforcement area should not exceed the values set forth in 9.6.2.3.

10.4.3.4.5 Maximum and minimum reinforcement spacing

Negative flexural reinforcement should not be spaced further apart than guide by 9.4.13, nor should it be placed closer than permitted by 9.4.9.

10.4.3.4.6 Cut off points

It should be permitted to suspend all the negative flexural reinforcement, except for cantilevers, at the locations indicated in 10.4.6 to 10.4.8 for each slab type. Where adjacent spans are unequal, negative flexural reinforcement cut-off points should be based on the guides for the longer span.

10.4.3.4.7 Reinforcement splicing

It should not be permitted to lap-splice negative flexural reinforcement between the cut-off point and the support.

10.4.3.4.8 End anchorage of reinforcement

Negative flexural reinforcement perpendicular to a discontinuous edge should be anchored with a standard hook into the edge girder, beam, or structural concrete wall that provides support at the edge, complying with the anchorage distance guide by 9.5.3. At the external edge of cantilevers negative flexural reinforcement perpendicular to the edge should end in a standard hook. It should be permitted to place the hook horizontally.

10.4.3.5 Shear reinforcement

The design procedures for slabs prescribed by these guidelines do not require the employment of transverse reinforcement in slabs. The procedures for design of transverse or shear reinforcement in slabs are beyond the scope of these guidelines.

10.4.3.6 Corner reinforcement

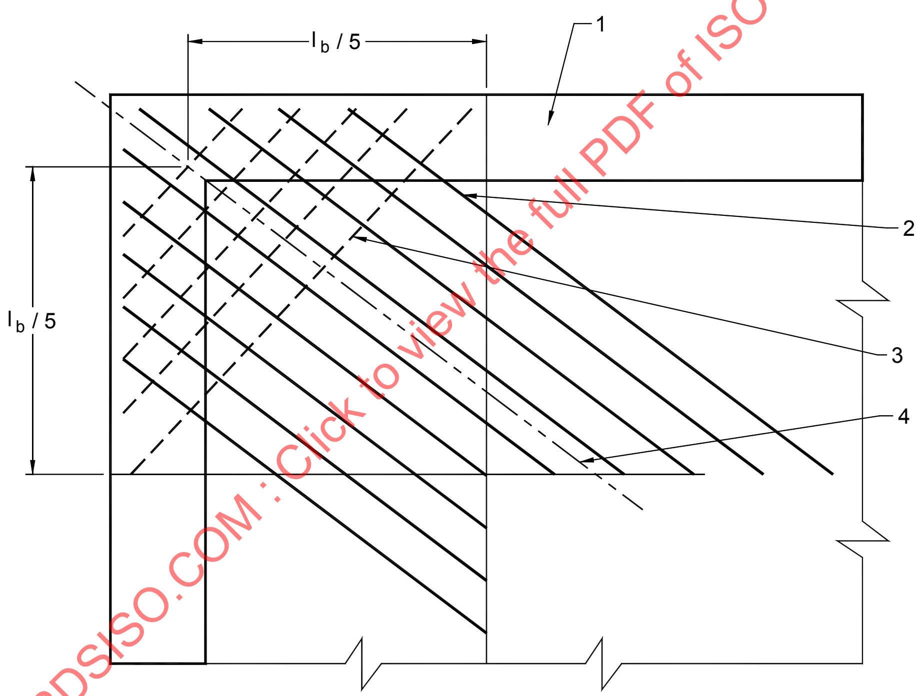
Special top and bottom slab reinforcement, in addition to other reinforcement, should be provided at exterior corners of the slab, different from cantilevers, for a distance equal to one-fifth of the longer clear span of the slab panel (see Figure 55), The amount of reinforcement, top and bottom, should be sufficient to resist a moment equal to the maximum positive factored design moment, per meter of width, in the slab panel, in accordance with 10.4.3.6.1 and 10.4.3.6.2.

10.4.3.6.1 Top corner reinforcement

Special reinforcement parallel to the diagonal of the panel should be placed in the top of the slab. This reinforcement should be anchored with a standard hook at the supporting girders, beams.

10.4.3.6.2 Bottom corner reinforcement

Special reinforcement perpendicular to the diagonal of the panel should be placed in the bottom of the slab. This reinforcement should be anchored with a standard hook at the supporting girders, beams.



Key

1 girder or beam

2 corner top reinforcement

3 corner bottom reinforcement

4 slab panel diagonal

Figure 55 — Special slab corner reinforcement

10.4.3.7 Practical considerations for the value of dc and d to employ in solid slabs

The determination of the distance from extreme tension fiber to centroid of tension reinforcement, [d_c], should include the appropriate concrete cover from 0, the bar diameters employed, and the existence of reinforcement in the perpendicular direction placed between the reinforcement under study and the concrete surface. It should be permitted to use the following values of d_c to compute d as $d = h - d_c$. For one-way slabs and for the reinforcement in the short direction in two way slabs, d_c = 40 mm for internal exposure and d_c = 60 mm for external exposure. For reinforcement in the long direction of two-way slabs, d_c = 55 mm for internal exposure and d_c = 75 mm for external exposure.

10.4.4 Top thin solid slab that spans between joists

10.4.4.1 Dimensional specifications

The thin solid slab that spans between joists should comply with the minimum thickness guides of 10.3.5.2. The top thin slab should not be permitted to cantilever out of the edge joist (see 10.3.1.3.1).

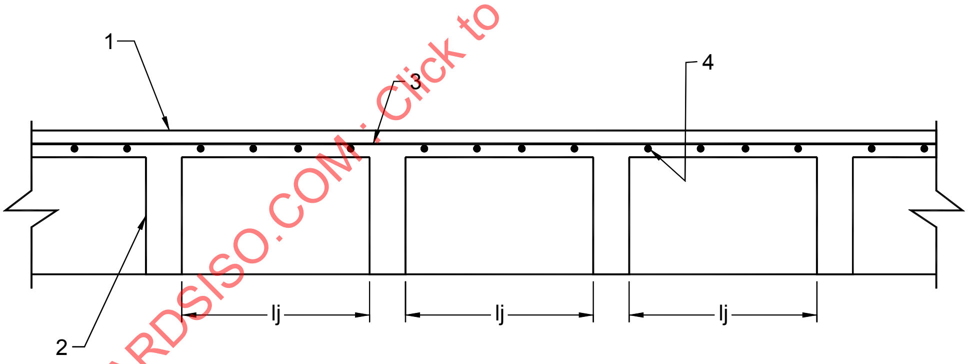
10.4.4.2 Factored flexural moment

The factored flexural moment, M_u, in N · m/m, for negative and positive flexural moment in the thin slab that spans between joist in joist construction should be calculated using Equation 41, where l_j is the clear spacing between joists in m and q_u should be employed in N/m^2. See Figure 56.

$$M_u^+ = M_u^- = \frac{q_u \cdot l_j^2}{12} \qquad \text{Equation (41)}$$

10.4.4.3 Reinforcement

The flexural reinforcement ratio ρ, perpendicular to the joist direction should be determined employing Equation 23 or Equation 24, with the value of M_u obtained from Equation 41, converted to N · mm (1 N ··m/m = 10^3 · N · mm/m), using d in mm as one-half the thickness of the thin slab, and b = 1 000 mm. ρ should be made greater or equal to the shrinkage and temperature ratio prescribed in 9.6.2.1. See Figure 56.The flexural reinforcing bar spacingshould meet the guides of 9.4.13. The reinforcement parallel to the joist direction should meet the guides of 10.4.3.2.1.



Key

1 top thin slab

2 slab

3 negative and positive flexural reinforcement

4 shrinkage and temperature reinforcement

Figure 56 — Reinforcement of the thin solid slab that span between joists

10.4.4.4 Shear strength verification

The factored shear V_u, in N/m, for the thin slab that span between joists in joist construction should be calculated using Equation 42, where l_j is the clear spacing between joists in m and q_u should be employed in N/m^2. See Figure 56.

$$V_u = \frac{q_u \cdot l_j}{2}$$ Equation (42)

The design shear strength $\phi \cdot V_n$, in N/m, should be calculated using Equation 33, with d as one-half the thickness of the thin slab, in mm, and b_w = b = 1 000 mm. Equation 31 should be complied with.

10.4.4.5 Calculation of the reactions on the joists

Factored uniformly distributed reaction on the supporting joists r_u, in N/m, should be the value obtained from Equation 43, where V_u is the factored shear from 10.4.4.4, in N/m, l is the center-to-center spacing of the joist, in m, and l_j is the clear spacing between joists, also in m.

$$r_u = \frac{2 \cdot V_u \cdot l}{l_j}$$ Equation (43)

10.4.5 Cantilevers of slabs supported on girders, beams

10.4.5.1 Dimensional specifications

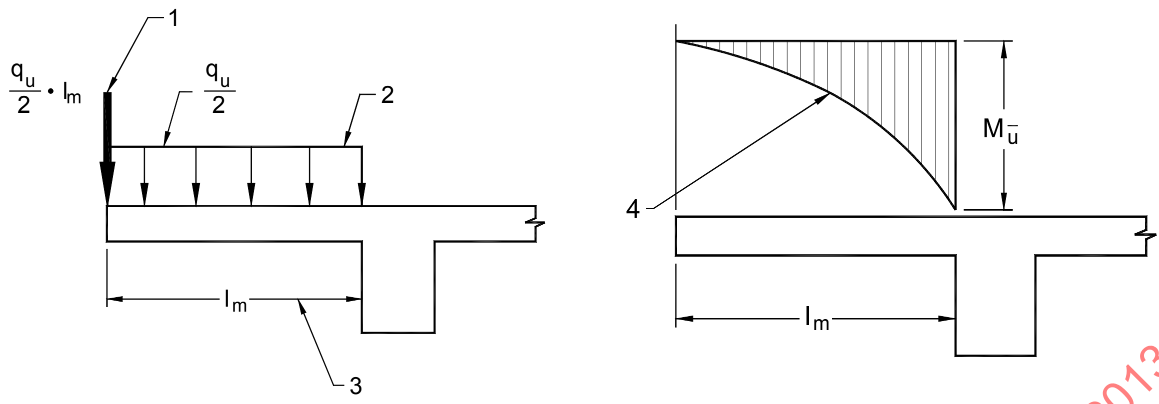
Solid slab cantilevers spanning out of the edge girder or beam should comply with the minimum thickness guides of 10.4.5.2. The cantilever span should not exceed the limits of 6.1. No openings for ducts or shafts should be permitted in the internal one-half span of the cantilever. It should be permitted for the slab to cantilever in two directions at corners, with the same limitations for single cantilevers. The thin top slab that spans between joists should not cantilever out of the edge joist.

10.4.5.2 Factored negative flexural moment

The factored negative flexural moment, M_u, for slab cantilevers that span out of the edge supporting girders, beams should be calculated supposing that one-half of the distributed factored load, q_u, acts as a concentrated load at the tip of the cantilever, and the other one-half acts as uniformly distributed load over the full span, using Equation 44, but it should not be less than the factored negative flexural moment of the first interior span at the exterior supporting girder, beam or structural concrete wall, nor less than one-third of the positive flexural moment, in the same direction, of the first interior span. See Figure 57.

$$M_u^- = \frac{3 \cdot q_u \cdot l_m^2}{4}$$ Equation (44)

where l_m should be the clear span of the cantilever in m, q_u should be employed in N/m^2, and M_u^- should be obtained in N·m/m.



Key

1 concentrated load

2 uniform load

3 cantilever clear span

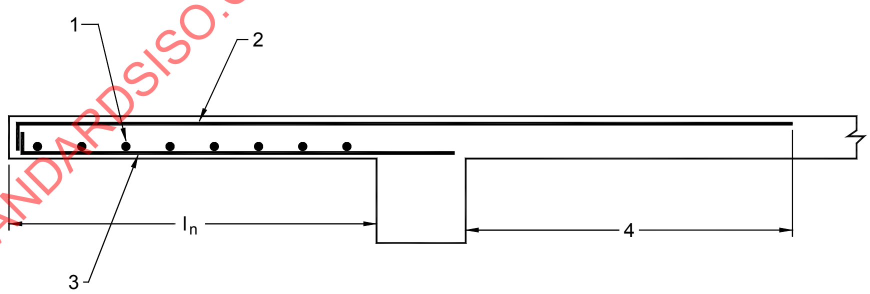
4 moment diagram

Figure 57 — Calculation of negative moment at slab cantilevers

10.4.5.3 Reinforcement

10.4.5.3.1 Negative flexural reinforcement

The negative flexural reinforcement ratio, ρ, in the direction of the cantilever should be determined employing Equation 23 or Equation 24, using the value of M_u obtained from Equation 44, converted to N mm/m (1 N m/m = 10^3 N mm/m), with the appropriate value of d in mm, and b = 1 000 mm. The negative flexural reinforcement should comply with 10.4.3.4. This reinforcement should be anchored in the first interior span not less than l_d for the reinforcing bar (see 9.5.1), nor the distance required for the negative reinforcement of the interior slab panel at the edge support. See Figure 58.



Key

1 shrinkage and temperature reinforcement

2 negative cantilever reinforcement

3 minimum positive reinforcement

4 distance as required for negative reinforcement of first interior span, but not less than ln for bar

Figure 58 — Reinforcement for slab cantilevers

10.4.5.3.2 Positive flexural reinforcement

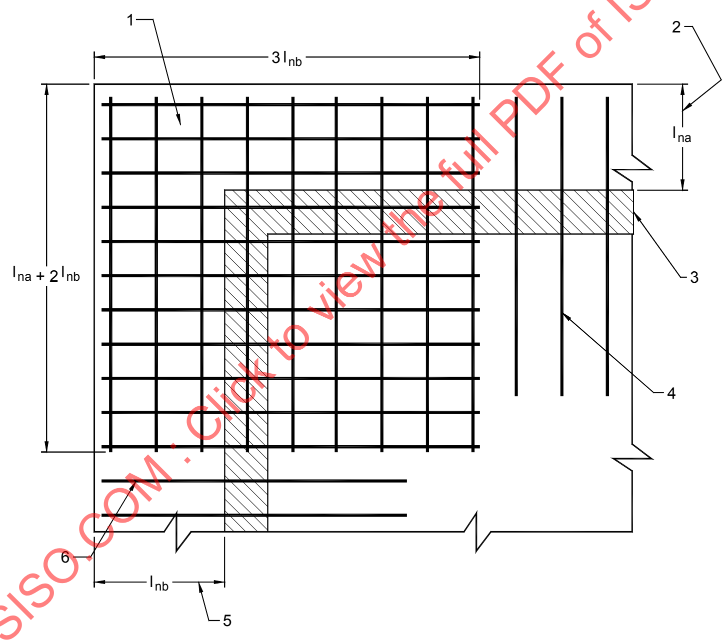
A minimum amount of positive flexural reinforcement with an area greater or equal to the shrinkage and temperature reinforcement, complying with the guides of 10.4.3.2 should be provided in the direction of the cantilever. See Figure 58.

10.4.5.3.3 Shrinkage and temperature reinforcement

Reinforcement parallel to the edge of the cantilever complying with 10.4.3.1 should be provided. See Figure 58.

10.4.5.3.4 Reinforcement of two-way cantilevers

At corners where the slab cantilevers in two-directions, the negative flexural reinforcement should be calculated for the larger span cantilever, using the guides of 10.4.5.3.1. This reinforcement should be placed in both directions (see Figure 59), for a distance measured from the corner equal to the cantilever clear span plus two times the larger cantilever span, but not less than the distance required for the negative flexural reinforcement of the first interior span plus the cantilever span. Reinforcement as guide by 10.4.5.3.2 should be placed in both directions.



Key

1 two-way cantilever negative reinforcement

2 smaller cantilever span

3 girder, beam or wall

4 one-way negative cantilever reinforcement

5 larger cantilever span

6 one-way negative cantilever reinforcement

Figure 59 — Negative flexural reinforcement in two-way slab cantilevers

10.4.5.4 Shear verification

The factored shear V_u, in N/m, at the support of cantilever slabs should be calculated using Equation 45, where l_m should be the clear span of the cantilever in m, and q_u should be employed in N/m^2.

$$V_u = q_u \cdot l_m \qquad \text{Equation (45)}$$

For two-way cantilevers the value of V_u should be taken as twice the value obtained from Equation 45 using the larger cantilever span.

The design shear strength $\phi \cdot V_n$, in N/m, should be calculated using Equation 33 with the appropriate value of d in mm, and b = 1 000 mm. Equation 31 should be complied with.

10.4.5.5 Calculation of the reactions on the supports

Uniformly distributed factored reaction on the support of the cantilever r_u, in N/m, should be the value obtained from Equation 46, where V_u is the factored shear from 10.4.5.4, in N/m, l is the span of the cantilever measured from the centerline of the supporting element, in m, and l_m is the clear span of the cantilever, also in m.

$$r_u = \frac{V_u \cdot l}{l_m} \qquad \text{Equation (46)}$$

Where two-way cantilevers exists it should be permitted in the calculation of the value of R_u to use Equation 26 employing the value of V_u obtained from Equation 45 for the larger cantilever span, without doubling it.

10.4.6 One-way one-span solid slabs spanning between girders or beams

10.4.6.1 Dimensional specifications

One-way one-span solid slabs should comply with the minimum thickness guides of 10.3.5.2. In addition to the appropriate guides of 10.4.6 these slabs should comply with the general dimensional guides set forth in 6.1, and the particular guides of 10.3.1.2 for slab-on-girder systems.

10.4.6.2 Factored flexural moment

The factored positive and negative flexural moment, M_u, in N · m/m, for one-span one-way slabs should be calculated using the Equations. given in Table 26.

Table 26 — Factored flexural moment for one-way, one-span slabs

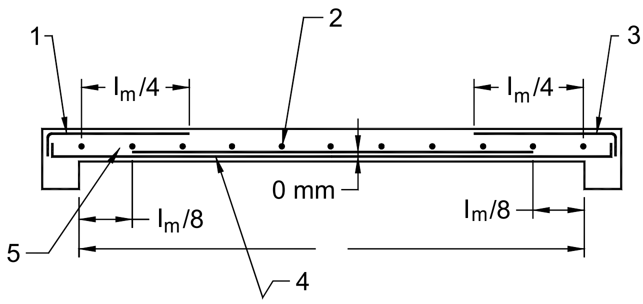
Positive moment: $M_u^+ = \frac{q_u \cdot l_m^2}{8}$	Equation (47)
Negative moment at supports: $M_u^- = \frac{q_u \cdot l_m^2}{24}$	Equation (48)

10.4.6.3 Longitudinal flexural reinforcement

10.4.6.3.1 Positive flexural reinforcement

The positive reinforcement ratio, ρ, in the direction of the span l_m, should be determined employing Equation 23 or Equation 24, with the value of M_u^+ obtained from Equation 47 converted to N · mm (1 N ·m/m = 10^3 · N ·

mm/m), using d in mm, and b = 1 000 mm. This reinforcement should comply with the guides of 10.4.3.3. In those cases in which the slab is cast monolithically with a supporting girder or beam, and the supporting element has a depth at least three times greater than the depth of the slab, it should be permitted to suspend up to one-half of the positive flexural reinforcement at a distance equal to $l_m/8$ measured from the internal face of the supports into the span. See Figure 60.



Key

1 negative flexural reinforcement

2 shrinkage and temperature reinforcement

3 negative flexural reinforcement

4 positive flexural reinforcement

5 positive flexural reinforcement suspension, only if slab built monolithically with support at least three times deeper than slab

Figure 60 — Reinforcement for one-span one-way slabs

10.4.6.3.2 Negative flexural reinforcement

The negative flexural reinforcement ratio, ρ, in the direction of the span l_m, should be determined employing Equation 23 or Equation 24, with the value of M_u^- obtained from Equation 48 converted to N · mm (1 N m/m = 10^3 · N ·mm/m), using d in mm, and b = 1 000 mm. This reinforcement should comply with the guides of 10.4.3.4. At a distance equal to lm/4 measured from the internal face of the supports into the span, all the negative flexural reinforcement should be permitted to be suspended. See Figure 60.

10.4.6.3.3 Shrinkage and temperature reinforcement

The reinforcement perpendicular to the span should meet the guides for shrinkage and temperature reinforcement of 10.4.3.2. See Figure 60.

10.4.6.4 Shear verification

The factored shear, V_u, in N/m, for the one-span one-way slab should be calculated at the face of the supports using Equation 49, where lm is the clear span in m and q_u should be employed in N/m^2. See Figure 60.

$$V_u = \frac{q_u \cdot l_m}{2} \qquad \text{Equation (49)}$$

The design shear strength, $\phi \cdot V_n$, in N/m, should be calculated using Equation 33, with d in mm, and b_w = b = 1 000 mm. Equation 31 should be complied with.

10.4.6.5 Calculation of the reactions on the supports

Uniformly distributed factored reaction on the supports of one-way one-span slabs, r_u, in N/m, should be the value obtained from Equation 49 plus the uniformly distributed reaction from any cantilever spanning from that support. In Equation 50 V_u is the factored shear from 10.4.6.4, in N/m, l is the center-to-center span of the slab, in m, and l_m is the clear span of the slab, also in m.

$$r_u = \frac{V_u \cdot l}{l_m}$$ Equation (50)

10.4.7 One-way solid slabs supported on girders or beams, with two or more spans

10.4.7.1 Dimensional specifications

One-way solid slabs with two or more spans should comply with the minimum thickness guides of 10.3.5.2. In addition to the appropriate guides of 10.4, slabs should comply with the general Dimensional specifications set forth in 6.1, and the particular guides of 10.3.1.2 for slab-on-girder systems.

The following restrictions should be in effect for slabs designed under 10.4.7:

a) there are two or more spans,

b) the spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 per cent (see 6.1),

c) loads are uniformly distributed,

d) unit live load, ql, does not exceeds three times unit dead load, q_d, and

e) for negative moment evaluation at internal supports, l_m should correspond to the largest of the neighbouring spans.

10.4.7.2 Factored flexural moment

The factored positive and negative flexural moment, M_u, in N ·m/m, for one-way slabs should be calculated using the Equation 51, Equation 52, Equation 53, Equation 54, Equation 55, and Equation 56 for slabs with two or more spans.

Positive moment

at end spans

$$M_u^+ = \frac{q_u \cdot l_n^2}{11}$$ Equation (51)

at interior spans:

$$M_u^+ = \frac{q_u \cdot l_n^2}{16}$$ Equation (52)

Negative moment at supports at interior face of external support

$$M_u^- = \frac{q_u \cdot l_n^2}{24}$$ Equation (53)

at exterior face of first internal support, only two spans

$$M_u^- = \frac{q_u \cdot l_n^2}{9}$$ Equation (54)

at faces of internal supports, more than two spans

$$M_u^- = \frac{q_u \cdot l_n^2}{10}$$ Equation (55)

at faces of all supports for slabs with spans not exceeding 3 m:

$$M_u^- = \frac{q_u \cdot l_n^2}{12}$$ Equation (56)

10.4.7.3 Longitudinal flexural reinforcement

10.4.7.3.1 Positive flexural reinforcement

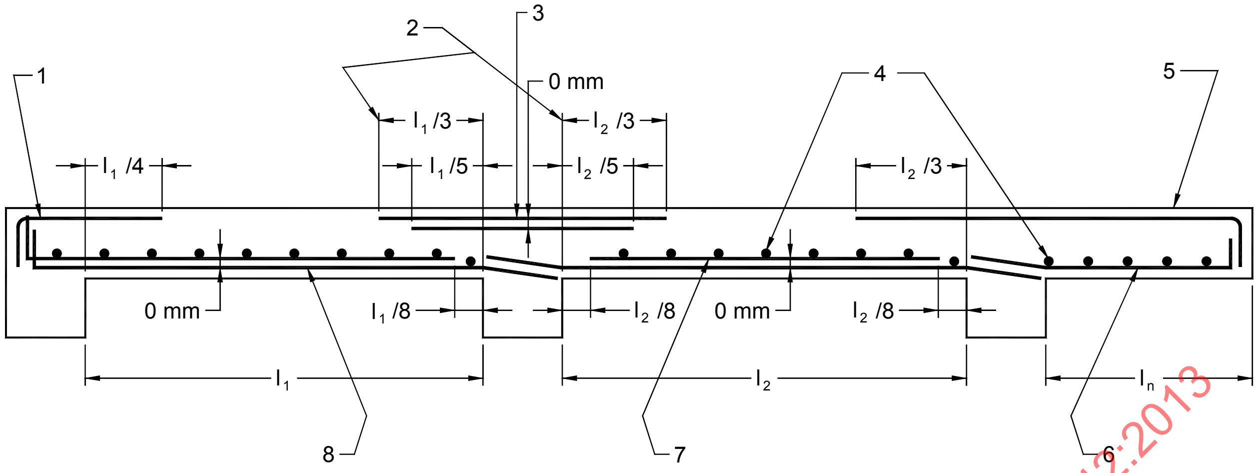
The positive reinforcement ratio, ρ, in the direction of the span l_m, should be determined employing Equation 23 or Equation 24, with the appropriate value of M_u^+ obtained from Equation 51 or Equation 52, converted to N · mm/m (1 N ·m/m = 10^3 ·N ·mm/m), using d in mm, and b = 1 000 mm. This reinforcement should comply with the guides of 10.4.3.3. At internal supports, at a distance equal to $l_m/8$ measured from the face of the supports into the span, up to one-half of the positive flexural reinforcement should be permitted to be suspended. See Figure 61.

10.4.7.3.2 Negative flexural reinforcement

The negative flexural reinforcement ratio, ρ, in the direction of the span, lm, should be determined employing Equation 23 or Equation 24, with the appropriate value of M_u^- obtained, from Equation 53, to Equation 56, converted to N ·mm/m (1 N ·m/m = 10^3 ·N ·mm/m), using d in mm, and b = 1 000 mm. This reinforcement should comply with the guides of 10.4.3.4. At internal supports, at a distance equal to $l_m/3$, where l_m should correspond to the largest of the neighboring spans, measured from the face of the support into the span, all the negative flexural reinforcement should be permitted to be suspended. At external supports, at a distance equal to $l_m/4$ measured from the internal face of the support into the span, all the negative flexural reinforcement should be permitted to be suspended. See Figure 61 and Figure 62.

10.4.7.3.3 Shrinkage and temperature reinforcement

The reinforcement perpendicular to the span should meet the guides for shrinkage and temperature reinforcement of 10.4.3.2. See Figure 61 and Figure 62.



Key

1 negative reinforcement at interior face of external support

2 negative reinforcement cut-off points should be based upon greater of the two neighboring spans

3 negative reinforcement interior support for two spans

4 shrinkage and temperature reinforcement

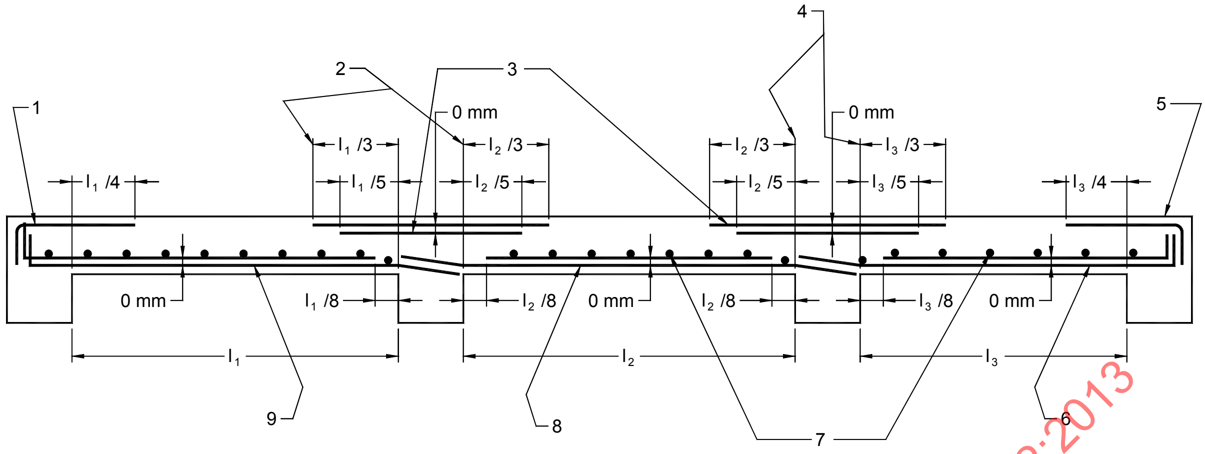
5 greater negative reinforcement from that required for the external support or for the cantilever

6 minimum cantilever positive reinforcement

7 positive reinforcement interior span

8 positive reinforcement end span

Figure 61 — Reinforcement for two-span one-way slabs supported by girders or beams



Key

1 negative reinforcement at interior face of external support

2 negative reinforcement cut-off points should be based upon greater of the two neighboring spans

3 negative reinforcement at faces of internal support more than two spans

4 negative reinforcement cut-off points should be based upon greater of the two neighboring spans

5 negative reinforcement at interior face of external support

6 positive reinforcement end span

7 shrinkage and temperature reinforcement

8 positive reinforcement interior span

9 positive reinforcement end span

Figure 62 — Reinforcement one-way slabs supported by girders or beams, or structural concrete walls, with three or more spans

10.4.7.4 Shear verification

The factored shear, V_u, in N/m, for the slab should be calculated at the faces of all supports using the Equation 57 and Equation 58, where l_m is the clear span in m and q_u should be employed in N/m^2. Figure 61 and Figure 62.

at exterior face of first interior support

$$V_u = 1{,}15 \cdot \frac{q_u \cdot l_m}{2} \qquad \text{Equation (57)}$$

at faces of all other supports

$$V_u = \frac{q_u \cdot l_m}{2} \qquad \text{Equation (58)}$$

The design shear strength, $\phi \cdot V_n$, in N/m, should be calculated using Equation 33, with d in mm, and b_w = b = 1 000 mm. Equation 31 should be complied with at all faces of supports.

10.4.7.5 Calculation of the reactions on the supports

Uniformly distributed factored reaction on the support contributed by any span of one-way slabs, r_u, in N/m, should be the value obtained from Equation 59, where V_u is the factored shear from 10.4.7.4, in N/m, l is the center-to-center span, in m, and l_m is the clear span, also in m.

$$r_u = \frac{V_u \cdot l}{l_m} \qquad \text{Equation (59)}$$

Total factored uniformly distributed reaction on the external supports should be equal to the value of the factored uniformly distributed reaction from the span, r_u, obtained from Equation 59 at the support, plus the uniformly distributed reaction of any cantilever spanning from that support. Total factored uniformly distributed reactions on internal supports should be the sum of the factored uniformly distributed reactions, r_u, obtained using Equation 59 for both neighbouring spans at that support.

10.4.8 Two-way solid slabs spanning between girders, or beams

10.4.8.1 Dimensional specifications

Two-way solid slabs having girders, beams in all edges should comply with the minimum thickness guides of 10.3.5.4. In addition to the appropriate guides of 10.4, two-way slabs should comply with the general Dimensional specifications set forth in 6.1, and the particular guides of 10.3.1.2 for slab-on-girder systems.

The following restrictions should be in effect for the use of the procedure of 10.4.8.1:

a) there are two or more spans,

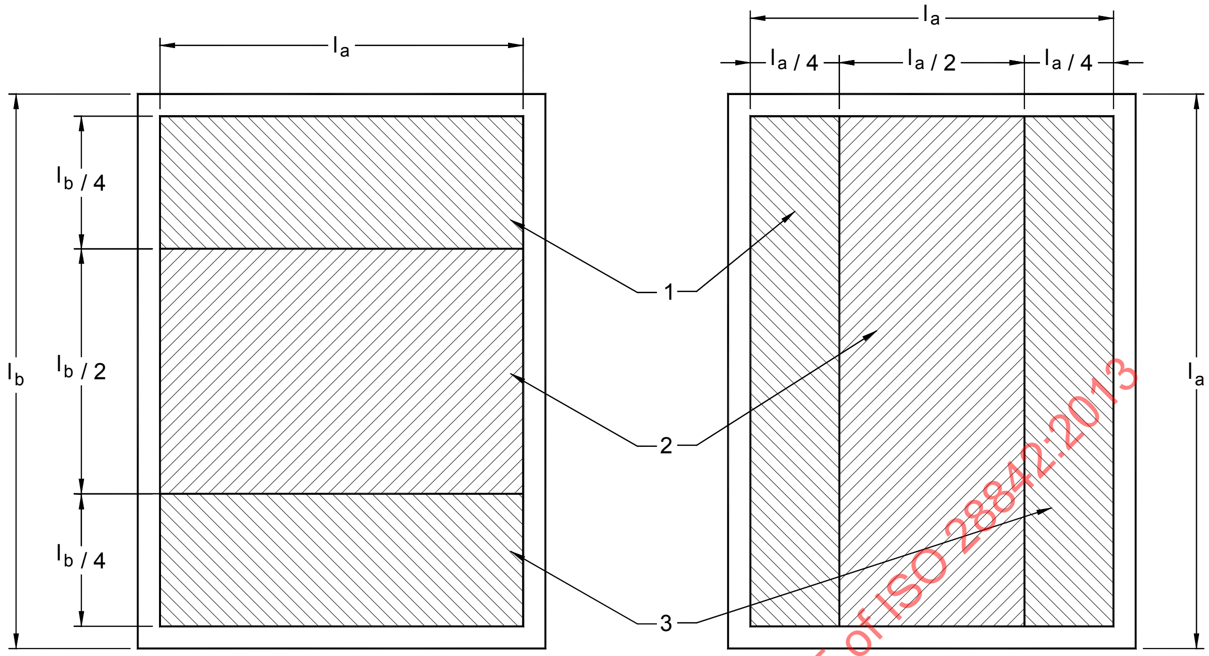
b) the spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 per cent of the larger (see 6.1),

c) the supporting girders or beams should be cast monolithically with the slab and should have a total depth not less than 3 times the slab thickness,

d) loads are uniformly distributed, and

e) unit live load, q_l, does not exceeds three times unit dead load, q_d.

The slab panel should be divided, in both directions, into central and border regions. The central region should be the central half of the panel, and the border regions should be two one-quarter regions adjacent on both sides of the central region. See Figure 63.



Key

1 border region

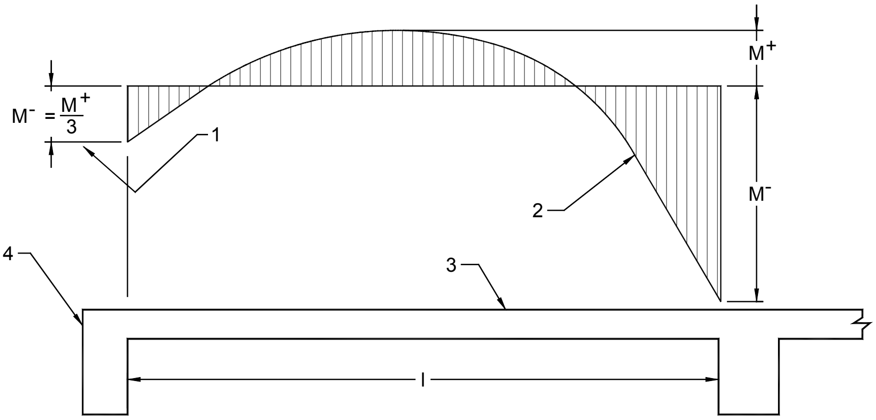
2 central region

3 border region

Figure 63 — Central and border regions for two-way slabs supported on girders or beams Factored flexural moment

The factored positive and negative moment, M_u, for two-way solid slabs should be calculated using the procedure set forth in 10.4.8.2. The negative and positive factored flexural moment for the central region of the panel, in each direction, should be calculated using the Eqs. given in Table 26 for central panels, in Table 27 for edge panels with the short span at the edge, in Table 27 for edge panels with the long span at the edge, and in Table 28 for corner panels. In each table the values of the factored flexural moments should be obtained for the appropriate ratio, β, of long clear span, l_b, to short clear span, l_a, and the corresponding edge continuity conditions.

The negative moment at discontinuous edges should be one-third of the positive moment in the same direction. See Figure 64.



Key

1 value of negative moment at discontinuous edge

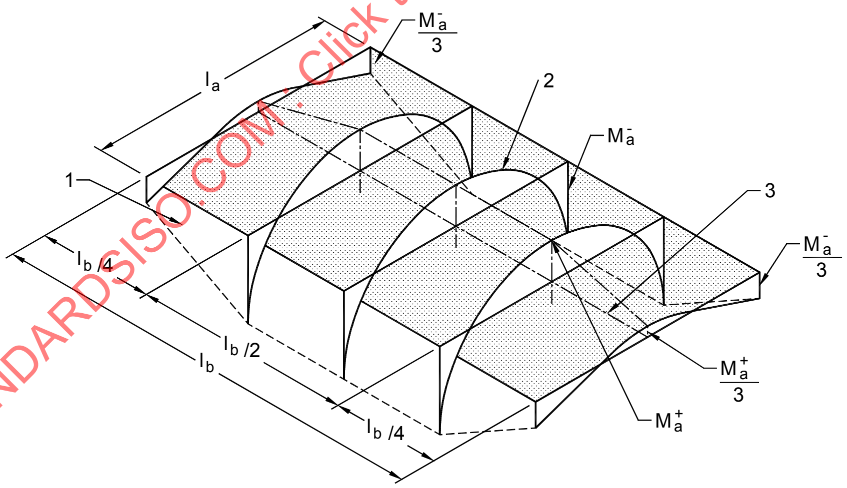
2 moment diagram

3 end slab span

4 discontinuous edge

Figure 64 — Negative moment at discontinuous edges of two-way solid slabs-on-girders

It should be permitted to decrease the moment strength values at the edge of the central regions, to one-third of this value at the edge of the panel, as shown in Figure 65 for moments in the short directions, and in Figure 66 for moments in the long direction.



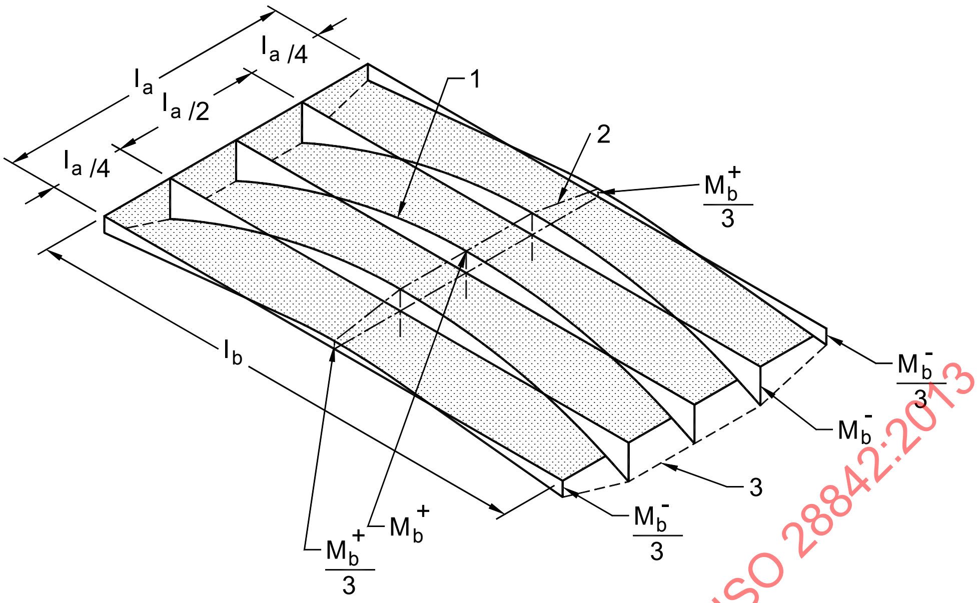
Key

1 variation of M'a along lb

2 variation of M'a along la

3 variation of M_a^+ along lb

Figure 65 — Variation of moment M_a across the width of critical sections for design, for two-way slabs supported on girders, beams



Key

1 variation of Mb along lb

2 variation of M_b^+ along la

3 variation of M_b^- along la

Figure 66 — Variation of moment, M_b, across the width of critical sections for design, for two-way slabs supported on girders or beams

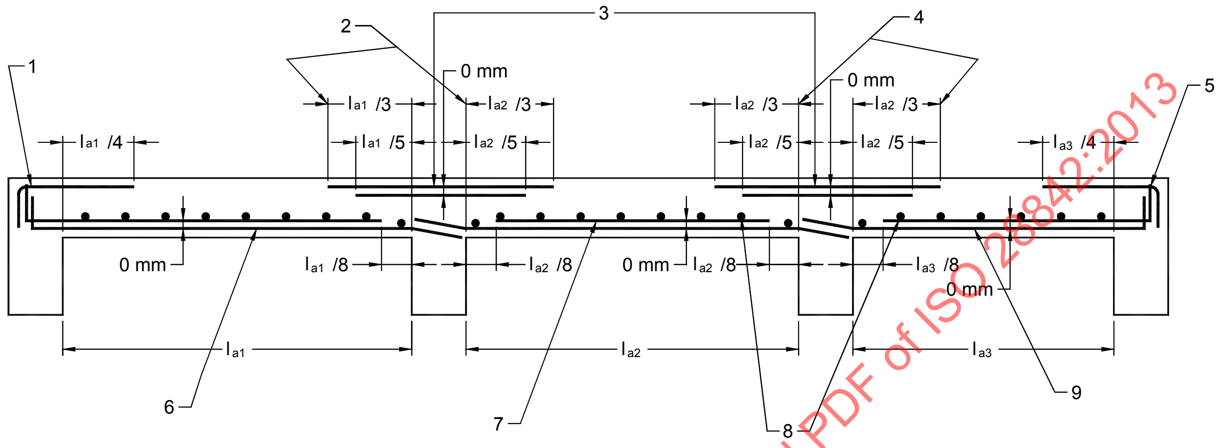
10.4.8.2 Longitudinal flexural reinforcement

10.4.8.2.1 Positive flexural reinforcement

In the central region of the slab panel the positive flexural reinforcement should be determined based on the values of the factored positive flexural moment obtained from the appropriate Eqs. of Table 27 to Table 30. The positive flexural steel ratio, ρ, for reinforcement parallel to the short span l_a, or the long span l_b, should be determined using Equation 23 or Equation 24 for the corresponding value of M_a^+ or M_b^+ in N · m/m. M_a^+ and M_b^+ should be converted to N mm/m (1 N · m/m = 10^3·· N · mm/m), using d in mm and b = 1 000 mm. All guides for positive flexural reinforcement of 10.4.3.3 should be met. At a distance equal to l_a/8 or l_b/8 measured from the face of any interior support, it should be permitted to suspend up to one-half of the positive flexural reinforcement required at the center of the corresponding span. No suspension of positive flexural reinforcement perpendicular to a discontinuous edge should be permitted. It should be permitted to decrease gradually the positive flexural reinforcement required at the central region from the edge of the central regions, to one-third of this value at the edge of the panel, but not below the value required for shrinkage and temperature. See Figure 67.

10.4.8.2.2 Negative flexural reinforcement

At the supporting edges of the central region of the slab panel the negative flexural reinforcement should be determined based on the values of the factored negative flexural moment obtained from the appropriate Eqs. of Table 27 to Table 30. The negative flexural steel ratio, ρ, for reinforcement parallel to the short span l_a, or the long span l_b, should be determined using Equation 23 or Equation 24 for the corresponding value of M_a^- or M_b^- in N · m/m. M_a^- and M_b^- should be converted to N · mm/m (1 N · m/m = 10^3 · N · mm/m), using d in mm and b = 1 000 mm. All guides for negative flexural reinforcement of 10.4.3.4 should be met. At a distance equal to $l_a/5$ or $l_b/5$ measured from the face of any interior support, it should be permitted to suspend up to one-half of the negative flexural reinforcement required at the support, and a distance equal to $l_a/3$ or $l_b/3$ measured from the face of any interior support, it should be permitted to suspend all the negative flexural reinforcement required at the support of the corresponding span. It should be permitted to decrease gradually the negative flexural reinforcement required at the central region from the edge of the central regions, to one-third of this value at the edge of the panel, but not below the value required for shrinkage and temperature. See Figure 67.



Key

1 negative reinforcement at interior face of external support without continuity

2 negative reinforcement cut-off points should be based upon greater of the two neighboring spans

3 negative reinforcement at faces of internal supports more than two spans

4 negative reinforcement cut-off points should be based upon greater of the two neighboring spans

5 negative reinforcement at interior face of external support without conting

6 positive reinforcement end span

7 positive reinforcement interior span

8 positive reinforcement in the other direction (negative reinforcement not shown)

9 positive reinforcement end span

Figure 67 — Reinforcement for two-way slabs supported by girders, beams

10.4.8.3 Shear verification

The factored shear, V_u, of the slab at the faces of the supporting elements should be determined employing the values of the load fractions, α_a and α_b, travelling in the short and long directions respectively, as given in Table 27 to Table 30 for the corresponding panel edge conditions and panel span ratio, β. See Figure 68. The factored shear should not be less than the factored shear caused by factored design load, qu (in N/m^2) acting on a tributary area bounded by 45° lines drawn from the corner of the panel and the centerline of the panel parallel to the long span. See Figure 69. The factored shear, V_u, in N/m, should not be less than the value obtained from Equation 60 at the short span supporting element.

$$V_u = \frac{\alpha_a \cdot q_u \cdot l_b}{2} \geq \frac{q_u \cdot l_a}{4} \qquad \text{Equation (60)}$$

and from Equation 61 at the long span supporting element.

$$V_u = \frac{\alpha_b \cdot q_u \cdot l_a}{2} \geq q_u \cdot \left[\frac{l_a}{2} - \frac{l_a^2}{4 \cdot l_b} \right]$$ Equation (61)

The design shear strength, $\phi \cdot V_n$, should be calculated employing Equation 33 with b_w = b = 1 000 mm. Equation 31 should be complied with. This should be accomplished by using an effective depth d, in mm, for the slab greater or equal to the largest value obtained from Equation 62, Equation 63, and Equation 64.

$$d \geq \frac{3 \cdot q_u \cdot \alpha_a \cdot l_a}{\phi \cdot \sqrt{f_c'}}$$ Equation (62)

$$d \geq \frac{3 \cdot q_u \cdot \alpha_b \cdot l_b}{\phi \cdot \sqrt{f_c'}}$$ Equation (63)

$$d \geq \frac{3 \cdot q_u \cdot l_a}{2 \cdot \phi \cdot \sqrt{f_c'}}$$ Equation (64)

In Equation 62 to Equation 64, q_u should be used in N/m^2, l_a and l_b in m, f_c' in MPa, and ϕ = [0,85].

10.4.8.4 Calculation of the reactions on the supports

Uniformly distributed factored reaction on the support contributed by any panel of two-way slabs, r_u in N/m, in the short direction should be the value obtained from Equation 65 and in the long direction the value obtained from Equation 66.

$$r_u = \frac{V_u \cdot l}{l_a}$$ Equation (65)

$$r_u = \frac{V_u \cdot l}{l_b}$$ Equation (66)

In Equation 65 or Equation 66, V_u is the corresponding factored shear from Equation 60 or Equation 61, in N/m, l is |the center-to-center span in that direction, in m, and l_a and l_b are the corresponding clear spans, also in m.

Total factored uniformly distributed reaction on the external supports of edge panels should be equal to the value of the factored uniformly distributed reaction from the panel, r_u, at the edge support, obtained from Equation 65 or Equation 66 plus the uniformly distributed reaction of any cantilever spanning from that support. Total factored uniformly distributed reactions on internal supports should be the sum of the factored uniformly distributed reactions, r_u, obtained using either Equation 65 or Equation 66, as appropriate, for both neighboring spans at that support.

10.5 Girders, beams and joists

10.5.1 General

The design of girders, beams and joists should be performed employing the requirements of present 10.5. The guides apply to isolated beams, to girders, beams and joists that are part of a deck system.

10.5.2 Design load definition

10.5.2.1 Loads to be included

The design load for girders, beams, and joists should be established from the guides of 8. The gravity loads that should be included in the design of the element should be divided in tributary loads from other structural elements supported by the element being designed, and loads applied directly on the element being designed. Adjustments for the effects of lateral loads should be performed employing the guides of 13.

10.5.2.1.1 Tributary loads

The reactions from other structural elements supported by the girder, beam or joist should consider:

a) Dead loads: including the selfweight of the supported structural elements, the loads caused by flat and standing non-structural elements and the loads from any fixed equipment carried by these supported elements, and

b) Live loads applied on the supported elements.

10.5.2.1.2 Loads carried directly by the beam, girder or joist

Loads carried directly by the beam, girder or joist should consider:

a) Dead loads: including selfweight of the structural element, and the flat and standing non-structural elements, and fixed equipment loads, applied directly on the element, and

b) Live loads applied directly to the element being designed.

10.5.2.2 Factored design load

10.5.2.2.1 Factored design load for loads carried directly by the element:

a) For uniformly distributed loads carried directly by the girder, beam, or joist, the value of the uniformly distributed factored design load, w_u in N/m, should be the greater value obtained combining w_d and w_l using Table 14 should also be investigated, choosing the greatest value of all nine combinations.

b) For all concentrated loads carried directly by the girder, beam, or joist, the value of any concentrated factored design load, p_u, in N, should be the greater value obtained combining p_d and p_l using Table 14, for each concentrated load locations in the girder, beam, or joist span.

10.5.2.2.2 Factored reactions from supported structural elements:

a) The largest factored uniformly distributed reaction from all tributary structural elements, r_u, in N/m, should be obtained.

b) For concentrated loads, the largest factored concentrated reactions from all the supported structural elements, R_u, in N, should be obtained for all concentrated load locations in the girder, beam, or joist span.

10.5.2.2.3 Total factored design load:

a) The total factored uniformly distributed load W_u, in N/m, should be the sum of the values obtained for factored uniformly distributed loads, w_u, from 10.5.2.2.1 and reactions, r_u, from 10.5.2.2.2.

b) For all concentrated load locations in the girder, beam, or joist span the total factored concentrated load P_u, in N, should be the sum of the values obtained for factored concentrated loads, p_u, from 10.5.2.2.1 and reactions, R_u, from 10.5.2.2.2.

10.5.3 Details of reinforcement

10.5.3.1 General

For the purposes of the present guidelines, the reinforcement of girders, beams, and joists, should be of the types described and should comply with the guides of 10.5.3.2 to 10.5.3.9.

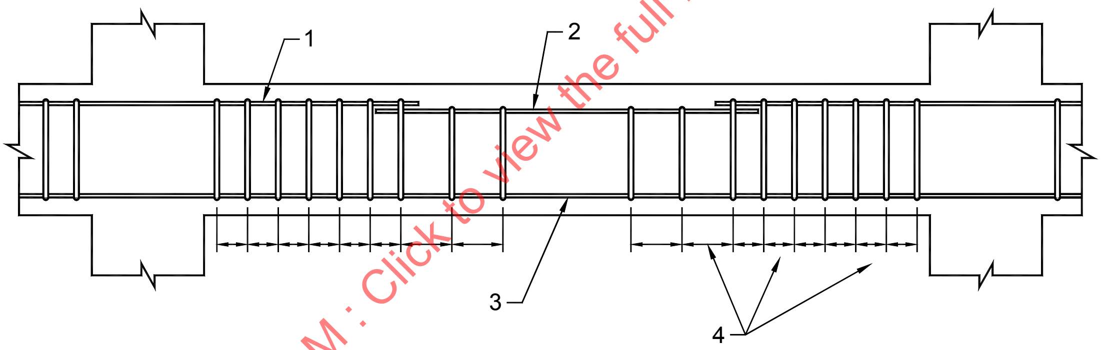
10.5.3.2 Transverse reinforcement

10.5.3.2.1 Description

Transverse reinforcement for girders, beams, and joists, should consist of stirrups that surround the longitudinal reinforcement and are placed perpendicular to the longitudinal axis of the element at varying intervals along the axis. The stirrup should consist of single or multiple vertical legs. Each vertical leg should engage a longitudinal bar either by bending around it when the stirrup continues, or by the use of a standard stirrup hook (see 9.4.6) surrounding the longitudinal bar at the end of the stirrup. See Figure 69. Under the present guidelines all stirrups in girders and beams should be closed stirrups with 135° hooks, as shown in Figure 69 (a). In joists it should be permitted to employ all the stirrup types shown in Figure 69.

10.5.3.2.2 Location

Stirrup spacing intervals, s, shall comply with 10.2.4.4 (Figure 68).



Key

1 negative flexural reinforcement

2 stirrup supporting bars

3 positive flexural reinforcement

4 stirrup separations

Figure 68 — Typical stirrups spacing along the girder, beam or joist

10.5.3.2.3 Minimum transverse reinforcement area

The minimum area of shear reinforcement, A_v, within a distance s, should comply with the guides of 10.2.4.4. A_v corresponds to the product of the area of the bar of the stirrup, A_b, multiplied by the number of vertical legs of the stirrup.

10.5.3.2.4 Maximum and minimum spacing of stirrups

Stirrups should not be spaced further apart than guide by 10.2.4.4, nor should it be placed closer than guide by 9.4.9.

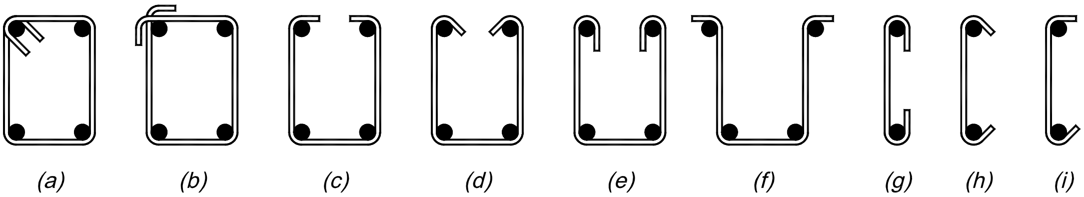


Figure 69 — Typical stirrup shapes

10.5.3.2.5 Stirrup leg splicing

It should not be permitted to lap-splice bars that are part of stirrups.

10.5.3.2.6 Hanger reinforcement

Where beams are supported by other girders or beams of similar height, special hanger reinforcement stirrups should be provided as guide in 10.5.4.5.4.

10.5.3.2.7 Support of stirrups

Stirrups should be attached and anchored in the upper part of the section to longitudinal negative supporting bars in order to avoid that the stirrups fall during casting of the concrete. See 10.5.3.4.10.

10.5.3.3 Positive flexural reinforcement

10.5.3.3.1 Description

Positive flexural reinforcement should be provided in the lower part of the girder, beam or joist section, as required in present 10.5, and should comply with the general guides of 10.5.3.3, and the particular guides for each element type as set forth in 10.5.4 or 11.1.

10.5.3.3.2 Location

Positive flexural reinforcement should be provided longitudinally in the girder, beam or joist. Positive flexural reinforcement should be located as close as concrete cover guides permit (see 9.4.4.1) to the bottom surface of the girder, beams or joist. The amount of positive flexural reinforcement should be that required to resist the factored positive design moment at the section. Where girders, beams or joists give support to other girders, beams, or joists, the positive flexural reinforcement of the supported element should be placed on top of the positive flexural reinforcement of the supporting element.

10.5.3.3.3 Minimum reinforcement area

Positive flexural reinforcement should have an area at least equal to the area guide by 9.6.3.1. The minimum number of bars guide by 10.5.3.6 should be complied with.

10.5.3.3.4 Maximum reinforcement area

Positive flexural reinforcement area should not exceed the values set forth in 9.6.3.2.

10.5.3.3.5 Minimum and maximum reinforcement spacing

Positive flexural reinforcement should not be spaced closer than guide by 9.4.9 and 10.5.3.5. The maximum reinforcement spacingshould comply with 10.5.3.6. When two or more layers of positive reinforcement are employed the layers should not be placed closer than permitted by 9.4.10.

10.5.3.3.6 Cut off points

It should be permitted to suspend, at the locations indicated in 10.5.4.5 or 11.1.5, no more than one-half of the positive flexural reinforcement required to resist the corresponding factored design positive moment at mid-span.

10.5.3.3.7 Reinforcement splicing

It should be permitted to lap-splice the remaining positive flexural reinforcement from 10.5.3.3.6 between the cut-off point and the opposite face of the support.

10.5.3.3.8 Embedment at interior supports

Positive flexural reinforcement suspended at an interior support should be embedded by continuing it to the opposite face of the support, plus the distance required to comply with the lap splice guide of 9.5.2.

10.5.3.3.9 End anchorage of reinforcement

Positive flexural reinforcement at the end of the girder, beam or joist should extend to the edge and should end with a standard hook.

10.5.3.3.10 Positive flexural reinforcement acting in compression

Positive flexural reinforcement acting in compression should be surrounded with stirrups or ties that comply with 9.7.3.

10.5.3.3.11 Minimum diameter of longitudinal reinforcement

Longitudinal bars of beams and girders should have a nominal diameter, db, of 12 mm or more.

10.5.3.4 Negative flexural reinforcement

10.5.3.4.1 Description

Negative flexural reinforcement should be provided in the upper part of the girder, beam or joist section, at edges and supports, in the amounts and lengths guide in present 10.5, and should comply with the general guides of 10.5.3.4, and the particular guides of 10.5.4 or 11.1.

10.5.3.4.2 Location

Negative flexural reinforcement should be provided at edge and intermediate supports. Negative flexural reinforcement should be located as close as concrete cover guides permit (see 9.4.4.1) to the upper surface of the girder, beams or joist. At supports where girders or beams intersect the negative flexural reinforcement of the elements with the larger span should be located above the negative flexural reinforcement of the intersecting element with the shortest span. The amount of negative flexural reinforcement should be that required to resist the factored negative design moment at the section.

10.5.3.4.3 Minimum reinforcement area

Negative flexural reinforcement should have an area at least equal to the area guide by 9.6.3.1. The minimum number of bars required by 10.5.3.6 should be complied with.

10.5.3.4.4 Maximum reinforcement area

Negative flexural reinforcement area should not exceed the values set forth in 9.6.3.2.

10.5.3.4.5 Minimum and maximum reinforcement spacing

Negative flexural reinforcement should not be spaced closer than guide by 9.4.9 and 10.5.3.5. The maximum reinforcement spacing should comply with 10.5.3.6. When two or more layers of negative reinforcement are employed the layers should not be placed closer than permitted by 9.4.10. Negative reinforcement of T-beam construction should comply with 10.5.3.8.1.

10.5.3.4.6 Cut off points

It should be permitted to suspend the negative flexural reinforcement, except for cantilevers, at the locations indicated in 10.5.4.5 or 11.1.5. Where adjacent spans are unequal, cut-off points of negative flexural reinforcement should be based on the guides for the longer span.

10.5.3.4.7 Reinforcement splicing

It should not be permitted to lap-splice negative flexural reinforcement between the cut-off point and the support.

10.5.3.4.8 End anchorage of reinforcement

Negative flexural reinforcement at the end of a girder, beam or joist should be anchored employing a standard hook into the edge girder, beam, or column, or structural concrete wall that provides support at the edge, complying with the anchorage distance required by 9.5.3. At the external edge of cantilevers negative flexural reinforcement perpendicular to the edge should end in a standard hook.

10.5.3.4.9 Negative flexural reinforcement acting in compression

Negative flexural reinforcement acting in compression should be surrounded with stirrups or ties that comply with 9.7.3.2.

10.5.3.4.10 Negative reinforcement for support of stirrups

In the distance along the span of the girder, beam, or joist, between negative reinforcement cut-off points, negative reinforcement should be provided for attachment and anchorage of stirrups. The diameter of the bars should be greater or equal to the bar diameter of the stirrups. It should be permitted to lap splice these bars a length greater or equal to 150 mm.

10.5.3.5 Maximum number of longitudinal bars in a layer

The maximum number of longitudinal bars in a layer should be determined for the longitudinal and transverse reinforcement bar diameters, the appropriate concrete cover (see 0), the maximum nominal coarse aggregate size, and the minimum clear spacing between bars (see 9.4.7). When these computations are not performed, it should be permitted to employ the guides of 10.5.3.5.1 to 10.5.3.5.3.

10.5.3.5.1 Girders and beams with bw ≥ 300 mm

For girders and beams whose width, b_w, is greater or equal to 300 mm it should be permitted to determine the maximum number of bars in a layer employing Equation 67 where b_w is the girder or beam width in mm. See Table 27.

$$\text{No. of bars in a layer} \leq \frac{b_w}{50} - 3 \qquad \text{Equation (67)}$$

10.5.3.5.2 Girders and beams with bw < 300 mm

Three longitudinal bars should be permitted for girders and beams whose width, b_w, is less than 300 mm and greater or equal to 250 mm. Two longitudinal bars should be employed for girders and beams whose width, b_w, is less than 250 mm. See Table 27.

Table 27 — Maximum number of longitudinal bars in a layer for girders and beams

Beam web width bw (mm)	Maximum number of longitudinal bars
b_w < 200 mm	section not permitted
200 mm ≤ b_w < 250 mm	2 bars
250 mm ≤ b_w < 300 mm	3 bars
300 mm ≤ b_w	$\leq \left(\frac{b_w}{50} - 3\right)$ bars

10.5.3.5.3 Joists

The maximum number of longitudinal bars in joists should be one for web widths b_w, (see Figure 53), less or equal to 150 mm, but it should be permitted to bundle in contact up to two bars locating one in top of the other. For web widths greater than 150 mm and less than 200 mm the maximum number of bars in a single layer should be two, and it should not be permitted to bundle them. For web widths greater and equal to 200 mm the maximum number of bars in a single layer should be one more than those allowed for girders and beams in 10.5.3.5.1 and 10.5.3.5.2.

10.5.3.6 Minimum number of longitudinal bars in a layer

To minimize flexural cracking width at points of maximum moment a larger number of smaller diameter bars should be employed as opposite to a small number of large diameter bars. For joists the minimum number of longitudinal bars should be one. The guides of 10.5.3.6.1 and 10.5.3.6.2 should be met at sections of maximum positive and negative moment for girders and beams whose width, b_w, is greater or equal to 300 mm. For girders and beams with b_w less than 300 mm the minimum number of longitudinal bars should be two.

10.5.3.6.1 Exterior exposure

The minimum number of longitudinal bars in a layer that should be employed for girders, and beams that are exposed to earth or weather, should be greater or equal to the value given by Equation 68, where b_w is the girder or beam width in mm.

$$\text{No. of bars in a layer} \geq \frac{b_w}{100} \qquad \text{Equation (68)}$$

10.5.3.6.2 Interior exposure

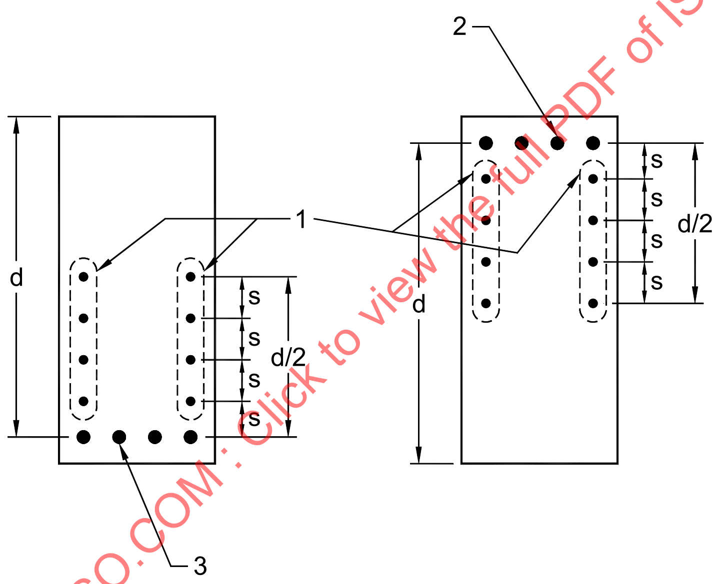
The minimum number of longitudinal bars in a layer that should be employed for girders, and beams, that are not exposed to earth or weather, should be greater or equal to the value given by Equation 69, where b_w is the girder or beam width in mm.

$$\text{No. of bars in a layer} \geq \frac{b_w}{200} \qquad \text{Equation (69)}$$

10.5.3.7 Skin reinforcement

If the effective depth d of a girder, beam, or joist exceeds 800 mm, longitudinal skin reinforcement should be uniformly distributed along both side faces of the member for a vertical distance equal to d/2 nearest the flexural tension reinforcement. The vertical spacing s, in mm, between bars should be obtained using Equation 70, but it should not exceed d/6, nor 300 mm. See Figure 70.

$$s = \frac{1000 \cdot A_b}{d - 750} \qquad \text{Equation (70)}$$



Key

1 skin reinforcement

2 negative reinforcement in tension

3 positive reinforcement in tension

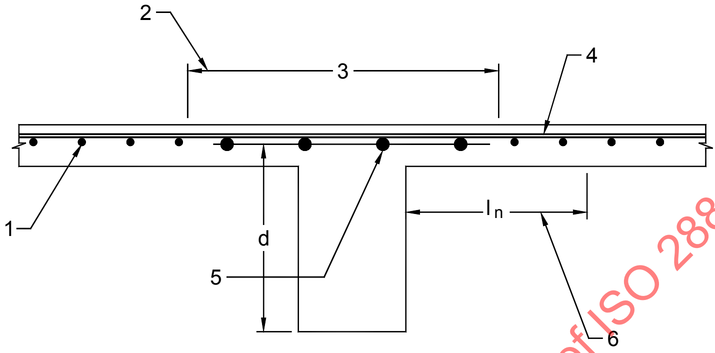
Figure 70 — Skin reinforcement for girders, beams and joists with d > 800 mm

10.5.3.8 Reinforcement in flanges of T-beams

For girders and beams that are shaped as T-beams, except joists, with a flange or slab in the upper part, the guides of 10.5.3.8.1 and 10.5.3.8.2 for the reinforcement located in the flange should be employed. When the girder or beam is part of a slab-on-girder system, the reinforcement in the flange should not be less than the required for the slab.

10.5.3.8.1 Distribution of negative flexural reinforcement in flanges of T-beams

Where flanges of T-beam construction are in tension negative flexural reinforcement in the direction of the beam should be distributed over width equal to the smaller of the effective flange width defined in 10.1.6 or one-tenth of the span of the beam. If the effective flange width of 10.1.6 exceeds one-tenth of the span, the rest of the effective flange width should have reinforcement in the direction of the beam greater or equal to the shrinkage and temperature reinforcement for slabs of 9.6.2.1. See Figure 71. For this case the guides of 10.5.3.5 should not apply.



Key

1 shrinkage and temperature reinforcement outside of with for distribution of negative reinforcement

2 whith for distribution of negative flexural reinforcement in tension

3 minimun of the effective flange whith or one-tenth of the beam span

4 transverse reinforcement calculated supposing the flange acts as a cantilever

5 beam negative flexural reinforcement in tension

6 clear cantilever span for obtaining transverse reinforcement, equal to overhanging portion of effective flange width or full overhang for isolated T-beams

Figure 71 — Reinforcement in flanges of T-beams

10.5.3.8.2 Transverse flange reinforcement

In the top of the flange reinforcement perpendicular to the beam should be provided to resist the factored negative moment obtained from Equation 44, using a value of l_m equal to the overhanging portion of the flange width defined in 10.1.6, and for isolated T-beams the full width of overhanging flange or the flange width defined in 10.1.6. This reinforcement should comply with the guides for negative flexural reinforcement in slabs as set forth in 10.4.3.4. See Figure 71.

10.5.3.9 Girder and beam reinforcement in seismic zones

In girders and beams supported directly on columns and structural concrete walls that are part of a moment resisting frame located in seismic zones, reinforcement should comply with the additional requirements of chapter 13. Joists and beams that are not part of a frame are exempt from the additional seismic guides.

10.5.4 Joists and beams supported on girders

10.5.4.1 General

The guides of 10.5.4 cover joists and beams that are supported on girders, and are cast monolithically with them. Two-way joist systems or waffle-on-beams systems, as described in 10.3.1.3.4, are covered and should comply with the general guides of 10.3.1.

10.5.4.2 Dimensional specifications

The following Dimensional specifications should be observed:

10.5.4.2.1 Joists

In addition to the appropriate guides of 1.5, joists should comply with the general Dimensional specifications of 6.1, and the particular guides of 10.3.1.3.1. The minimum allowable depth should comply with 10.3.5.3 for one-way joist and of 10.3.5.5 for two-way joists.

10.5.4.2.2 Beams

In addition to the appropriate guides of 10.5, beams supported on girders should comply with the general Dimensional specifications of 6.1, and the particular guides of 10.3.1.2. The minimum allowable depth should comply with 10.3.5.3. The width of the web of beams, b_w, should not be less than 200 mm. The spacing between lateral supports of isolated beams should not exceed 50 times the least width b of compression flange or face.

10.5.4.2.3 Cantilevers of joists and beams

All cantilevers of joists or beams should be the external continuation of an element that spans between supports provided by beams, girders or structural walls. No double cantilever should be permitted.

10.5.4.3 Factored flexural moment

10.5.4.3.1 Cantilevers of joists and beams supported on beams or girders

The factored negative flexural moment, M^-_u, for beam and joist cantilevers that span out of the edge supporting girders, beams, should be calculated supposing that one-half of the distributed factored load, W_u, acts as a concentrated load at the tip of the cantilever along with all concentrated loads that act on the span of the cantilever, $\sum P_u$, and the other one-half acts as uniformly distributed load over the full span, using Equation 71, but it should not be less than the factored negative flexural moment of the first interior span at the exterior supporting girder, beam, nor less than one-third of the positive moment, in the same direction, of the first interior span.

$$M_u^- = \frac{3 \cdot W_u \cdot l_n^2}{4} + l_n \cdot \sum P_u \qquad \text{Equation (71)}$$

where l_m should be the clear span of the cantilever in m, W_u should be employed in N/m, $\sum P_u$ should be in N, and M_u^- should be obtained in N · m.

10.5.4.3.2 One-span joists and beams supported on beams or girders

The factored positive and negative flexural moment, M_u, in N · m, for one-span beams and one-span one-way joists should be calculated using the Equation 71 and Equation 72, where l_m should be the clear span of the beam or joist in m, W_u should be employed in N/m, and ΣP_u should be in N.

Positive moment:

$$M_u^+ = \frac{W_u \cdot l_m^2}{8} + \frac{l_m}{4} \cdot \sum P_u \qquad \text{Equation (72)}$$

Negative moment at supports:

$$M_u^- = \frac{W_u \cdot l_m^2}{24} + \frac{l_m}{16} \cdot \sum P_u \qquad \text{Equation (73)}$$

10.5.4.3.3 Joists and beams supported on beams, girders , with two or more spans

The factored positive and negative flexural moment, M_u, in N · m, for beams and one-way joists supported on beams, girders should be calculated using the Equation 74 to Equation 79, where l_m should be the clear span of the beam or joist in m, W_u should be employed in N/m, and ΣP_u should be in N.

Positive moment

at end spans

$$M_u^+ = \frac{W_u \cdot l_m^2}{11} + \frac{l_m}{9} \cdot \sum P_u \qquad \text{Equation (74)}$$

at interior spans:

$$M_u^+ = \frac{W_u \cdot l_m^2}{16} + \frac{l_m}{5} \cdot \sum P_u \qquad \text{Equation (75)}$$

Negative moment at supports

at interior face of external support

$$M_u^- = \frac{W_u \cdot l_m^2}{24} + \frac{l_m}{16} \cdot \sum P_u \qquad \text{Equation (76)}$$

at exterior face of first internal support, only two spans

$$M_u^- = \frac{W_u \cdot l_m^2}{9} + \frac{l_m}{6} \cdot \sum P_u \qquad \text{Equation (77)}$$

at faces of internal supports, more than two spans

$$M_u^- = \frac{W_u \cdot l_m^2}{10} + \frac{l_m}{7} \cdot \sum P_u \qquad \text{Equation (78)}$$

at faces of all supports for joists with spans not exceeding 3 m:

$$M_u^- = \frac{W_u \cdot l_m^2}{12} + \frac{l_m}{8} \cdot \sum P_u \qquad \text{Equation (79)}$$

10.5.4.3.4 Use of frame analysis for joists and beams supported on beams or girders

It should be permitted to use a frame analysis for obtaining the factored moment and shear as a substitute for the values obtained from 10.5.4.3.1 to 10.5.4.3.3, and 10.5.4.4.1 to 10.5.4.4.3, if the following guides are met:

a) The analysis procedure should be based on established principles of structural mechanics.

b) The procedure should take into account equilibrium, compatibility of deformations, general stability, and short- and long-term material properties.

c) The analysis procedure should take into account the flexibility of the supports and the interaction between flexure and torsion of the supported and supporting elements.

d) The modulus of elasticity of concrete should be permitted to be taken as $E_c = 4500\sqrt{f_c'}$, in MPa.

e) Use of any set of reasonable assumptions should be permitted for computing relative flexural and torsional stiffness of the structural elements. The assumptions adopted should be consistent throughout the analysis.

f) Span length should be taken as the distance center-to-center of supports, but it should be permitted to obtain the factored moment and shear at faces of supports.

g) It should be permitted to assume that the arrangement of live load is limited to combinations of factored dead load on all spans with full factored live load on two adjacent spans, and factored dead load on all spans with full factored live load on alternate spans.

10.5.4.3.5 Two-way joists supported on beams or girders

It should be permitted to obtain the factored moment for two-way joists supported on beams, employing the guides of 10.4.8.1 and 10.4.8.2, except the minimum depth of the supporting beams or girders guide in 10.4.8.1 (c) (see 10.3.1.3.4).

10.5.4.4 Factored shear

10.5.4.4.1 Cantilevers of joists and beams supported on beams or girders

The factored shear, V_u, at the support of cantilevers should be calculated using Equation 80, where l_m should be the clear span of the cantilever in m, W_u should be employed in N/m, and $\sum P_u$ in N.

$$V_u = W_u \cdot l_m + \sum P_u \quad \text{Equation (80)}$$

10.5.4.4.2 One-span joists and beams supported on beams or girders

The factored shear, V_u, in N, for one-span beams and one-span one-way joists should be calculated using Equation 81, where l_m should be the clear span of the beam or joist in m, W_u should be employed in N/m, and $\sum P_u$ should be in N.

$$V_u = \frac{W_u \cdot l_m}{2} + 0{,}8 \cdot \sum P_u \quad \text{Equation (81)}$$

10.5.4.4.3 Joists and beams supported on beams or girders, with two or more spans

The factored positive and negative flexural moment, M_u, in N · m, for beams and one-way joists supported on beams or girders should be calculated using the Equation 82 and Equation 83, where l_m should be the clear span of the beam or joist in m, W_u should be employed in N/m, and ΣP_u should be in kN.

at exterior face of first interior support

$$V_u = 1{,}15 \cdot \frac{W_u \cdot l_m}{2} + 0{,}80 \cdot \sum P_u \quad \text{Equation (82)}$$

at faces of all other supports

$$V_u = \frac{W_u \cdot l_m}{2} + 0{,}75 \cdot \sum P_u$$ Equation (83)

10.5.4.4.4 Use of frame analysis

It should be permitted to use a frame analysis for obtaining the factored shears as a substitute for the values obtained from 10.5.4.4.1 to 10.5.4.4.3 if the guides of 10.5.4.3.4 are met.

10.5.4.4.5 Two-way joists supported on beams or girders

It should be permitted to obtain the factored shear for two-way joists supported on beams or girders employing the guides of 10.4.8.1 and 10.4.8.4, except the minimum depth of the supporting beams or girders guide in 10.4.8.11(c) (see 10.3.1.3.4).

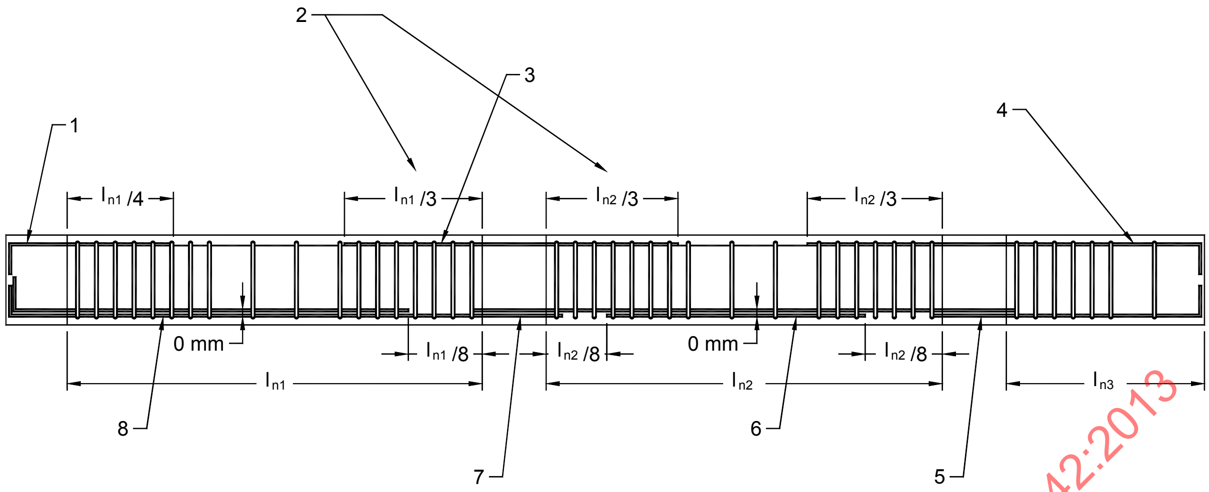
10.5.4.5 Reinforcement

10.5.4.5.1 Positive flexural reinforcement

The positive reinforcement area should be determined employing Equation 23 or Equation 24, with the appropriate value of M_u^+ obtained from 10.5.4.3 converted to N · mm (1 N · m = 10^3 · N · mm), using d and b in mm. When a slab is present in the upper part of the section, or when the beam or joist is T-shaped, it should be permitted to employ the T-beam effect as indicated in 10.1.6. The positive flexural reinforcement should comply with the guides of 10.5.3.3. At internal supports, at a distance equal to $l_m/8$ measured from the face of the supports into the span, up to one-half of the positive flexural reinforcement should be permitted to be suspended if there are no concentrated loads within that distance. For one-span beams and joists no suspension of positive reinforcement should be permitted. See Figure 72.

10.5.4.5.2 Negative flexural reinforcement

The negative flexural reinforcement area should be determined employing Equation 23 or Equation 24, for the greater value of M_u^- obtained from 10.5.4.3 for both sides of the support, converted to N · mm (1 N · m = 10^3 · N · mm), using d and b in mm. This reinforcement should comply with the guides of 10.5.3.4. When a slab is present in the upper part of the section, or when the beam or joist is T-shaped, negative flexural reinforcement should comply with 10.5.3.8. At a distance equal to $l_m/4$ for external supports, and $l_m/3$ for internal supports, measured from the internal face of the support into the span, all the negative flexural reinforcement should be permitted to be suspended. See Figure 72. No suspension of negative reinforcement should be permitted in cantilevers.



Key

1 negative reinforcement at face of external support

2 negative reinforcement cut-off points based greater of the neighboring spans

3 negative reinforcement interior support

4 greater of cantilever negative reinforcement or required for internal support

5 splice according to 9.5.2

6 positive reinforcement interior span

7 splice according to 9.5.2

8 positive reinforcement end span

Figure 72 — Reinforcement for beams and joists supported on beams or girders

10.5.4.5.3 Transverse reinforcement

The values of V_u at the faces of the right and left supports should be obtained using the appropriate equation from 10.5.4.4. A diagram showing the shear variation within the span should be constructed, starting with the value of V_u in N, at the face of the left support taken as positive. The shear from this point proceeding to the right should be decreased at a rate equal to $[(V_u)_{\text{left supp.}} + (V_u)_{\text{right supp.}} - \sum P_u]/l_m$, in kN/m. At any place were a concentrated load is applied, the value of P_u in N, should be subtracted from the value of shear shown in the diagram at the left of the load. Proceeding as described, at the face of the left support the negative value of V_u in N there should be reached in the diagram. See Figure 73. At any place within the span, the value of $(l \cdot V_n)$ as calculated following the guides of 10.2.4 should be greater or equal than the absolute value of $V_u(x)$ as shown in the calculated diagram.

The shear reinforcement should comply with the guides of 10.5.3.2 and 10.2.4. The limits for $(\phi \cdot V_n)$ as defined in 10.2.4.4 should be marked in the shear diagram, and a minimum amount of shear reinforcement as defined by Equation 36 should be established. Appropriate values of the spacing of stirrups s should be defined for the different region within the shear diagram. A minimum practicable spacing of stirrups as guide by 9.4.9 should be observed. The first stirrup should not be placed further than $s/_2$ from the face of the supporting element, being s the required spacing of stirrups at the support.

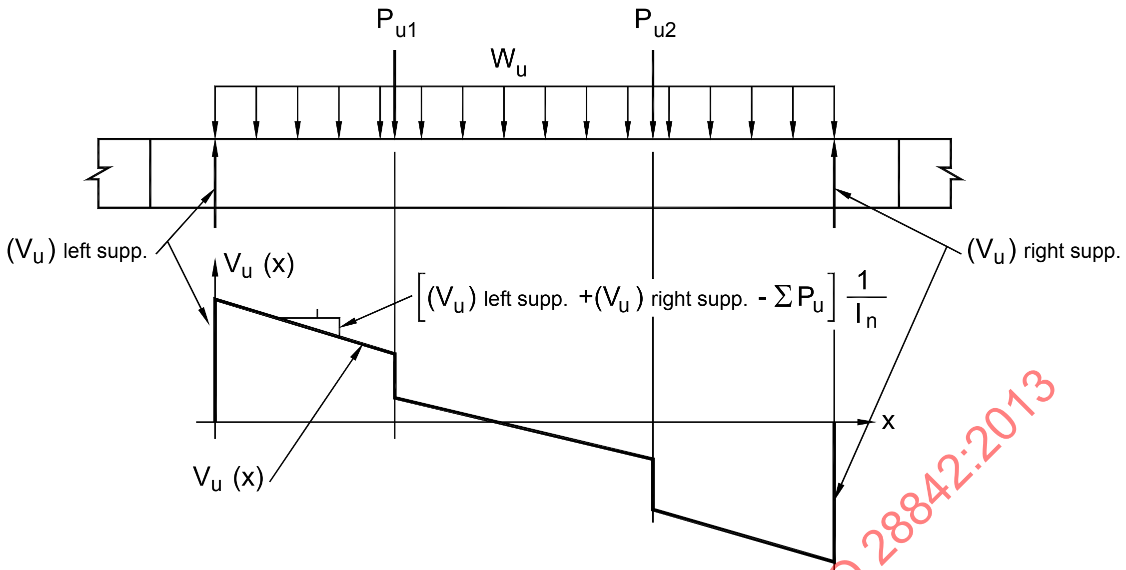


Figure 73 — Calculation of the shear diagram of the beam or joist supported on beams or girders

10.5.4.5.4 Hanger reinforcement

When a beam is supported by a girder of essentially the same depth, hanger reinforcement should be provided in the joint. The forces from the reaction from the supported beam tend to push down the bottom of the supporting girder, and should be resisted by hanger reinforcement in the form of closed stirrups placed in both elements in addition to the stirrups for shear. See Figure 74. The determination of the hanger reinforcement should be made complying with:

a) If V_u from the supported beam at the interface is less than $\left[\phi \cdot \frac{\sqrt{f_c'}}{4} \cdot b_w \cdot d\right]$ it should be permitted to waive the hanger reinforcement, where ϕ = [0,85]

b) If h_b is greater that one-half the total depth of the supporting girder, where h_b is the vertical distance measured from the bottom of the supporting girder to the bottom of the supported beam, it should be permitted to waive the hanger reinforcement.

c) The hanger reinforcement should be full depth closed stirrups with a total area A_i as determined from Equation 84.

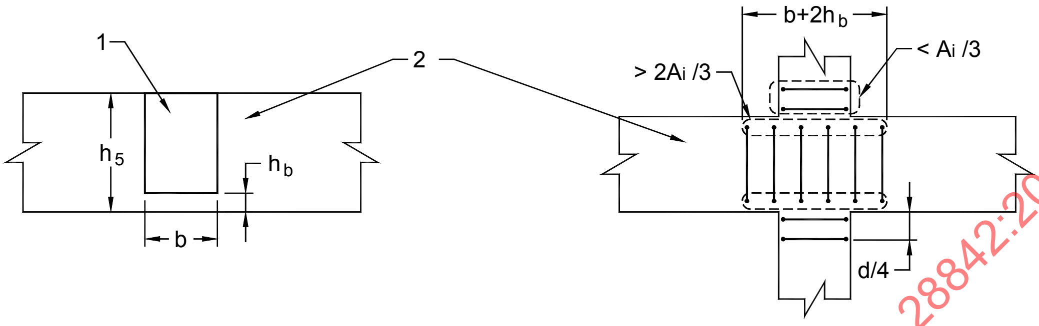
$$A_i \geq \frac{\left(1 - \frac{h_b}{h_s}\right) V_u}{\phi \cdot f_y} \qquad \text{Equation (84)}$$

d) In Equation 84 V_u is the shear from the supported beam at the interface, A_i is the area of the additional stirrups, h_s is the total height of the supporting girder, f_y is the specified yield strength of the steel of the stirrups, and ϕ = [0,85]

e) Additional stirrups with an area greater or equal to 2/3 of A_i should be placed in the supporting girder for a distance along the longitudinal axis of the supporting girder equal or less than the width of the supported beam, b_w, plus h_b at each side. In the computation of the 2/3 of A_i, only the area of legs of the additional stirrups that are located at the side of the supported beam should be taken into account.

f) Additional stirrups with an area not greater than 1/3 of A_i should be placed in the supported beam for a distance d/4 along the longitudinal axis of the supported beam from the face of the supporting girder, where d is the effective depth of the supported beam.

g) The bottom longitudinal reinforcement of supported beam should be placed over the bottom longitudinal reinforcement of the supporting girder.



Key

1 supported beam

2 supported girder

Figure 74 — Hanger reinforcement

10.5.4.6 Calculation of the reactions on beams and girders

10.5.4.6.1 One-way joists

The factored reaction on the supports of joist should be permitted to be considered uniformly distributed. The factored reaction on the supports r_u, in N/m, should be the value obtained from Equation 85 plus the uniformly distributed reaction from any cantilever spanning from that support. In Equation 85 V_u is the factored shear from 10.5.4.4, in N, l is the center-to-center span of the joist, in m, ℓ_n is the clear span of the joist in m, and s is the center-to-center spacing between joists also in m (see Figure 53).

$$r_u = \frac{V_u \cdot l}{s \cdot l_n} \qquad \text{Equation (85)}$$

10.5.4.6.2 Two-way joists supported on beams or girders

It should be permitted to obtain the required factored reactions for two-way joists supported on beams, girders or structural walls employing the guides of 10.4.8.1 and 10.4.8.4, except the minimum depth of the supporting beams or girders required in 10.4.8.1(c), (see 10.3.1.3.4).

10.5.4.6.3 Beams

The factored reaction on the supports R_u, in N, should be the value obtained from Equation 86 plus the factored reaction from any cantilever spanning from that support. In Equation 86 V_u is the factored shear from 10.5.4.4, in N, l is the center-to-center span of the beam, in m, and l_m is the clear span of the beam, also in m.

$$R_u = \frac{V_u \cdot l}{l_m} \qquad \text{Equation (86)}$$

10.6 Railings

Railings shall be provided along the edges of structures for protection of traffic and pedestrians. Except on urban expressways, a pedestrian walkway may be separated from an adjacent roadway by a traffic railing or barrier with a pedestrian railing along the edge of the structure. On urban expressways, the spacing shall be made by a combination railing.

Materials for railings shall be concrete, metal, timber or a combination thereof.

10.6.1 Vehicular Railing

10.6.1.1 General

Although the primary purpose of traffic railings is to contain the average vehicle using the structure, consideration should be also given to:

- Protection of the occupants of a vehicle in case of a collision with the railing.
- Protection of vehicles or pedestrians on roadways underneath the structure.
- Appearance and freedom of view from passing vehicles.

10.6.1.2 Geometry

The height of railings shall be measured with respect to a reference surface which may be the top of the roadway, the top of the future overlay, if resurfacing is anticipated, or the top of curb.

Traffic railings height shall not be less than 0.70 m from the reference surface. Parapets designed with sloping traffic faces intended to allow vehicles to ride them up with low angle contacts shall be at least 0.80 m in height.

The lower element of a traffic railing should consist of a parapet projecting at least 0.50 m above the reference surface. The maximum clear opening below the bottom rail shall not exceed 0.50 m and the maximum opening between succeeding rails shall not exceed 0.40 m.

10.6.2 Bicycle Railing

10.6.2.1 General

Bicycle railing components shall be designed with consideration to safety, appearance and, when the bridge carries mixed traffic, freedom of view from passing vehicles.

10.6.2.2 Geometry

The minimum height of a railing used to protect a bicyclist shall be 1.40 m, measured from the top of the surface on which the bicycle rides to the top of the upper rail member.

All railing elements located below 0.7 m above the surface level should not be spaced more than 0.15 m from each other. Elements located between 0.7 m and the total height of the railings may have any spacing length. If a railing assembly employs both horizontal and vertical elements, the spacing requirements shall apply to one or the other, but no to both.

10.6.3 Pedestrian Railing

10.6.3.1 General

Railing components shall be proportioned according to the type and volume of anticipated pedestrian traffic. Consideration should be given to appearance, safety and freedom of view from passing vehicles.

10.6.3.2 Geometry

The minimum height of a pedestrian railing shall be 1.10 m, measured from the top of the walkway to the top of the upper rail member.

All railing elements located below 0.7 m above the surface level should not be spaced more than 0.15 m from each other. For elements between 0.70 m and 1.10 m above the walking surface, elements shall be spaced no more than 0.30 m between each other.

11 Substructure

A substructure is any structural, load supporting component generally referred to by the terms abutment, pier, column, frame, structural wall, or other similar terminology.

11.1 Girders that are part of a frame

11.1.1 General

The guides of 11.1 cover girders that are part of a moment resistant frame where the girders are cast monolithically and are supported directly by columns or structural concrete walls.

11.1.2 Dimensional specifications

11.1.2.1 General

In addition to the appropriate guides of 10.5, girders that are part of a frame should comply with the general Dimensional specifications set forth in 6.1, and the particular guides for beams spanning between columns of 10.3.1.

11.1.2.2 Girder depth and width

The girder should be prismatic without haunches, brackets or corbels. The height h should comply with the minimum thickness guides of 10.3.5.2. The clear span of the member should not be less than four times its height h. The width-to-height (b_w/h) ratio should not be less than 0,3. The width b_w should not be less than 200 mm, nor more than the width of the supporting column (measured on a plane perpendicular to the longitudinal axis of the girder) plus distances on each side of the supporting member not exceeding 3/4 of the height h of the girder. See Figure 75.

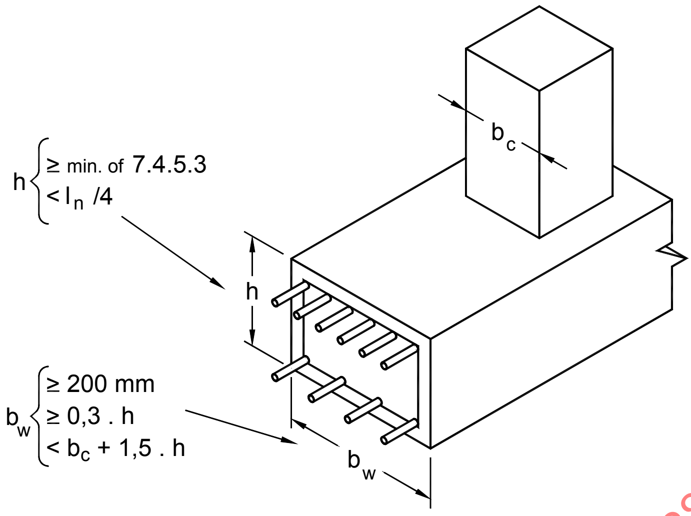


Figure 75 — Limits on girder depth and width

11.1.2.3 Girders supported by structural concrete walls

Girders supported by structural concrete walls should continue along the full horizontal length of the wall when the wall is located in the plane of the frame. The width of the girder should not be less than the thickness of the wall. When girders are supported by walls perpendicular to the longitudinal axis of the girder, the walls should be provided with a beam that runs along the full horizontal length of the wall at the same level and having the same depth of the girder. The width of the beam should not be less than the thickness of the wall, neither 200 mm. Vertical reinforcement of the wall should pass through the girder or beam as guide in 11.6.

11.1.2.4 Lateral Support

In girders that are not laterally supported by the floor slab or secondary beams the clear distance between lateral supports should not exceed 50 times the least width b of compression flange or face.

11.1.2.5 Special guides

The following restrictions should be in effect for girders of frames designed under 11.1:

a) there are two or more spans,

b) the spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 per cent of the larger span (see 6.1),

c) loads are uniformly distributed, and adjustments for concentrated loads are performed,

d) unit live load, wl, does not exceed three times unit dead load, wd, and

e) sloping girders should not have a slope exceeding 15°.

11.1.3 Factored flexural moment

11.1.3.1 Factored positive and negative moment

The factored positive and negative moment, M_u, in N · m, for girders and beams that are part of a frame where the vertical elements are columns and concrete structural walls, should be calculated using the Equation 87 to

Equation 93, where l_m is the clear span in m, W_u should be employed in N/m, and $\sum P_u$ corresponds to the sum of all total factored concentrated loads that act on the span, in N.

Positive moment

at end spans

$$M_u^+ = \frac{W_u \cdot l_m^2}{14} + \frac{l_m}{6} \cdot \sum P_u \quad \text{Equation (87)}$$

at interior spans:

$$M_u^+ = \frac{W_u \cdot l_m^2}{16} + \frac{l_m}{7} \cdot \sum P_u \quad \text{Equation (88)}$$

Negative moment at supports

at interior face of external column or perpendicular structural wall

$$M_u^- = \frac{W_u \cdot l_m^2}{16} + \frac{l_m}{10} \cdot \sum P_u \quad \text{Equation (89)}$$

at exterior face of first internal column or perpendicular structural wall, only two spans

$$M_u^- = \frac{W_u \cdot l_m^2}{9} + \frac{l_m}{6} \cdot \sum P_u \quad \text{Equation (90)}$$

at faces of internal columns or perpendicular structural walls, more than two spans

$$M_u^- = \frac{W_u \cdot l_m^2}{10} + \frac{l_m}{6.5} \cdot \sum P_u \quad \text{Equation (91)}$$

at faces of structural walls parallel to the plane of the frame

$$M_u^- = \frac{W_u \cdot l_m^2}{12} + \frac{l_m}{7} \cdot \sum P_u \quad \text{Equation (92)}$$

at support of girder cantilevers

$$M_u^- = \frac{3 \cdot W_u \cdot l_m^2}{4} + l_m \cdot \sum P_u \quad \text{Equation (93)}$$

11.1.3.2 Girders of frames parallel to the direction of one-way joist systems

In order to take into account the effect of the distribution ribs of the joist system (see 10.3.1.3.3) on girders of frames parallel to the direction of one-way joist systems, a factored load equivalent to two times that used to design the individual joist should be employed in addition to the loads on the girder. This effect should also be employed in obtaining the factored shear in 11.1.4.1.

11.1.3.3 Use of frame analysis

It should be permitted to use a frame analysis for obtaining the factored flexural moment and shear as a substitute for the values obtained from 11.1.3.1 and 11.1.4.1 if the following guides are met:

a) The analysis procedure should be based on established principles of structural mechanics.

b) The procedure should take into account equilibrium, compatibility of deformations, general stability, and short- and long-term material properties.

c) The modulus of elasticity of concrete should be permitted to be taken as $E_c = 4500\sqrt{f'_c}$, in MPa.

d) Use of any set of reasonable assumptions should be permitted for computing relative flexural and torsional stiffness of columns, walls, beams and girders. The assumptions adopted should be consistent throughout the analysis.

e) Span length should be taken as the distance center-to-center of supports, but it should be permitted to obtain the factored moment and shear at faces of supports.

f) It should be permitted to assume that the arrangement of live load is limited to combinations of factored dead load on all spans with full factored live load on two adjacent spans, and factored dead load on all spans with full factored live load on alternate spans.

11.1.4 Factored shear

11.1.4.1 Factored shear

V_u, in N, for the slab should be calculated at the faces of all supports using the Equation 94 to Equation 96, where l_m is the clear span in m, W_u should be employed in N/m, and $\sum P_u$ corresponds to the sum of all total factored concentrated loads that act on the span, in N.

at exterior face of first interior column

$$V_u = 1{,}15 \cdot \frac{W_u \cdot l_m}{2} + 0{,}80 \cdot \sum P_u \qquad \text{Equation (94)}$$

at faces of all other columns

$$V_u = \frac{W_u \cdot l_m}{2} + 0{,}75 \cdot \sum P_u \qquad \text{Equation (95)}$$

at supports of girder cantilevers

$$V_u = W_u \cdot l_m + \sum P_u \qquad \text{Equation (96)}$$

11.1.4.2 Use of frame analysis

It should be permitted to use a frame analysis for obtaining the factored shears as a substitute for the values obtained from 11.1.4.1 if the guides of 11.1.3.3 are met.

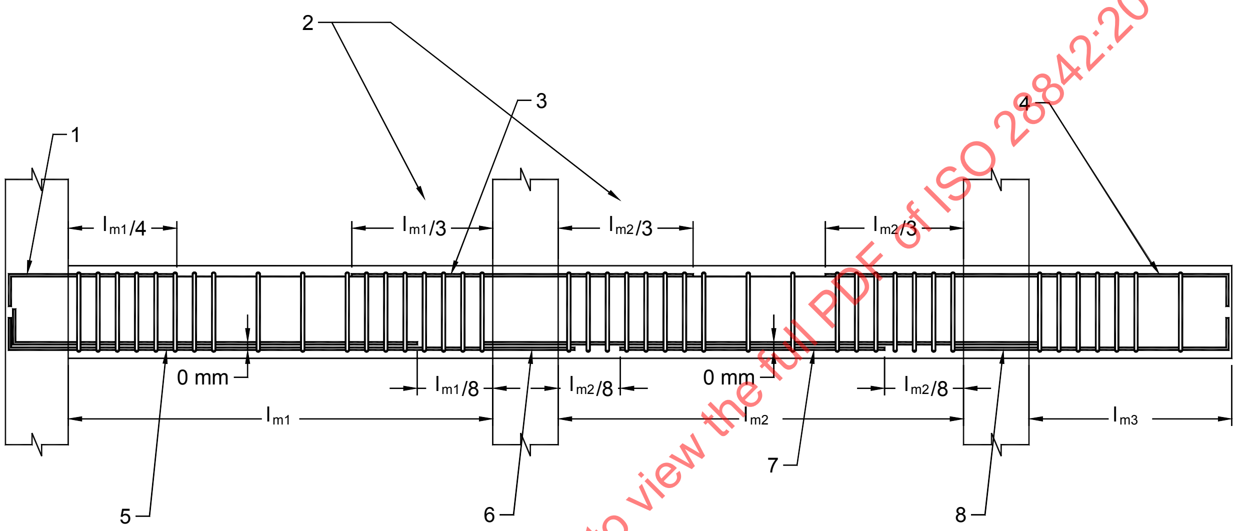
11.1.5 Reinforcement

11.1.5.1 Positive flexural reinforcement

The positive flexural reinforcement area should be determined employing the guides of 10.1.4, with the appropriate value of M_u^+ obtained from Equation 87 or Equation 88, converted to N · mm (1 N · m = 10^3 · N · mm), using d and b in mm. If a slab exist in the upper part of the girder, it should be permitted to employ the guides to take into account the T-beam effect of 10.1.6. Positive flexural reinforcement should comply with the guides of 10.5.3.3. At internal supports, at a distance equal to $l_m/8$ measured from the face of the supports into the span, up to one-half of the positive flexural reinforcement should be permitted to be suspended if there are no concentrated loads within that distance. See Figure 76 and Figure 77.

11.1.5.2 Negative flexural reinforcement

The negative flexural reinforcement area should be determined employing the guides of 10.1.4, for the greater value of M_u^- for both sides of the support, obtained from Equation 89 to Equation 93, converted to N · mm (1 N · m = 10^3 · N · mm), using d and b in mm. This reinforcement should comply with the guides of 10.5.3.4. When a slab is present in the upper part of the section, or when the beam or joist is T-shaped, negative flexural reinforcement should comply with 10.5.3.8. At internal supports at a distance equal to $l_m/3$, and at external supports at a distance equal to $l_m/4$, measured from the face of the support into the span, all the negative flexural reinforcement should be permitted to be suspended (see Figure 76), except in the beams belonging to the perimeter ring of beams guide by 10.3.3.2, where one-quarter of the negative reinforcement should be continuous through the span, or spliced at midspan (see Figure 77). No suspension of negative reinforcement should be permitted in cantilevers.



Key

1 negative reinforcement at interior face of external support

2 negative reinforcement cut-off points based greater of the neighboring spans

3 negative reinforcement interior support

4 greater of cantilever negative reinforcement or required for internal support

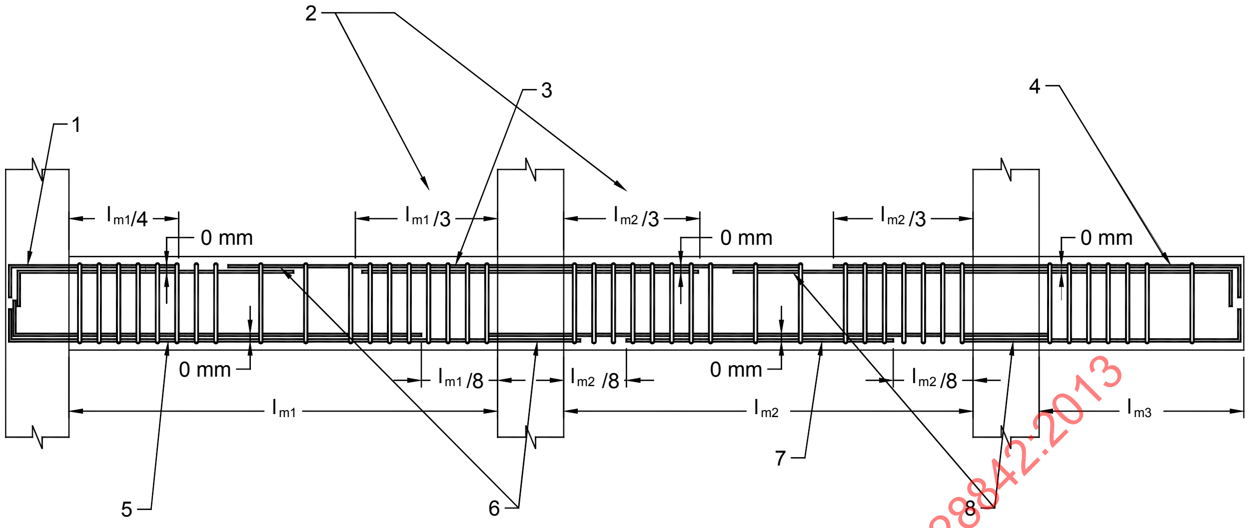
5 positive reinforcement end span

6 splice according to 9.5.2

7 positive reinforcement interior span

8 splice according to 9.5.2

Figure 76 — Reinforcement in girders that are part of a moment resisting frame by columns or structural concrete walls



Key

1 negative reinforcement at interior face of external support

2 negative reinforcement cut-off points based greater of the neighboring spans

3 negative reinforcement interior support

4 greater of cantilever negative reinforcement or required for internal support

5 positive reinforcement exterior span

6 splice according to 9.5.2

7 positive reinforcement interior span

8 splice according to 9.5.2

Figure 77 — Reinforcement in girders that are part of the perimeter frame

11.1.5.3 Transverse reinforcement

The values of V_u at the faces of the right and left supports should be obtained using the appropriate value from Equation 94 to Equation 96. A diagram of shear variation along the span should be calculated employing the guides of 10.5.4.5.3 (see Figure 73). At any place within the span, the value of $(\phi \cdot V_n)$ as calculated following the guides of 10.2.4 should be greater or equal than the absolute value of $V_u(x)$ as shown in the calculated diagram.

The shear reinforcement should comply with the guides of 10.5.3.2 and 10.2.4. The limits for $(\phi \cdot Vn)$ as defined in 10.2.4.4 should be marked in the shear diagram, and a minimum amount of shear reinforcement as defined by Equation 36 should be established. Appropriate values of the spacing of stirrups s should be defined for the different region within the shear diagram. A minimum practicable spacing of stirrups as guide by 9.4.9 should be observed. The first stirrup should not be placed further than s/2 from the face of the supporting element, being s the required spacing of stirrups at the support.

11.1.5.4 Hanger reinforcement

When the girder supports a beam of essentially the same depth hanger reinforcement as guide by 10.5.4.5.4 should be employed.

11.1.6 Calculation of the reactions on beams and girders

11.1.6.1 Vertical reaction at columns and walls

The factored reaction on the supports R_u, in N, should be the value obtained from Equation 97 plus the factored reaction from any cantilever spanning from that support. In Equation 97 V_u is the factored shear from Equation 94 to Equation 96, in N, l is the center-to-center span of the beam, in m, and l_m is the clear span of the beam, also in m.

$$R_u = \frac{V_u \cdot l}{l_m} \qquad \text{Equation (97)}$$

11.1.6.2 Unbalanced moment from vertical loading applied to girder

The moment reaction on columns should be evaluated using the unbalanced factored moment, ΔM_u, caused by the factored vertical loads on the girders on the plane of the frame that span from the column at that level. The unbalanced moment should be distributed to the column above and below the girder in proportion to their relative stiffnesses. The following procedure should be employed to calculate the unbalanced moment:

a) The unbalanced moment, ΔMu, should correspond to the largest difference in girder factored negative moment at the column when two load cases are evaluated.

b) In the first case (Case A of Figure 78) the whole girder should be loaded with the factored dead load and alternate spans should be loaded with the factored live load.

c) In the second case (Case B of Figure 78) the whole girder should be loaded with the factored dead load and the other alternate spans should be loaded with the factored live load.

11.1.6.3 Distribution of the unbalanced moment to the columns and walls

The following procedure should be employed to distribute the unbalanced moment to the columns or walls above and below the girder:

a) For joints of columns, or walls, supporting the roof girders (Types B, D, and F of Figure 79), the column factored moment should correspond to ΔMu.

b) For joints of columns, or walls, of floors different from the roof (Types A, C, and E of Figure 79), the unbalanced moment should be distributed to the column, or wall, above using Equation 98.

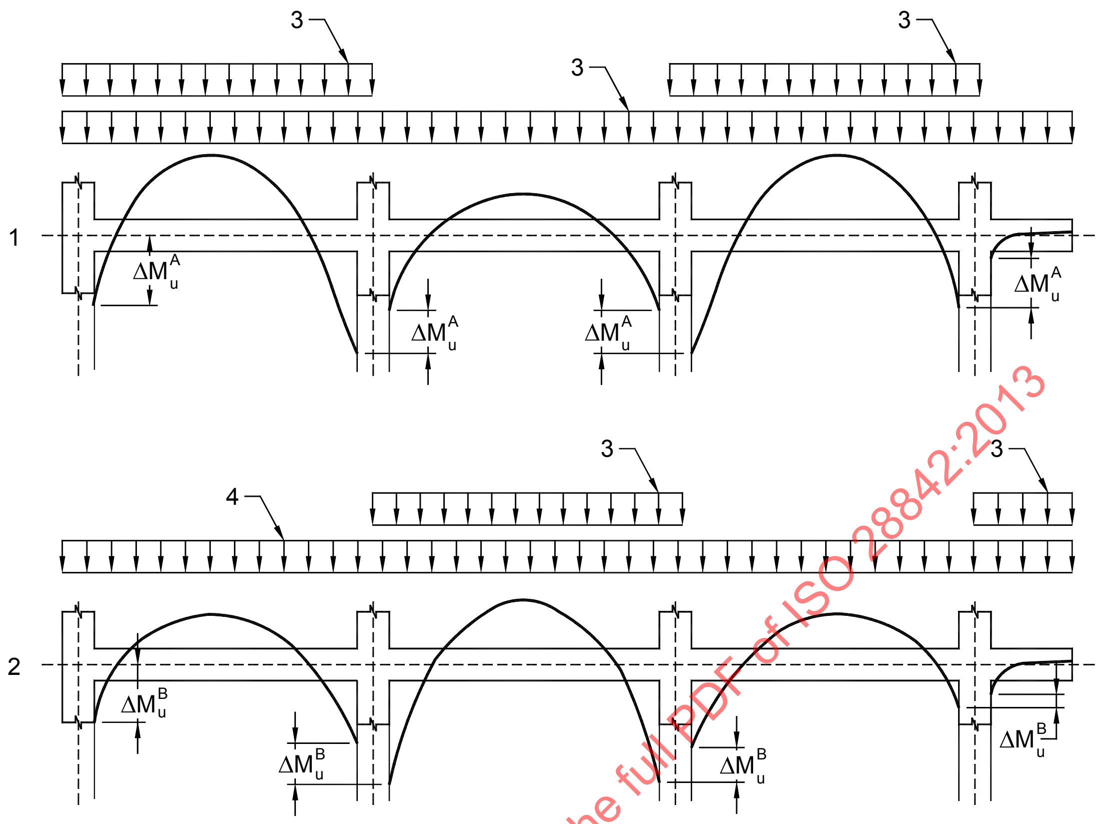
$$(M_u)_{up} = \Delta M_u \cdot \frac{(I_c/h_{pi})_{up}}{(I_c/h_{pi})_{up} + (I_c/h_{pi})_{down}} \qquad \text{Equation (98)}$$

c) For joints of columns, or walls, of floors different from the roof (Types A, C, and E of Figure 79), the unbalanced moment should be distributed to the column, or wall, below using Equation 99.

$$(M_u)_{down} = \Delta M_u \cdot \frac{(I_c/h_{pi})_{down}}{(I_c/h_{pi})_{up} + (I_c/h_{pi})_{down}} \qquad \text{Equation (99)}$$

d) In Equation 98 and Equation 99, Ic should be evaluated employing Equation 100, where bcol is the dimension of the column, or wall, section in the direction perpendicular to the girder span in m, hcol is the dimension of the column, or wall, section in the direction perpendicular to the girder span in m, hpi is the story height corresponding to the column or wall (see Figure 78).

$$I_c = \frac{b_{col} \cdot h_{col}^3}{12} \qquad \text{Equation (100)}$$



Key

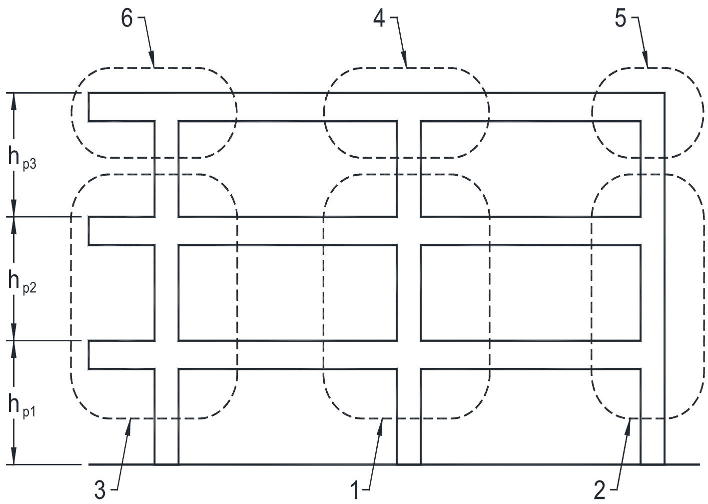
1 case A

2 case B

3 factored live load

4 factored dead load

Figure 78 — Girder unbalanced moment to be transferred to columns



Key

1	Tipe A	4	Type D
2	Type B	5	Type E
3	Type C	6	Type F

Figure 79 — Types of joints for determination of column moments

11.2 Strength of members subjected to axial loads with or without flexure

11.2.1 General

Calculation of the design strength of member sections of columns and structural concrete walls subjected to axial loads or axial loads accompanied by flexural moments should be performed employing the requirements of 11.2.

11.2.2 Combined factored axial load and factored flexural moment

The factored axial load, P_u, and the factored flexural moment, M_u, which accompanies it and are caused by the factored loads applied to the structure, should be determined, for the particular element type, from the guides of 10 to 14.

11.2.3 Design strength for axial compression

11.2.3.1 Design strength for axial compression without flexure

Equation 101 should be used to determine the design axial strength for axial compression without flexure, $\phi \cdot P_{0n}$.

$$\phi \cdot P_{0n} = \phi \cdot \left[0{,}85 \cdot f_c' \cdot \left(A_g - A_{st}\right) + A_{st} \cdot f_y\right] \qquad \text{Equation (101)}$$

In Equation 101 ϕ = [0,70] for columns with ties and structural concrete walls, and ϕ = [0,75] for columns with spiral reinforcement.

11.2.3.2 Maximum design axial load strength

The design strength for axial load, $\phi \cdot P_n$, in columns and structural concrete walls subjected to compression, with or without flexure, should not be taken greater than the following:

Columns with ties and structural concrete walls:

$\phi \cdot P_{n(max)} \leq 0{,}80 \cdot \phi \cdot P_{0n}$ (with ϕ = [0,70]) Equation (102)

Columns with spiral reinforcement:

$\phi \cdot P_{n(max)} \leq 0{,}85 \cdot \phi \cdot P_{0n}$ (with ϕ = [0,75]) Equation (103)

11.2.4 Balanced strength for axial compression with flexure

11.2.4.1 Square and rectangular tied columns, and structural concrete walls

The values for axial force, $\phi \cdot P_{bn}$, and moment, $\phi \cdot M_{bn}$, at the balanced design strength point should be determined using Equation 104 and Equation 105 respectively. However these equations only apply to rectangular columns with symmetrical reinforcement.

$$\phi \cdot P_{bn} = \phi \cdot 0{,}42 \cdot f_c' \cdot h \cdot b \qquad \text{Equation (104)}$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0{,}32 \cdot h + \phi \cdot [0{,}6 \cdot A_{se} + 0{,}15 \cdot A_{ss}] \cdot f_y \cdot \left(\frac{h}{2} - d'\right) \qquad \text{Equation (105)}$$

For Equation 105 the total longitudinal reinforcement area, A_{st}, should be divided into extreme steel, A_{se}, and side steel, A_{ss}, in such a manner that $A_{se} + A_{ss} = A_{st}$. See Figure 80 In Eq. (87) and Eq. (88) ϕ = [0,70].

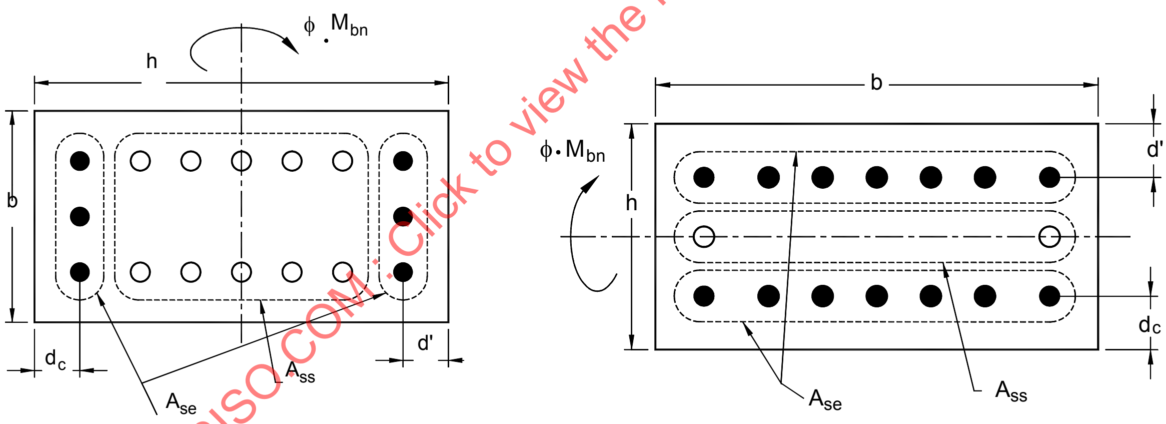


Figure 80 — Dimensions for calculation of balanced moment design strength

11.2.4.2 Circular section columns with spiral reinforcement

The values for axial force, $\phi \cdot P_{bn}$, and moment, $\phi \cdot M_{bn}$, at the balanced design strength point should be determined using Equation 106 and Equation 107 respectively:

$$\phi \cdot P_{bn} = \phi \cdot 0{,}5 \cdot f_c' \cdot A_c \qquad \text{Equation (106)}$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0{,}2 \cdot h + \phi \cdot 0{,}6 \cdot A_{st} \cdot f_y \cdot \left(\frac{h}{2} - d'\right) \qquad \text{Equation (107)}$$

Equation 107 h should be taken as the diameter of the section of the column. In Equation 106 and Equation 107 ϕ = [0,75].

11.2.5 Design strength for axial tension without flexure

The design strength for axial tension without flexure, $\phi \cdot P_{tn}$, should be determined using Equation 108: ϕ = [0,90].

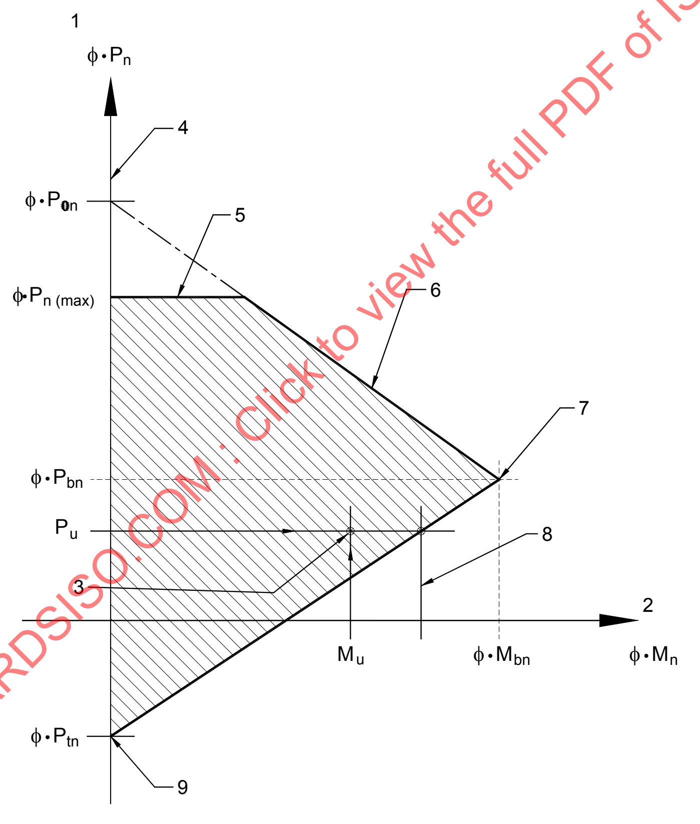
$$\phi \cdot P_{tn} = \phi \cdot A_{st} \cdot f_y \qquad \text{Equation (108)}$$

11.2.6 Design combined axial load and moment strength

The design moment strength at the section of the element, ($\phi \cdot M_n$), at the level of applied factored axial load, P_u, should be greater or equal than the greater factored flexural moment, M_u, that can accompany the factored axial load, P_u, as shown in Equation 109:

$$\phi \cdot M_n \geq M_u \qquad \text{Equation (109)}$$

The compliance with Equation 109 should be accomplished by proving that the coordinates of (M_u, P_u) in a moment vs. axial load interaction diagram relating $\phi \cdot M_n$ and $\phi \cdot P_n$, are inside the interaction design strength surface, shaded portion in Figure 81.



Key

1 axial load

2 moment

3 required factored axial load and moment

4 design strength for axial compression

5 maximum allowable axial compression load

6 interaction design strength surface

7 balance design strength point

8 design moment strength at factored axial load level, Pu

9 design strength for axial tension

Figure 81 — Interaction diagram for ($\phi \cdot$ Mn, $\phi \cdot$ Pn)

The following conditions should be met for all couples of P_u and M_u that act on the column section:

$$P_u \leq \phi \cdot P_{n(max)} \quad \text{Equation (110)}$$

$$P_u \geq -(\phi \cdot P_{tn}) \quad \text{Equation (111)}$$

For values of $P_u \geq \phi \cdot P_{bn}$:

$$M_u \leq \phi \cdot M_n = \frac{(\phi \cdot P_{0n}) - P_u}{(\phi \cdot P_{0n}) - (\phi \cdot P_{bn})} \cdot (\phi \cdot M_{bn}) \quad \text{Equation (112)}$$

For values of $P_u < \phi \cdot P_{bn}$:

$$M_u \leq \phi \cdot M_n = \frac{P_u + (\phi \cdot P_{tn})}{(\phi \cdot P_{bn}) + (\phi \cdot P_{tn})} \cdot (\phi \cdot M_{bn}) \quad \text{Equation (113)}$$

11.2.7 Use of interaction diagrams

It should be permitted to use interactions diagrams for columns from authoritative sources, if the employment of the strength reduction factors, as set forth in these guidelines is warranted.

11.2.8 Biaxial moment strength

Corner columns, and other columns subjected to moments about each axis simultaneously should comply with Equation 114:

$$\frac{(M_u)_x}{(\phi \cdot M_n)_x} + \frac{(M_u)_y}{(\phi \cdot M_n)_y} \leq 1{,}0 \quad \text{Equation (114)}$$

where $(M_u)_x$ and $(M_u)_y$ correspond to the factored moments that act about axis x and y, simultaneously with the factored axial load P_u. $(M_n)_x$ and $(M_n)_y$ correspond to the values of the design moment strength obtained from Equation 112 or Equation 113 for the factored axial load value P_u, and for the appropriate direction x or y.

11.2.9 Shear in structural concrete walls

11.2.9.1 General

The guides in 11.2.9 should be applied to the design of structural concrete walls for shear. The following general guides should be employed:

a) the design for shear forces perpendicular to the face of the structural concrete wall should be in accordance to the provisions for solid slabs in 11.2.7. The design for shear forces in the plane of the structural concrete wall should be performed following the guides of 11.2.9.

b) the structural concrete wall should be continuous for all the way down to the foundation and have no openings for windows or doors.

c) the structural concrete wall should have distributed reinforcement in the vertical and horizontal direction, not less than the minimum values of 9.6.5 and 9.7.5, and complying with the maximum spacing of 9.4.15.

d) where shear reinforcement is used the design shear strength, $\phi \cdot V_n$, should be computed using Equation 115.

$$\phi \cdot V_n = \phi \cdot (V_c + V_s)$$ Equation (115)

In Equation 115, $\phi \cdot V_c$ is the contribution of the concrete to the design shear strength, and $\phi \cdot V_s$ is the contribution of the reinforcement to the design shear strength. In Equation 115 ϕ = [0,85].

11.2.9.2 Contribution of concrete to shear strength in structural concrete walls

At each critical location to be investigated, only the contribution of the concrete of the web of the structural concrete wall should be taken into account and it should be computed using Equation 116 with ϕ = [0,85]

$$\phi \cdot V_c = \phi \cdot \left[\frac{\sqrt{f'_c}}{6}\right] \cdot b_w \cdot l_w$$ Equation (116)

where b_w is the thickness of the web of the structural concrete wall, and l_w its horizontal length.

11.2.9.3 Shear reinforcement in structural concrete walls

The contribution to the design shear strength of the horizontal reinforcement located in the web of the structural concrete wall should be:

$$\phi \cdot V_s = \phi \cdot \lfloor \rho_h \cdot f_y \cdot b_w \cdot l_w \rfloor$$ Equation (117)

where ρ_h is the ratio of horizontal reinforcement and f_y its yield strength. In Equation 117 ϕ = [0,85].

11.2.9.4 Design of shear reinforcement

Where the factored shear, V_u exceed $\phi \cdot V_c$, the ratio of horizontal reinforcement should not be less than the amount determined from Equation 118, with ϕ = [0,85]:

$$\rho_h \geq \frac{V_u - \phi \cdot V_c}{\phi \cdot f_y \cdot b_w \cdot l_w}$$ Equation (118)

In addition the following requirements should be met:

a) two curtains of reinforcement should be employed, both in vertical and horizontal reinforcement,

b) if $\frac{h_w}{l_w}$ is less than 2, the vertical steel ratio, ρ_v, should not be less than the horizontal steel ratio, ρ_h.

c) The value of $\phi \cdot$ Vn should not exceed the value given by Equation 119.

$$\phi \cdot V_n = \phi \cdot (V_c + V_s) \leq \phi \cdot \left[\frac{5}{6}\right] \cdot \sqrt{f'_c} \cdot b_w \cdot l_w$$ Equation (119)

11.3 Torsion

Design for torsion is beyond the scope of the present guidelines, and it should be permitted to neglect torsion effects when the calculated factored torsion, T_u, is less than the value obtained from Equation 120:

$$T_u \leq \phi \cdot \left[\frac{\sqrt{f'_c}}{24}\right] \cdot \left[\frac{h^2 \cdot b^2}{h+b}\right]$$ Equation (120)

Notwithstanding, in members where torsion smaller than the value given by Equation 120 is present, closed stirrups with a minimum bar diameter of 10 mm, should be provided near the supports with a spacing measured along the length of the element not greater than b/4 or d/4, the smaller, for a distance equal to 1/4 of the clear span of the element measured from the internal face of each support. In Equation 120 ϕ = [0,85].

11.4 Bearing strength

The factored compression normal load, P_u, applied concentrically on an area, A_c, should not exceed the design bearing strength on concrete ($\phi \cdot P_n$) obtained using Equation 121:

$$\phi \cdot P_n = \phi \cdot 0{,}85 \cdot f'_c \cdot A_c \qquad \text{Equation (121)}$$

where A_c corresponds to the contact area in mm^2, and ϕ = [0,70].

11.5 Columns and Piers

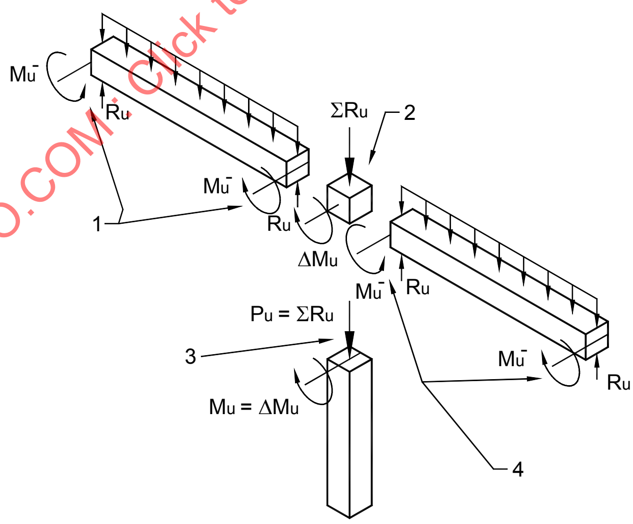
11.5.1 General

The design of columns should be performed using the guides of present 11.5. The members covered by this subclause are members reinforced with longitudinal bars and lateral ties, and members reinforced with longitudinal bars and continuous spiral. Both rectangular and circular sections are covered.

11.5.2 Design load definition

11.5.2.1 Loads to be included

The design load for columns belonging to frames or slab-column systems should be established from the tributary loads from each deck located above the column, plus the selfweight of the column. Tributary loads should be established from the guides of chapter 8 and the particular guides of each tributary element type. See Figure 82.



Key

1 reactions at the ends of the element

2 actions at the joint

3 loads applied to the top of the column

4 reactions at the ends of the element

Figure 82 — Column factored design forces from one story and in one direction

11.5.2.2 Dead load and live load

The values of P_d for dead load and P_l for live load should be in N. P_d should include the selfweight of the column, assuming concrete unit weight as 25 x 10^3 N/m^3. The selfweight should be factored employing the load factors for dead load of the corresponding combination equation from 8.9.1. It should be permitted to apply the selfweight of the column corresponding to each deck at the lower part of the column in that deck.

11.5.2.3 Factored design forces

The value of the factored design forces P_u and M_u should be established for the column at the upper and lower part of the column in each story. A distinction should be made about the direction of the axis in plan along which the moments M_{ux} and M_{uy} act.

11.5.3 Dimensional additional specifications

11.5.3.1 General

The specification set forth in the present subclause, are in addition to the general Dimensional specifications set forth in 6.1. Columns should be aligned vertically, without eccentricity between upper and lower columns, and should be continuous all the way down to the foundation. Column section shape should be either rectangular or circular. All other cross-section shapes are beyond the scope of these guidelines.

11.5.3.2 Limiting section dimensions

11.5.3.2.1 Minimum section dimensions for rectangular columns

Under the present guidelines, section dimension for rectangular columns should comply with the following limits (see Figure 83):

a) The shortest cross-sectional dimension should not be less than 300 mm.

b) The ratio of the largest cross-sectional dimension to the perpendicular shortest dimension should not exceed 3, except that for columns in bridges located in seismic risk zones, this ratio should comply with 13.5.3.1.

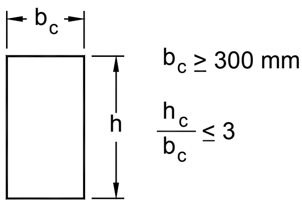


Figure 83 — Minimum cross-section dimensions for rectangular columns

11.5.3.2.2 Minimum section dimensions for circular columns

Columns with circular cross-section should have a diameter of at least 300 mm.

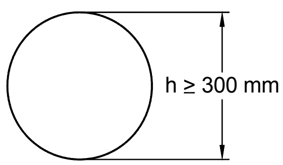
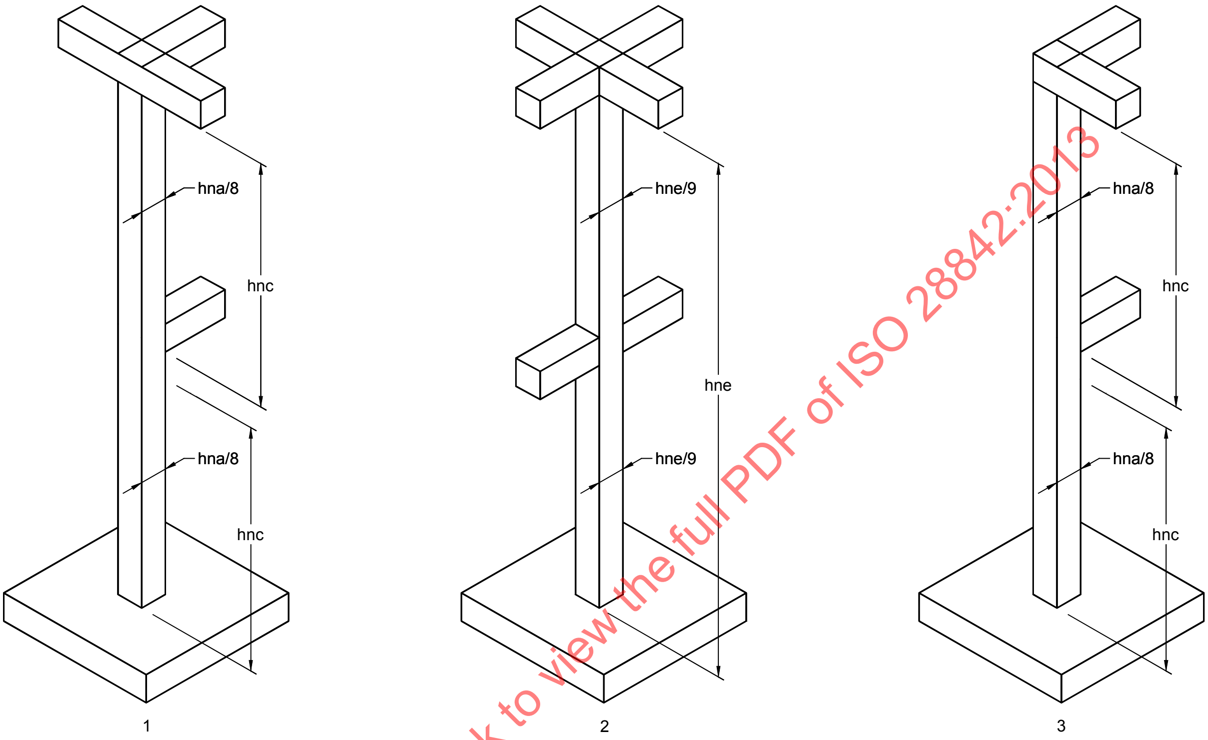


Figure 84 — Minimum cross-section dimension for circular columns

11.5.3.3 Distance between lateral supports

11.5.3.3.1 General

It should be considered that lateral restraint is provided by the deck system in the two horizontal directions at all levels that are supported by the column. See Figure 85.



Key

1 edge

2 central

3 corner

Figure 85 — Lateral restraint for columns

11.5.3.3.2 Central columns

The clear distance between lateral supports, h_n, for central columns should not exceed 10 times the dimension of the column cross-section parallel to the direction of the support. See Figure 85.

11.5.3.3.3 Edge columns

The clear distance between lateral supports, h_n, perpendicular to an edge for edge columns should not exceed 9 times the dimension of the column cross-section perpendicular to the edge. See Figure 85.

11.5.3.3.4 Corner columns

The clear distance between lateral supports, h_n, for corner columns should not exceed 8 times the minimum dimension of the column cross-section. See Figure 85.

11.5.4 Details of reinforcement

11.5.4.1 General

For the purposes of these guidelines, the reinforcement of columns should be of the types described in this subclause and should comply with the guides of 11.5.4.2 to 11.5.4.4.

11.5.4.2 Longitudinal reinforcement

11.5.4.2.1 Description and location

Longitudinal reinforcement should be provided in the periphery of the column section, as guide in 9.6.4.4. Longitudinal reinforcement should be located as close as concrete cover guides permit (see 9.4.4.1 and 11.5.4.2.9) to the lateral surfaces of the column. The amount of longitudinal reinforcement should be that guide to resist the simultaneous action of a combination of factored axial load and factored moments at the section acting about the two main axis of the section of the column. See Figure 86.

11.5.4.2.2 Minimum and maximum longitudinal reinforcement area

The maximum and minimum longitudinal reinforcement area should comply with the guides of 9.6.4.1 ($0,01 \leq \rho_t \leq 0,06$).The maximum longitudinal reinforcement area is also limited by the beam reinforcement in the beam – column joint.

11.5.4.2.3 Minimum diameter of longitudinal bars

Longitudinal bars of columns should comply with the minimum guide nominal diameter, d_b, as set forth in 9.6.4.2 (16 mm).

11.5.4.2.4 Minimum number of longitudinal bars

The minimum number of longitudinal bars in rectangular and round columns should be as set forth in 9.6.4.3 (4 bars in rectangular columns or 6 in circular columns).

11.5.4.2.5 Minimum and maximum reinforcement spacing

Longitudinal reinforcement should not be spaced closer than guide by 9.4.11 (1,5 d_b or 40 mm).

11.5.4.2.6 Reinforcement splicing

It should be permitted to lap-splice up to one-half the longitudinal reinforcement at any given section, as long as only alternate bars are lap-spliced. See Figure 86. All lap splices of longitudinal reinforcement should comply with 9.5.2.1. ["alternative methods like gas pressure welding or mechanical connectors could be used taking account of the practical situation of each country"]

11.5.4.2.7 End anchorage of reinforcement

Longitudinal reinforcement at the upper end of the columns, and at the foundation elements that transmits the loads to the underlying soil should extend to the extreme and end with a standard hook.

11.5.4.2.8 Longitudinal bar offset

Offset bent longitudinal bars should conform to the following:

a) Slope of inclined portion of an offset bar with axis of column should not exceed 1 in 6.

b) Portions of bar above and below an offset should be parallel to axis of column.

c) Horizontal support at offset bends should be provided by lateral ties or spirals.

d) Horizontal support provided should be designed to resist 1,5 times the horizontal component of the computed force in the inclined portion of an offset bar.

e) Lateral ties or spirals should be placed not more than 150 mm from points of bend.

f) Offset bars should be bent before placement in the forms.

g) Where a column face is offset from the face of the column below more than 1/6 of the depth of the girder or slab, or 80 mm, longitudinal bars should not be offset bent. Separate dowels, lap spliced with the longitudinal bars adjacent to the offset column faces should be provided. Lap splices should conform to 9.5.2.1.

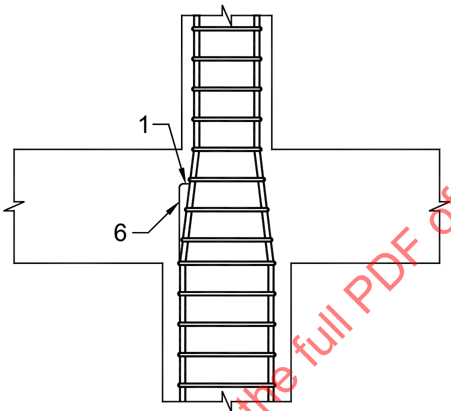


Figure 86 — Longitudinal bar offset

11.5.4.2.9 Maximum number of longitudinal bars per face of rectangular column

The maximum number of longitudinal bars in a layer should be determined for the longitudinal and transverse reinforcement bar diameters, the appropriate concrete cover, the maximum nominal coarse aggregate size, and the minimum clear spacing between bars (see 9.4.10). When these computations are not performed, it should be permitted to employ the following guides:

a) For columns section dimension under study, b_c, greater or equal to 400 mm it should be permitted to determine the maximum number of bars in a layer employing Equation 122, where b_c is the column dimension under study in mm. See Table 28.

$$\text{No. of bars per face} \leq \frac{b_c}{68} - 1 \qquad \text{Equation (122)}$$

b) Three longitudinal bars should be permitted in the face of columns whose dimension under study, b_c, is less than 400 mm and greater or equal to 300 mm. See Table 28.

Table 28 — Maximum number of longitudinal bars per face of rectangular column

column dimension b_c (mm)	Maximum number of longitudinal bars
$b_c < 300$ mm	section not permitted
$300 \text{ mm} \le b_c < 400$ mm	3 bars
$400 \text{ mm} \le b_c$	$\le \left(\frac{b_c}{68} - 1\right)$ bars

11.5.4.2.10 Maximum number of longitudinal bars in circular columns

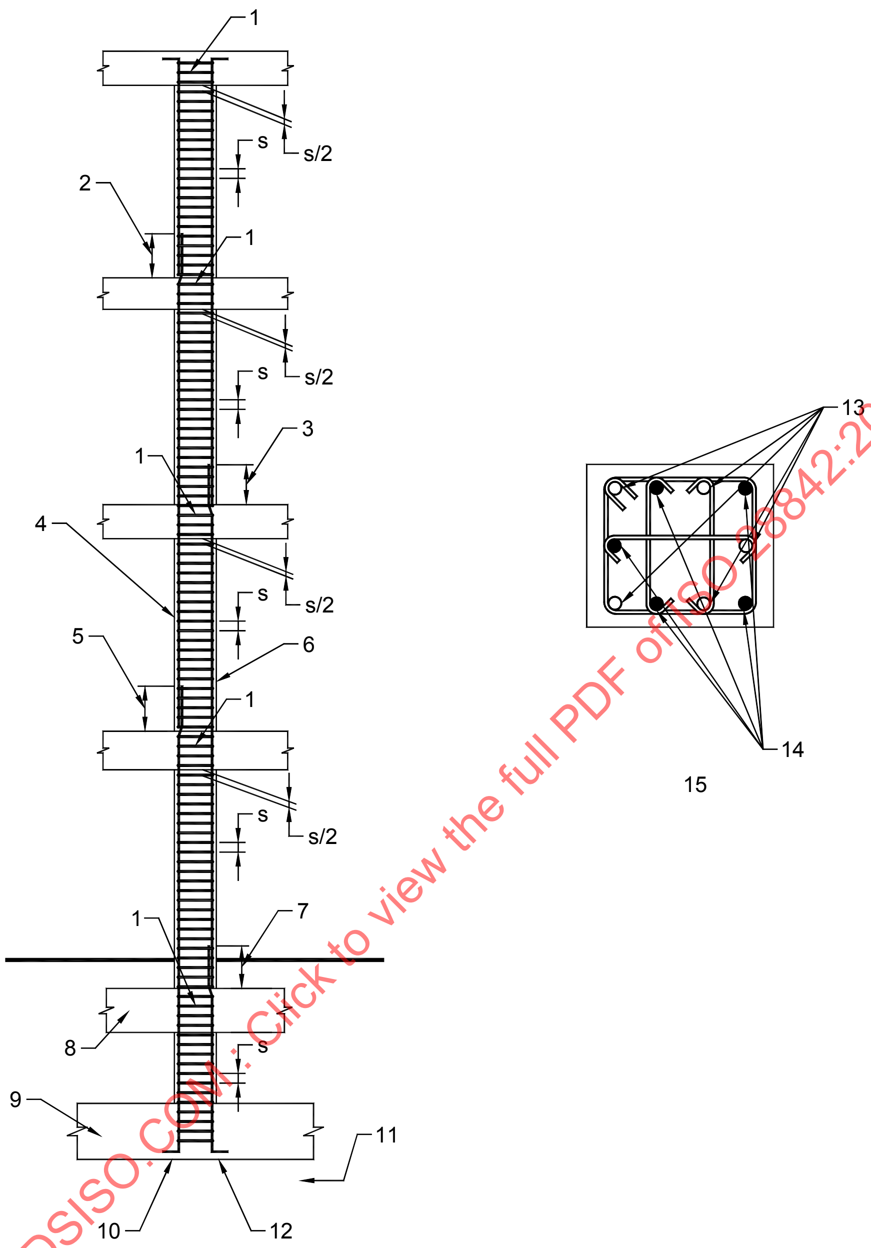
The maximum number of longitudinal bars in circular columns should be determined for the longitudinal and transverse reinforcement bar diameters, the appropriate concrete cover, the maximum nominal coarse aggregate size, and the minimum clear spacing between bars (see 9.4.10). When these computations are not performed, it should be permitted to determine the maximum number of bars employing Equation 123, where h is the column diameter in mm. See Table 29.

$$\text{No. of bars} \le \frac{h}{22} - 6 \qquad \text{Equation (123)}$$

Table 29 — Maximum number of longitudinal bars in circular columns

column diameter h (mm)	Maximum number of longitudinal bars
$h < 300$ mm	section not permitted
$300 \text{ mm} \le h$	$\le \left(\frac{h}{22} - 6\right)$ bars

Figure 87 shows a typical column reinforcement layout.



Key

1	joint ties	9	foundation element
2	lap splice as guide in 9.5.2	10	type A long. bars
3	lap splice as guide in 9.5.2	11	bearing soil
4	type A long. bars	12	type B long. bars
5	lap splice as guide in 9.5.2	13	type A
6	type B long. bars	14	type B
7	lap splice as guide in 9.5.2	15	column cross-section
8	grade beam		

Figure 87 — Typical column reinforcement layout

11.5.4.3 Transverse reinforcement

11.5.4.3.1 General

Transverse reinforcement in the form either of tie reinforcement, complying with 9.7.4.1, or spiral reinforcement, complying with 9.7.4.2, should be provided in all columns. At column-beam joints a minimum amount of ties, as guide by 9.7.4.3 should be placed. It should not be permitted to lap-splice bars that are part of column ties.

11.5.4.3.2 Maximum and minimum tie and spiral spacing

The maximum spacing along the axis of the column of the ties or spiral should be as indicated in 9.7.4.1 or 9.7.4.2, respectively. Tie and spirals should not be spaced closer than guide by 9.4.9.

11.5.4.3.3 Tie hooks

Under the present guidelines all ties of columns should have 135° hooks (see 9.4.6). It should be permitted to employ crossties with a 135° hook in one end and a 90° in the other end, and consecutive crossties engaging the same longitudinal bar should have their 90° hooks at opposite sides of the flexural member. (Referring flexural members to the columns).

11.5.4.4 Column reinforcement in seismic zones

In columns that are part of a moment resisting frame located in seismic zones, reinforcement should comply with the additional guides of 13. Columns that are part of slab-column frames in seismic zones should comply with the additional guides of 13.

11.5.5 Flexural additional specifications

11.5.5.1 Guide factored loads

The guide factored axial load, P_u, and moment, M_u, at section under study should be obtained following the requirements of 11.5.2.

11.5.5.2 Initial trial cross-section dimensions and longitudinal reinforcement

11.5.5.2.1 Trial cross-section dimensions

It should be permitted to establish initial trial cross-section dimensions in the following manner:

a) First trial gross cross-sectional area A_g should be obtained from Equation 124.

$$A_g \approx \frac{2 \cdot (P_u)_{max.}}{f'_c} \qquad \text{Equation (124)}$$

b) For rectangular cross-sections least dimension, b, should comply with:

$$b \begin{cases} \geq 300\ \text{mm} \\ \geq h/3 \\ \geq h_n/10 \quad \text{for central columns} \\ \geq h_n/9 \quad \text{for edge columns} \\ \geq h_n/8 \quad \text{for corner columns} \end{cases} \qquad \text{Equation (125)}$$

c) For rectangular cross-sections larger dimension, h, should comply with:

$$h\begin{cases}\geq 300\text{ mm}\\ \leq 3\cdot b\\ \geq h_n/10 \text{ for central columns}\\ \geq h_n/9 \text{ for edge columns}\\ \geq h_n/8 \text{ for corner columns}\end{cases}$$ Equation (126)

d) For circular columns diameter, h, should comply with:

$$h\begin{cases}\geq 300\text{ mm}\\ \geq h_n/10 \text{ for central columns}\\ \geq h_n/9 \text{ for edge columns}\\ \geq h_n/8 \text{ for corner columns}\end{cases}$$ Equation (127)

11.5.5.2.2 Trial longitudinal reinforcement

It should be permitted to establish initial trial area of longitudinal reinforcement, A_{st}, in the following manner:

a) For rectangular cross-sections the first trial area of longitudinal reinforcement, A_{st}, should comply with:

$$A_{st} \geq \begin{cases}0{,}01\cdot A_g\\ 4\cdot (A_b)_{min.}\end{cases}$$ Equation (128)

b) For circular cross-sections the first trial area of longitudinal reinforcement, A_{st}, should comply with:

$$A_{st} \geq \begin{cases}0{,}01\cdot A_g\\ 6\cdot (A_b)_{min.}\end{cases}$$ Equation (129)

11.5.5.3 Factored flexural moment verification

Interaction diagrams for the column dimensions and reinforcement should be calculated in both directions using the guides of 11.2.2 to 11.2.5. The design moment in both directions should be verified employing the guides of 11.1.6. If the factored flexural moment, M_u, at required factored axial load, P_u, exceeds the design moment strength at axial load level P_u, the area of longitudinal reinforcement should be increased, without exceeding the maximum reinforcement area permitted by 9.6.4.1 or the maximum number of bars in the face of column of 11.5.4.2.10. If an increase of the column dimensions is required because these limits are exceeded, the columns selfweight should be corrected, and the column should be verified for the new dimensions. These verifications should be performed at the upper and lower sections of the column of the same story.

11.5.5.4 Biaxial moment strength verification

Once the column is verified for both directions independently, the biaxial design moment should be verified employing the guides of 11.2.8 at the upper and lower sections of the column of the same story.

11.5.6 Shear additional specifications

11.5.6.1 Factored shear

The factored shear, V_u, should be determined from the vertical loads, and from the horizontal loads. The value should be that obtained from the appropriate load combinations from 8.9.1.

11.5.6.1.1 Factored shear from vertical loads

The factored shear caused by the vertical loads should be determined from Equation 130 for each direction:

$$V_u = \frac{(M_u)_{top} + (M_u)_{bottom}}{h_n}$$ Equation (130)

where $(M_u)_{top}$ corresponds to the factored moment at the upper end of the column, $(M_u)_{bottom}$ to the factored moment at the lower end of the columns, and h_n is the clear distance between lateral supports of the column.

11.5.6.1.2 Factored shear from horizontal loads

The factored shear, Vu, caused by horizontal loads should be determined from the horizontal loads prescribed in 8, employing the appropriate load combinations from 8.9.1.

11.5.6.2 Shear strength verification

The shear strength verification should be performed for beam-action shear employing the guides of 11.2.9.1 and 11.2.9.2. The contribution of concrete to shear strength should be evaluated using Equation 33. The contribution of the transverse reinforcement of the column should be determined in the direction under study using Equation 34 where A_v corresponds to the area of the tie legs parallel to the shear, and s to the larger tie spacing within the clear height of the column. The verification in the direction under study should be performed employing Equation 31 and Equation 32. If Equation 31 is not met, the tie spacing s should be reduced.

11.5.6.3 Biaxial shear strength verification

When the column is subjected to shear in the direction of each axis simultaneously, it should comply with Equation 131:

$$\sqrt{\left[\frac{(V_u)_x}{(\phi \cdot V_n)_x}\right]^2 + \left[\frac{(V_u)_y}{(\phi \cdot V_n)_y}\right]^2} \leq 1{,}0$$ Equation (131)

where $(V_u)_x$ and $(V_u)_y$ correspond to the factored shear that act in the direction of axis x and y, and $(\phi \cdot V_n)_x$ and $(\phi \cdot V_n)_y$ correspond to the values of the design shear strength obtained from Equation 130 for the appropriate direction x or y.

11.6 Concrete walls

11.6.1 General

The design of structural concrete walls should be performed using the guides of present 11.6 Both in-plane and out-of-plane effects on reinforced concrete structural walls are covered.

11.6.2 Design load definition

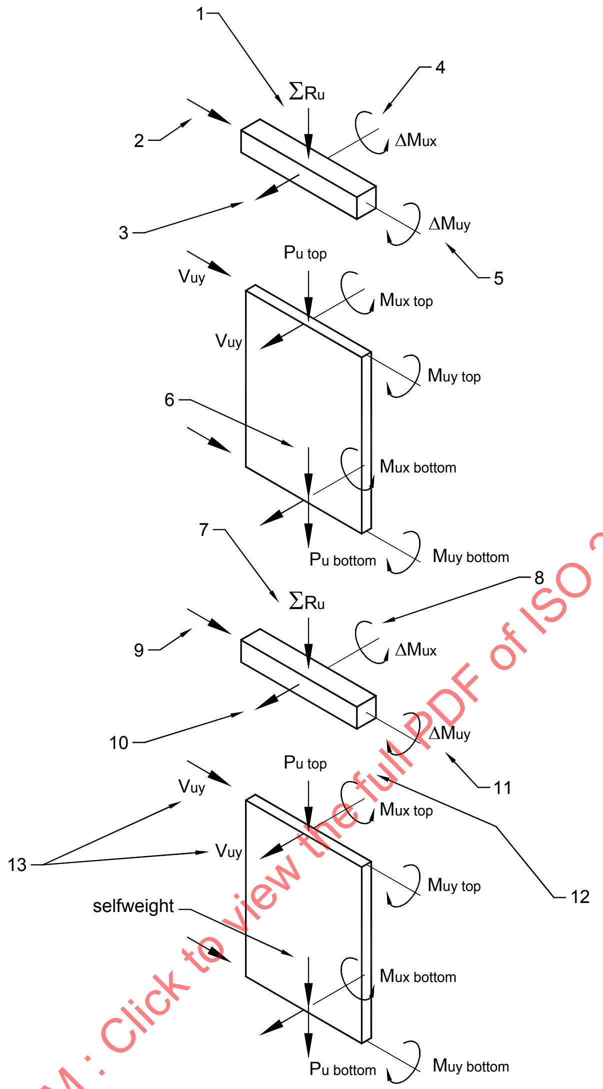
11.6.2.1 Loads to be included

The design load for structural concrete walls should be established from the guides of 8. The loads that should be included in the design are (See Figure 88):

a) Tributary live and dead loads from the tributary structural elements from each deck located above. Tributary loads should be established from the guides of 8 and the particular guides of each tributary element type.

b) Selfweigth of the structural concrete wall.

c) Lateral forces from wind, earthquake or soil lateral pressures.



Key

1 actions at the joint of story n from tributary elements

2 lateral force applied to the wall at story n in direction x

3 lateral force applied to the wall at story n in direction y

4 unbalanced moment from tributary story elements in direction x

5 unbalanced moment from tributary story elements in direction y

6 selfweight

7 actions at the joint of story n-1 from tributary

8 unbalanced moment from tributary story elements in direction x

9 lateral force applied to the wall at story n-1 in direction x

10 lateral force applied to the wall at story n-1 in direction y

11 unbalanced moment from tributary story elements in direction y

12 Pu top of story n-1 wall equal to P u bottom from story n plus ΣRu from story n-1

13 Vu of story n-1 wall equal to Vu from story n plus lateral force from story n

Figure 88 — Structural concrete wall applied factored design forces

11.6.2.2 Dead load and live load

The values of P_d for dead load and P_l for live load should be in N. P_d should include the selfweight of the structural concrete wall, at 25 x 10^3 N/m^3. The selfweight should be factored employing the load factors for dead load of the corresponding combination equation from 8.9.1. It should be permitted to apply the selfweight of the wall corresponding to each deck at the lower part of the structural concrete wall in that deck. The value of the unbalanced moment caused by vertical loads should be obtained from the guides of the supported element. See 11.1.6.

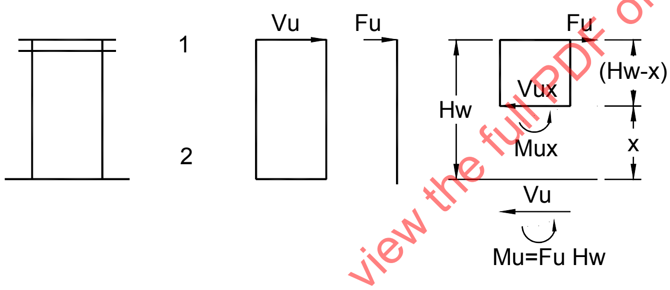
11.6.2.3 Lateral design load

The value of the applied factored horizontal design shear, V_u, should be obtained from the guides of 13. The value of the factored lateral load moment, M_u, should be established in the following manner:

a) The lateral load factored shear at x, V_{xu}, should be obtained for the wall as per section 13.

b) Factored lateral load moment, M_{xu}, at any height x should be obtained employing Equation 132.

$$M_{xu} = F_U \cdot (H_U - x) \qquad \text{Equation (132)}$$



Key

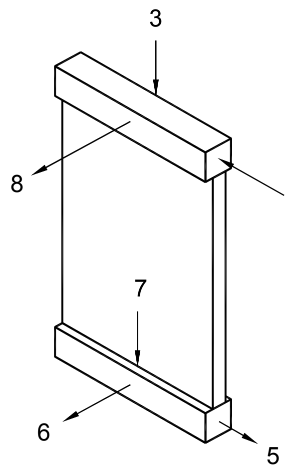
1 deck level

2 foundation level

Figure 89 — Calculation of the lateral load factored moment

11.6.2.4 Factored design load

The value of the factored design forces P_u, V_u, and M_u should be established for the structural concrete wall, for both inplane and out of plane forces. See Figure 90. P_u at the top of the wall is the portion of the total weight of the superstructure which is transferred to the wall. In addition to his, P_u at the bottom of the wall includes the self weight of the wall. V_u acting in the plane of the wall is the lateral force transferred from the superstructure to the wall due to wind and earthquake loads. V_u acting out of the plane of the wall is the force transferred from the superstructure to the wall due to thermal changes.



Key

3 axial force

4 in plane shear

5 out of plane moment

6 in plane moment

7 selfweight

8 out of plane shear

Figure 90 — In-plane and out-of-plane forces

11.6.3 Dimensional additional specifications

11.6.3.1 General

In addition to the appropriate specifications of the present subclause, structural concrete walls should comply with the general dimensional specifications set forth in 6.1 and 13.4.1. Structural concrete wall section shape should be rectangular. All other cross-section shapes are beyond the scope of these guidelines, with the exception permitted by 11.6.3.3.1. Structural concrete walls should be continuous all the way down to the foundation.

11.6.3.2 Limiting dimensions

11.6.3.2.1 Minimum thickness structural concrete walls

Under the present guidelines, the thickness of structural concrete walls should not be less than 150 mm (see Figure 91) nor 1/25 of the length of the wall l_w, and at changes of thickness in contiguous stories, the guides of 13.4.1 c) should be met.

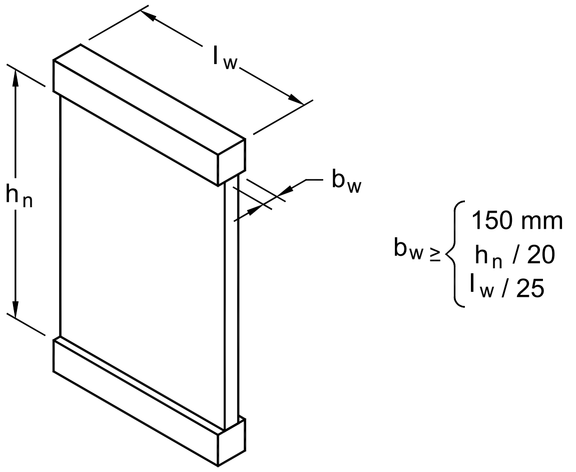


Figure 91 — Minimum cross-section dimensions for rectangular structural concrete walls

11.6.3.3 Lateral supports

It should be considered that lateral restraint is provided to the wall by the deck system and by the foundation system in the two horizontal directions. To avoid the risk of wall buckling, the thickness of the structural concrete wall shold not be less than $\frac{1}{20}$ of its total height.

11.6.3.3.1 Columns embedded in walls

Columns may be built monolithically embedded in walls to avoid buckling, without having to increase the thickness of the whole wall cross-section. The transverse dimension of the column should be calculated as per in Equation 133 and Equation 134.

For Rectangular Columns

$$bcw = \sqrt[4]{3.1\frac{Pu \cdot \alpha \cdot Hw^2}{E}} \qquad \text{Equation (133)}$$

Where $\alpha = \frac{bcw}{Lcw}$, bcw, Lcw and Hw are shown in Figure 94

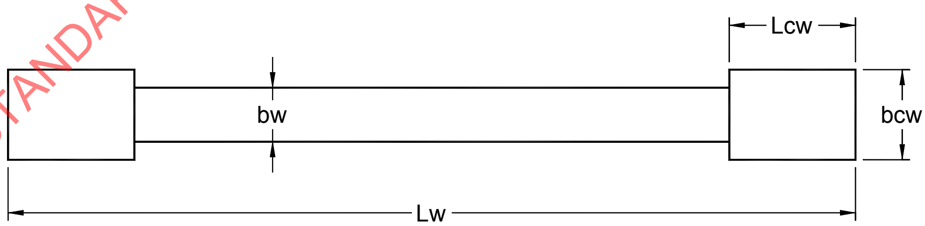


Figure 92 — Rectangular Columns Embedded in walls

For Circular Columns

$$rc = \sqrt[4]{0.33\frac{Pu \cdot Hw^2}{E}} \qquad \text{Equation (134)}$$

Where rc and Hw are shown in Figure 93

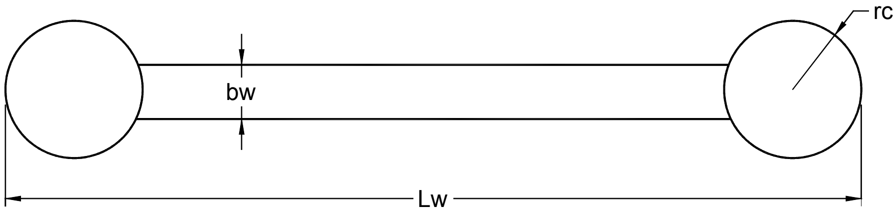


Figure 93 — Circular Columns Embedded in walls

11.6.3.4 Beams on top of walls

Beams or girders should be provided for the full horizontal length of the wall at every deck supported by the structural wall. These beams or girders should comply with the guides of 11.1.2.3, and should be reinforced as collector elements following the guides of 10.5.4.3.

11.6.4 Details of reinforcement

11.6.4.1 General

For the purposes of these guidelines, the reinforcement of structural concrete walls should be of the types described in this subclause and should comply with the guides of 11.6.4.2 to 11.6.4.4.

11.6.4.2 Number of curtains of reinforcement

11.6.4.2.1 Two curtains of reinforcement

Two curtains of reinforcement parallel with the faces of the wall should be employed in the following cases:

a) When the wall is more than 250 mm thick.

b) In walls where the vertical reinforcement ratio, ρ_v, exceeds 0,01. See 9.6.5.2 and 11.6.4.3.2.

c) In walls where the in-plane factored shear force, V_u, in the wall exceeds ($\phi \cdot V_c$) as given by Equation 116.

The division of reinforcement into layers, and their location within the wall section should comply with 9.4.15.2.

11.6.4.2.2 One curtain of reinforcement

In all the other cases not covered by 11.6.4.2.1 it should be permitted to employ only one curtain of reinforcement located in the center of the thickness of the wall.

11.6.4.3 Vertical reinforcement

11.6.4.3.1 Description

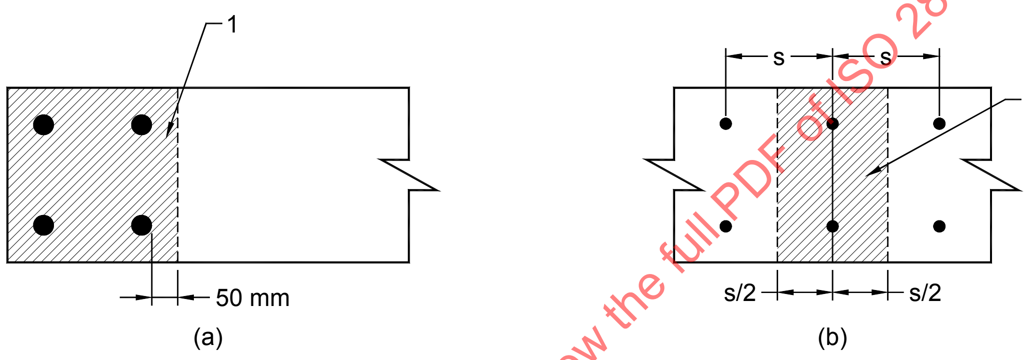
Vertical reinforcement should consist in one or two layers of bars or welded-wire fabric placed parallel with the faces of the walls. The amount of vertical reinforcement should be that required to resist the simultaneous action of a combination of factored axial load and factored moments at the section acting about the two main axis of the section of the structural concrete wall.

11.6.4.3.2 Minimum and maximum vertical reinforcement area

The maximum and minimum vertical reinforcement area should comply with the guides of 9.6.5. When the amount and separation of vertical reinforcement vary within the wall cross-section, or columns are built monolithically embedded within the wall cross-section, the following guides should be met:

a) Where the vertical reinforcement is concentrated either by increasing the vertical bar diameter or reducing the spacing between bars, the vertical reinforcement ratio, ρ_v, in that portion of the wall should not exceed maximum vertical reinforcement ratio set forth by 9.6.5.2. The vertical reinforcement ratio should be evaluated over an area bounded by the faces of the wall and 50 mm measured along the length of wall from the last bars with a closer spacing or larger diameter. See Figure 94 a).

b) Where the vertical reinforcement is reduced either by separating it further apart or by decreasing the vertical bar diameter, the vertical reinforcement ratio, ρ_v, should not be less than the minimum vertical reinforcement ratio set forth by 9.6.5.1 at any place within the wall cross-section. See Figure 94 b).



Key

1 area for computation of the steel ratio

2 area for computation of the steel ratio

Figure 94 — Computation of the vertical reinforcement ratio

11.6.4.3.3 Maximum reinforcement separation

Vertical reinforcement should not be spaced further apart than guide by 9.4.15.

11.6.4.3.4 Reinforcement splicing

Lap splices of vertical wall reinforcement should comply with the lap splice length of 9.5.2. It should be permitted to lap-splice all the vertical reinforcement at any given section, except at the supported element of the deck system.

11.6.4.3.5 End anchorage of reinforcement

Vertical reinforcement at the upper end of the structural concrete walls, and at the foundation elements that transmit the loads to the underlying soil should extend to the extreme and end with a standard hook.

11.6.4.4 Horizontal reinforcement

11.6.4.4.1 General

Horizontal reinforcement should consist in one or two layers of bars or welded-wire fabric placed parallel with the faces of the walls, and under the circumstances described in 11.6.4.3.2. Transverse reinforcement, as in columns, should be provided. The amount of horizontal reinforcement should be that required to resist the factored in-plane shear at the section of the structural concrete wall.

11.6.4.4.2 Walls with transverse reinforcement as in columns

Where the vertical reinforcement ratio, ρ_v, exceeds 0,01, vertical reinforcement should be enclosed by ties complying with the guides for column tie transverse reinforcement as guide by 9.7.4.1. See 9.6.5.2 and 11.6.4.3.2 a). The vertical spacing of this tie reinforcement should meet the guides for columns.

11.6.4.4.3 Minimum horizontal reinforcement area

The minimum horizontal reinforcement area should comply with the guides of 9.7.5.

11.6.4.4.4 Maximum horizontal reinforcement spacing

The maximum vertical spacing of the horizontal reinforcement should be as indicated in 9.4.15.1.

11.6.4.4.5 Reinforcement splicing

It should be permitted to lap-splice the horizontal reinforcement complying with the lap splice length of 9.5.2.

11.6.4.4.6 End anchorage of reinforcement

Horizontal reinforcement terminating at the edges of structural walls should have a standard hook engaging the edge vertical reinforcement, or should have U-shaped stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

11.6.4.5 Structural concrete wall reinforcement in seismic zones

Structural concrete walls that are part of the lateral load resisting system in seismic zones, reinforcement should comply with the additional guides of 13.

11.6.5 Flexural additional specifications

11.6.5.1 Required factored loads

The required factored axial load, P_u, and moment, M_u, at section under study should be obtained following the requirements of 11.6.2.

11.6.5.2 Initial trial vertical reinforcement

It should be permitted to establish initial trial area of vertical reinforcement, A_{st}, employing the minimum vertical reinforcement ratio of 9.6.5.1.

11.6.5.3 Required factored moment strength verification

Interaction diagrams for the structural concrete wall dimensions and reinforcement should be calculated in both directions using the guides of 11.2.2 to 11.2.5. The total vertical reinforcement area, A_{st}, should be divided into total extreme steel area, A_{se}, and total side steel area, A_{ss}, for the direction under study, as guide by 11.2.4.1. The design moment in both directions should be verified employing the guides of 11.1.6. If the

