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Guidelines for the simplified design of structural reinforced concrete for buildings

Lignes directrices pour la conception simplifiée du béton armé pour les structures de bâtiments

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ISO copyright office
Case postale 56 • CH-1211 Geneva 20
Tel. + 41 22 749 01 11
Fax + 41 22 749 09 47
E-mail copyright@iso.org
Web www.iso.org

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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.

International Standards are drafted in accordance with the rules given in the ISO/IEC Directives, Part 2.

The main task of technical committees is to prepare International Standards. Draft International Standards adopted by the technical committees are circulated to the member bodies for voting. Publication as an International Standard requires approval by at least 75 % of the member bodies casting a vote.

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.

ISO 15673 was prepared by Technical Committee 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 low-rise concrete structures of small floor area to be built in the less developed areas of the world. The document is developed for countries that do not have existing national standards. This document shall not be used in place of a national standard unless specifically considered and accepted by the national standard body or other appropriate regulatory organization. The design rules are based in simplified worldwide-accepted strength models. The document is self-contained; therefore actions (loads) and simplified analysis procedures are included, as well as minimum acceptable construction practice guidelines.

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

The earthquake-resistance provisions are included to account for the fact that numerous underdeveloped regions of the world occur in earthquake-prone areas. The earthquake resistance 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.

The document contains provisions that can be modified by the national standards body due to local design and construction requirements and practices. The specifications that can be modified are indicated 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 the document and provide foolproof procedures. Notwithstanding, the economic implications of the conservatism inherent in approximate procedures as a substitution to sound and experienced engineering should be a matter of concern to the designer who employs the document, and to the owner who hires him.

Guidelines for the simplified design of structural reinforced concrete for buildings

1 Scope

This International Standard applies to the planning, design and construction of structural reinforced concrete structures to be used in new low-rise buildings with restricted occupancy, number of stories, and area. The purpose of this International Standard is to provide a registered civil engineer or architect with sufficient information to design the reinforced-concrete structural framing of a low-rise building that complies with these limitations; see 6.1. The rules of design as set forth in the present document are simplifications of the more elaborate requirements.

This document may be used as an alternative to the development of a national concrete building code, or equivalent document, in countries where no national design codes themselves are available, or as an alternative to the national concrete building code in countries where it is specifically considered and accepted by the national standard body or other appropriate regulatory organization.

Although the provisions contained in this document were established to produce, when properly employed, a reinforced concrete structure with an appropriate margin of safety, this International Standard is not a substitute for sound and experienced engineering. In order for the resulting structure designed in accordance with these provisions to attain the intended margin of safety, the document must be used as a whole, and alternative procedures should be employed only when explicitly permitted by the provisions. The minimum dimensional provisions as prescribed in the document replace, in most cases, more elaborate procedures such as those prescribed in the national building code, and an eventual economic impact is realized from the simplicity of the procedures prescribed.

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

2 Normative references

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

ISO 679, *Methods of testing cements — Determination of strength*

ISO 680, *Cement — Test methods — Chemical analysis*

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

ISO 2103, *Loads due to use and occupancy in residential and public buildings*

ISO 2633, *Determination of imposed floor loads in production buildings and warehouses*

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

ISO/TR 3956, *Principles of structural fire-engineering design with special regard to the connection between real fire exposure and the heating conditions of the standard fire-resistance test (ISO 834)*

ISO 4354, *Wind actions on structures*

ISO 4355, *Bases for design of structures — Determination of snow loads on roofs*

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 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 6935-3:1992, (as amended in 2000), *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, *Cements — Test methods — Determination of setting time and soundness*

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

3 Terms and definitions

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

3.1
acceleration of gravity
g
acceleration produced by gravity at the surface of earth

NOTE For the purposes of this International Standard, its value can be approximated as $g \approx [10]$ m/s^2.

3.2
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

3.3
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

3.4
anchorage
device used to anchor a non-structural element to the structural framing

3.5
bar diameter, nominal
approximate diameter of a steel reinforcing bar, often used as a class designation

NOTE The nominal diameter for deformed bars is usually taken as the diameter of a plain bar having the same area.

3.6
base of structure
level at which earthquake motions are assumed to be imparted to a building

NOTE This level does not necessarily coincide with the ground level.

3.7
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

3.8
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 The value of the bearing capacity of the soil is defined at the working stress level.

3.9
bending moment
product of a force and the distance to a particular axis, producing bending effects in a structural element

3.10
boundary element
portion along a wall edge strengthened by longitudinal and transverse reinforcement

NOTE A boundary element does not necessarily require an increase in thickness of the wall.

3.11
building
structure, usually enclosed by walls and a roof, constructed to provide support or shelter intended for occupancy

3.12
caisson
foundation pile of large diameter, built partly or totally above ground and sunk below ground usually by digging out the soil inside

3.13
cement
material as specified in the corresponding referenced International Standards, which, when mixed with water, has hardening properties, used either in concrete or by itself

3.14
column
vertical member used primarily to support axial compressive loads

3.15
collector element
element that serves to transmit the inertia forces within the diaphragm to members of the lateral-force resisting system

3.16
combined footing
footing that transmits to the supporting soil the load carried by several columns or structural concrete walls

3.17
compression reinforcement
reinforcement provided to resist compression stresses induced by flexural moments acting on the member section

3.18
concrete
mixture of Portland cement and any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixtures

3.19
concrete mix design
choice and proportioning of the ingredients of concrete

3.20
confinement hook
hook on a stirrup, hoop, or crosstie having a bend of 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

3.21
confinement stirrup
tie
closed stirrup, tie or continuously wound spiral

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

3.22
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

3.23
cover
⟨concrete⟩ thickness of concrete between the surface of any reinforcing bar and the nearest face of the concrete member

3.24
crosstie
continuous reinforcing bar having a 135° hook at one end and a hook of not less than 90° at least a six-diameter extension at the other end

NOTE The hooks normally engage peripheral longitudinal bars. The 90° hooks of two successive crossties engaging the same longitudinal bars are normally alternated end for end.

3.25
curing
keeping the concrete damp for a period of time, usually several days, starting from the moment it is cast, in order to provide the cement with enough water to harden and attain the intended strength

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

3.26
curtain wall
wall that is part of the façade or enclosure of the building

3.27
deformed reinforcement
steel reinforcement that has deformations in its surface to increase its bond to the concrete

NOTE The following steel reinforcement are normally considered deformed reinforcement under this International Standard: deformed reinforcing bars, deformed wire, welded plain wire fabric, and welded deformed wire fabric conforming to the appropriate International Standards.

3.28
depth of member
h
vertical size of a cross-section of a horizontal structural element

3.29
design load combination
combination of factored loads and forces as specified in this International Standard

3.30
design strength
product of the nominal strength multiplied by a strength reduction factor, ϕ

3.31
development length
length of embedded reinforcement required to develop the design strength of reinforcement at a critical section

3.32
development length
⟨bar with a standard hook⟩ the 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

3.33
differential settlement
movement of the foundation of different parts of a structure by different amounts

3.34
effective depth of section
d
distance measured from the extreme compression fibre to the centroid of tension reinforcement

3.35
embedment length
length of embedded reinforcement provided beyond a critical section

3.36
essential facility
building or other structure that is intended to remain operational in the event of extreme environmental loading from wind, snow, or earthquakes

3.37
factored load
factored force
specified nominal load or force multiplied by the load factors specified in this International Standard

3.38
fire protection of reinforcement
amount of concrete cover necessary to insulate 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.

3.39
flange
top or bottom part of an I-shaped section separated by the web

3.40
flexural
pertaining to the flexure bending moment

3.41
flexural reinforcement
reinforcement provided to resist the tensile stresses induced by flexural moments acting on the member section

3.42
floor system
structural elements that comprise the floor of a story in a building

NOTE The floor system includes the beams and girders, the joists (if employed), and the slab that spans between them.

3.43
footing
portion of the foundation that transmits loads directly to the soil

NOTE The footing is often the widened part of a column, a structural concrete wall or several columns, in a combined footing.

3.44
formwork
temporary construction to contain concrete in a plastic state while it is cast and setting and which forms the final shape of the element as the concrete hardens

3.45
foundation
any part of the structure that serves to transmit loads to the underlying soil, or to contain it

3.46
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

3.47
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

3.48
girder
main horizontal support beam, usually supporting other beams

3.49
gravity load
load that acts downward and is caused by the acceleration of gravity, g, acting on the mass of the elements that causes the dead and live loads

3.50
hook
bend at the end of a reinforcing bar

NOTE Hooks are classified by the angle that the bend forms with the bar as 90°, 135° or 180° hooks.

3.51
joist
T-shaped beam used in parallel series to directly support floor and ceiling loads, and are supported in turn by larger girders, beams, or bearing structural concrete walls

3.52
lap splice
splice between two reinforcing bars obtained by overlapping them for a specified length

3.53
lateral-force resisting system
that portion of the structure composed of members proportioned to resist loads related to earthquake effects

3.54
lightweight aggregate concrete
concrete made with coarse granular material that weighs less than the granular material used in normal-weight aggregates

NOTE This type of concrete is not covered in this International Standard.

3.55
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)

3.56
live load
load produced by environmental factors or the use and occupancy of the building and do not include construction or environmental loads

EXAMPLE Wind load, snow load, rain load, earthquake load, flood load, or dead load (without load factors).

3.57
load effect
force and deformation produced in structural members by the applied loads

3.58
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

3.59
load
force or other action that results from the weight of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes

3.60
longitudinal reinforcement
reinforcement that is laid parallel to the longitudinal axis of the element, generally to account for flexural effects

3.61
mass
quantity of matter in a body

3.62
mesh wire
welded-wire fabric reinforcement

3.63
modulus of elasticity
ratio of the normal stress to the corresponding strain for tensile or compressive stresses below the proportional limit of the material

3.64
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

3.65
negative reinforcement
flexural reinforcement in horizontal or nearly horizontal elements, required for negative moment and which is placed in the upper part of the section of the element

3.66
nominal load
magnitude of the load specified in this International Standard (dead, live, soil, wind, snow, rain, flood, and earthquake)

3.67
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 in this International Standard

NOTE The specified material strengths and dimensions in turn 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.

3.68
non-structural element
element corresponding to an architectural, a mechanical or an electrical component or system permanently attached to the building

3.69
occupancy
purpose for which a building or other structure, or part thereof, is used or intended to be used

3.70
partition
non-structural wall that is employed to divide spaces

NOTE A non-structural wall does not support parts of the building other than itself. When it is on the exterior, it is sometimes referred to as a curtain wall.

3.71
pedestal
upright compression member with a ratio of unsupported height to average least lateral dimension of less than 3

3.72
permanent load
load for which the variations over time are rare or of small magnitude

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

3.73
pile
slender timber, concrete or structural steel element embedded in the ground to support loads

3.74
plain reinforcement
smooth-surfaced steel reinforcement or reinforcement that does not conform to the definition of deformed reinforcement

3.75
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

3.76
positive reinforcement
flexural reinforcement in horizontal or nearly horizontal elements required for positive moment and that is placed in the lower part of the section of the element

3.77
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

3.78
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

3.79
required factored strength
strength of a member or cross-section required to resist factored loads or related internal moments and forces in such combinations as are stipulated by this International Standard

3.80
retaining wall
wall built to hold back earth

3.81
selfweight
weight of the structural element, due to the material that composes the element

3.82
service load
load specified by this International Standard (without load factors)

3.83
settlement
downward movement of the supporting soil

3.84
shear
internal force acting tangential to the plane where it acts

NOTE Also called diagonal tension.

3.85
shear reinforcement
reinforcement designed to resist shear

3.86
shores
vertical or inclined support members designed to carry the weight of the formwork, concrete and construction loads above

3.87
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 only one direction

3.88
slab
upper flat part of a reinforced concrete floor carried by supporting joists or beams or columns

3.89
slab on grade
slab set directly on the ground that serves either as an internal traffic surface or as part of the foundation

3.90
solid slab
slab of uniform thickness that does not have voids to make it lighter

3.91
span length
horizontal distance between supports of a horizontal structural element

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

3.92
specification
written document describing in detail the scope of work, materials to be used, method of installation and quality of workmanship

3.93
specified compressive strength
f'_c
⟨concrete⟩ compressive cylinder strength of concrete used in design and evaluated in accordance with the appropriate International Standard

NOTE 1 The specified compressive strength is expressed in units of megapascals, MPa.

NOTE 2 Whenever the quantity f'_c is under a radical sign $\left(\sqrt{f'_c}\right)$, the positive square root of the numerical value only is intended, and result has units of megapascals.

3.94
specified lateral earthquake forces
lateral forces corresponding to the appropriate distribution of the design base shear force prescribed by this International Standard for an earthquake-resistant design

3.95
specified wind forces
nominal pressure of the wind to be used in design in accordance with this International Standard

3.96
spiral reinforcement
continuously wound reinforcement in the form of a cylindrical helix

3.97
spread footing
isolated footing that transmits to the supporting soil the load carried by a single column

3.98
stairway
flight of steps leading from one level to another

3.99
stirrup
reinforcement used to resist shear and torsion stresses in a structural member

NOTE Typically, bars, wires, or welded wire fabric, either plain or deformed, either single leg or bent into an “L”, a “U”, or rectangular shapes, and located perpendicular to or at an angle to the longitudinal reinforcement. (The term “stirrups” is usually applied to lateral reinforcement in girders, beams, and joists and the term “ties” to those in columns and walls.) See also **tie** (3.109).

3.100
story height
vertical distance between the upper part of the slab of a story and the upper part of the slab of the floor below

3.101
strength reduction factor
ϕ
coefficient that accounts for deviations of the actual strength from the nominal strength, according to the manner and consequences of failure

NOTE A strength reduction factor includes the probability of understrength members due to variations in material strengths and dimensions and approximations in the design equations, to reflect the degree of ductility and required reliability on the member under the load effects being considered and the importance of the element in the structure.

3.102
stress
intensity of force per unit area

3.103
structural concrete
all concrete used for structural purposes including plain and reinforced concrete

3.104
structural concrete walls
walls proportioned to resist combinations of shear, moments and axial forces

NOTE A shearwall is a structural wall.

3.105
structural diaphragm
structural member, such as floor and roof slabs, which transmits the effects induced by earthquakes

3.106
support
structural element that provides support to an other structural element

3.107
tank
container for the storage of water or other fluids

3.108
temporary facility
building or other structure that is to be in service for a limited time and has a limited period of exposure to environmental loadings

3.109
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.

3.110
tie element
element which serves to transmit internal forces and to prevent separation of such building components as footings and walls

3.111
transverse reinforcement
reinforcement, such as stirrups, ties, spiral reinforcement, etc., located perpendicular to the longitudinal axis of the element

3.112
wall
member, usually vertical, used to enclose or separate spaces

3.113
web
thin vertical portion with an I-shaped section that connects the flanges

3.114
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.

3.115
wire
reinforcing bar of small diameter

3.116
working stress
allowable stress to be used with unfactored loads

3.117
yield strength
f_y
specified minimum yield strength or yield point of reinforcement

NOTE 1 The yield strength is denominated 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

4.1 Symbols

a depth of equivalent uniform compressive stress block, expressed in millimetres

a_f narrowest dimension between the sides of a form

A_b area of an individual reinforcement bar or wire, expressed in square millimetres

A_c loaded area of bearing on concrete or the area of the confined column core, in a column with spiral reinforcement, measured centre to centre of the spiral, expressed in square millimetres

A_g gross area of the section of an element, expressed in square millimetres

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, expressed in square millimetres

A_s area of longitudinal tension reinforcement, expressed in square millimetres

A'_s area of longitudinal compression reinforcement, expressed in square millimetres

$A_{s,min}$ minimum area of longitudinal tension reinforcement, expressed in square millimetres

A_{se} total extreme steel area in a column or structural concrete wall for computation of the balanced moment strength, expressed in square millimetres

A_{ss} total side steel area in a column or structural concrete wall for computation of the balanced moment strength, expressed in square millimetres

A_{st} total area of longitudinal reinforcement, expressed in square millimetres

A_{su} wind exposed surface area, expressed in square metres

A_v area of shear reinforcement within a distance s, expressed in square millimetres

b width of the compression face of the member, or width of the section of the member, expressed in millimetres

b_{ave} average value of b

b_c width of the column section, or largest plan dimension of capital or drop panel, for punching shear evaluation, expressed in millimetres

b_{col} dimension of the column section in the direction perpendicular to the girder span, expressed in metres

b_f effective width of the compression flange in a T-shaped section, expressed in millimetres

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, expressed in millimetres

b_0 perimeter of the critical section for punching shear in slabs, expressed in millimetres

d effective depth, which should be taken as the distance from the extreme compression fibre to the centroid of tension reinforcement, expressed in millimetres

d' distance from the extreme compression fibre to the centroid of compression reinforcement, expressed in millimetres

d_b nominal diameter of reinforcing bar or wire, expressed in millimetres

d_c distance from the extreme tension fibre to the centroid of tension reinforcement or diameter of the confined core of a column with spiral reinforcement, expressed in millimetres

D dead loads, or related internal moments and forces

E load effects of an earthquake or related internal moments and forces

E_c modulus of elasticity of concrete, expressed in megapascals

f'_c specified compressive strength of concrete, expressed in megapascals

$\sqrt{f'_c}$ positive square root of the specified compressive strength of concrete, expressed in megapascals

f_{cd} compressive strength of concrete reduced by the material factor, expressed in megapascals

f_{cu} extreme fibre-factored compressive stress at the edges of structural walls, expressed in megapascals

f_y specified yield strength of reinforcement, expressed in megapascals

f_{yd} yield strength of reinforcement reduced by the material factor, expressed in megapascals

f_{ypr} probable specified maximum strength of reinforcement ($f_{ypr} = 1{,}25 \cdot f_y$), expressed in megapascals

f_{ys} specified yield strength of transverse or spiral reinforcement, expressed in megapascals

f_{ysd} yield strength of transverse or spiral reinforcement reduced by the material factor, expressed in megapascals

F loads due to the weight and the pressure 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, expressed in kilonewtons

F_{iu}, F_{xu} factored design lateral force applied to the wall at level i or x, respectively, expressed in newtons

h depth or thickness of a structural element, expressed in millimetres

h_b vertical distance measured from the bottom of the supporting girder to the bottom of the supported beam, expressed in millimetres

h_{cp} dimension of a column section in the direction parallel to the girder span, expressed in metres

h_c height of the column section, expressed in millimetres

h_f slab thickness, expressed in millimetres

h_i, h_x height above the base to level i or x, respectively, expressed in metres

h_n clear vertical distance between lateral supports of columns and walls, expressed in millimetres

h_{pi} story height of floor i, measured from floor finish of the story to floor finish of the story immediately below, expressed in millimetres

h_s total height of the supporting girder, in expressed in millimetres

h_w height of entire structural concrete wall from base to top, expressed in millimetres

H loads due to the weight and pressure of soil, water in soil, or other materials, or related internal moments and forces

I_c moment of inertia of the column section, expressed in metres to the fourth power

l span of a structural element or length of a span measured centre-to-centre of beams or other supports

l_a length of clear span in the direction of two-way slabs, measured face-to-face of beams or other supports, expressed in metres

l_b length of clear span in the long direction of two-way slabs, measured face-to-face of beams or other supports, expressed in metres

l_d development length for reinforcing bar, expressed in millimetres

l_j clear spacing between joists, expressed in metres

l_m length of clear span in the direction that moments are being determined, measured face-to-face of supports, expressed in metres

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, expressed in millimetres

l_w horizontal length of structural concrete wall, expressed in millimetres

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

M_{bn} nominal flexural moment strength at section at balanced conditions, expressed in neweton·millimetres

M_{br} flexural moment strength at section at balanced conditions, expressed in newton·millimetres

M_{iu}, M_{xu} factored story moment caused by lateral loads at story i or x, respectively, expressed in newtons

M_n nominal flexural moment strength at section, expressed in newton·millimetres

M_r flexural moment strength at section, expressed in newton·millimetres

M_{pr} probable flexural moment strength of the element at the joint face, computed using f_{ypr} and $\phi = 1$, expressed in newton·metres

M_u factored flexural moment at section, expressed in newton·metres

M_u^- factored negative flexural moment at section, expressed in newton·metres

M_u^+ factored positive flexural moment at section, expressed in newton·metres

ΣM_c sum of the lowest flexural strengths ($\phi \cdot M_n$) of columns framing into a joint, expressed in newton·metres

ΣM_g sum of flexural strengths ($\phi \cdot M_n$) of girders framing into a joint, expressed in newton·metres

ΔM_u factored unbalanced moment at a column-girder joint or factored unbalanced moment at a wall-girder joint, expressed in newton·metres

N_{st} nominal strength, P_d, the non-factored dead-load axial force at section or non-factored concentrated dead load applied directly to the element, expressed in newtons

N_b is the number of bars in a layer

P_{bn} nominal compression axial load strength at section at balanced conditions, expressed in newtons

P_{br} nominal compression axial load strength at section at balanced conditions, expressed in newtons

P_{cu}	factored compression load on a 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, expressed in newtons
P_{n}	nominal axial load strength at section, expressed in newtons
$P_{\text{n(max)}}$	maximum compression nominal axial load strength at section, expressed in newtons
P_{tn}	axial tension strength at section, expressed in newtons
P_{tu}	factored tension force on a 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, expressed in newtons
P_{0n}	axial compressive strength at section, expressed in newtons
ΣP_{u}	sum of all factored concentrated design loads within the span, expressed in newtons
q_{d}	non-factored dead load per unit area, expressed in newtons·per square metre
q_{l}	non-factored live load per unit area, expressed in newtons·per square·metre
q_{u}	factored load per unit area, expressed in newtons·per square metre
r_{u}	factored uniformly distributed reaction from the slab on the supporting girder, beam or structural concrete wall, expressed in newtons·per metre
R_{a}	rain load or related internal moments and forces
R_{u}	total factored concentrated reaction from a supported structural element, expressed in newtons
ΣR_{u}	sum of all factored reactions from supported structural elements at the same story, expressed in newtons
s	centre-to-centre 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, expressed in millimetres
S	snow load or related internal moments and forces
T	cumulative effect of temperature, creep, shrinkage or differential settlement, or related internal moments and forces
T_{u}	factored torsional moment at section, expressed in newton·millimetres
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, expressed in newtons
ΔV_{e}	factored design shear force from the development of the probable flexural capacity of the element at the faces of the joints, expressed in newtons
$V_{i\text{u}}$, $V_{x\text{u}}$	factored story shear caused by lateral loads at story i or x, respectively, expressed in newtons
V_{n}	nominal shear strength at section, expressed in newtons

V_s contribution of the horizontal reinforcement to the nominal shear strength at section, expressed in newtons

V_u factored shear force at section, expressed in newtons

w_d non-factored uniformly distributed dead load per unit element length applied directly to the element, expressed in newtons per metres

w_l non-factored uniformly distributed live load per unit element length applied directly to the element, expressed in newtons per metres

w_u factored uniformly distributed design load per unit element length applied directly to the element, expressed in newtons per metres

W wind loads or related internal moments and forces

W_u total factored uniformly distributed design load per unit element length, expressed in kilonewtons per metres

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, equal to $\frac{A_s}{b \cdot d}$

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 equal to $\frac{A_{st}}{b \cdot d}$

ρ_v ratio of vertical reinforcement in structural concrete walls

4.2 Abbreviated terms

max. maximum

min. minimum

5 Design and construction procedure

5.1 Procedure

The design procedure consists of steps A through K; see also Figure 1.

5.1.1 Step A

Step A includes definition of the layout in plan and height of the structure, following the provisions of 7.1, and verification that the limitations specified in 6.1 are met.

5.1.2 Step B

Step B includes calculation of all gravity loads that act on the structure using the provisions of 7.2, excluding the selfweight of the structural elements.

5.1.3 Step C

Step C includes definition of an appropriate floor system, depending on the span lengths and the magnitude of the gravity loads, in accordance with the provisions of 7.4.

5.1.4 Step D

Step D includes trial dimensions for the slab of the floor system, calculation of the selfweight of the system, and design of the elements of which it is composed, correcting the dimensions as required by the strength and serviceability limit states, and in accordance with the provisions of 7.5 for slab systems with beams.

5.1.5 Step E

Step E includes trial dimensions for the beams and girders, calculation of their selfweight, flexural and shear design of the beams and girders, correcting the dimensions as required by the strength and serviceability limit states and in accordance with the provisions include in 7.6.

5.1.6 Step F

Step F includes trial dimensions for the columns, calculation of their selfweight, column slenderness verification and design for the combination of axial load and moment, and shear; correcting the dimensions as required by the strength and serviceability limit states, in accordance with the provisions of 7.7.

5.1.7 Step G

If lateral loads such as earthquake, wind, or lateral earth pressure exist, their magnitude is established using the provisions of 7.2; otherwise the designer should proceed to Step I.

5.1.8 Step H

Preliminary location and trial dimensions for structural concrete walls capable of resisting the lateral loads are established in accordance with the provisions of 7.8 for earthquake forces. The influence of their selfweight is evaluated and flexure and shear design of the structural concrete walls is performed in accordance with the provisions of 7.9.

5.1.9 Step I

The loads at the foundation level are determined, and a definition of the foundation system is formulated in accordance with the provisions of 7.10. The structural elements of the foundation are designed.

5.1.10 Step J

Step J includes the production of the structural drawings.

5.1.11 Step K

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

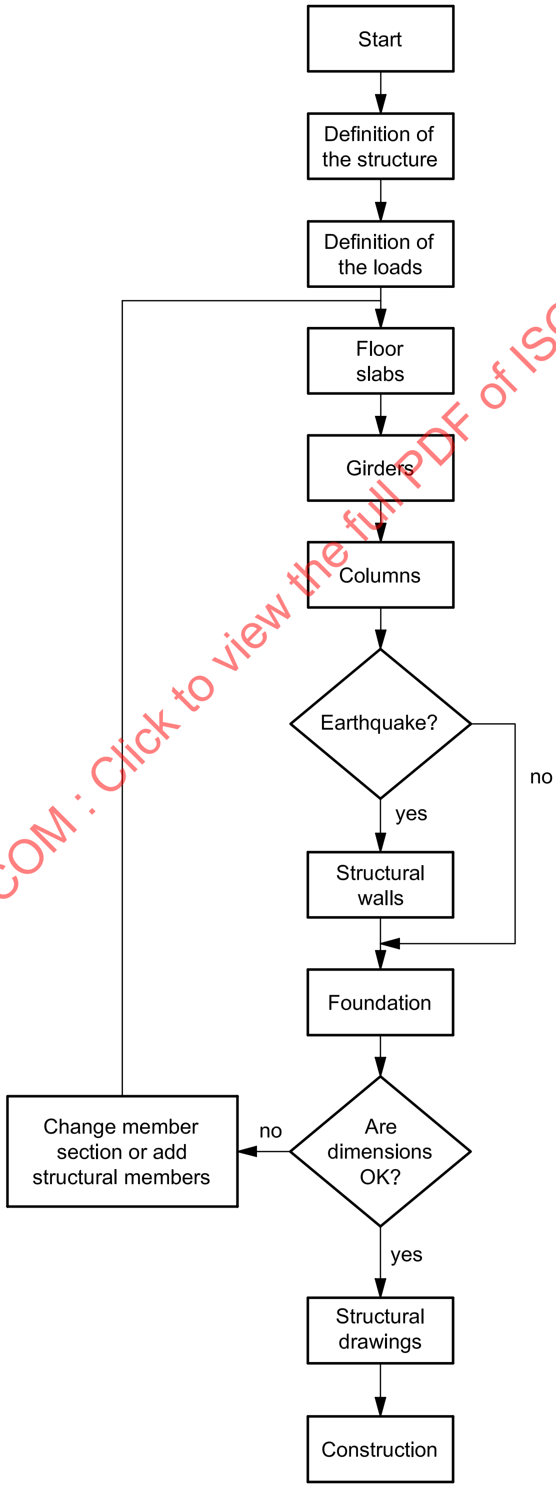


Figure 1 — Design procedure

5.2 Design documentation

The design steps should be fully recorded in the documents described in 5.2.1 to 5.2.4.

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) general structural requirements of the project, in accordance with 7.1;

b) 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 the design of any soil retaining structure and all other information required in 7.10.

5.2.3 Structural drawings

This includes all the drawings required for construction of the structure of the building.

5.2.4 Specifications

The construction specifications are required.

6 General provisions

6.1 Limitations

These provisions should be employed only when the building being designed complies with all the limitations set forth in 6.1.1 to 6.1.10.

6.1.1 Occupancy

6.1.1.1 Permitted occupancies

The intended use of the building being designed should be permitted for the occupancy subgroup as specified in Table 1.

Table 1 — Permitted occupancies

Occupancy group		Occupancy subgroup	Permitted
Group A – Assembly	A-1	churches, theatres, stadiums, coliseums, gymnasiums	NO
	A-2	building having an assembly room with a capacity of fewer than 100 persons and not having a stage	[YES]
Group B – Business	B	building for use as offices, and for professional services, containing eating and drinking establishments with fewer than 50 occupants	[YES]
Group E – Educational	E-1	classrooms for schools up to high-school	[YES]
	E-2	classrooms for universities	[YES]
Group F – Industrial	F-1	light industries not employing heavy machinery	[YES]
	F-2	heavy industries employing heavy machinery	NO
Group G – Garages	G-1	garages for vehicles with a carrying capacity up to 2 000 kg	YES
	G-2	garages for trucks of more than 2 000 kg carrying capacity	NO
Group H – Health	H-1	nurseries for day-care of infants	[YES]
	H-2	health care centres for ambulatory patients	[YES]
	H-3	hospitals	NO
Group M – Mercantile	M	display and sale of merchandise	YES
Group R – Residential	R-1	hotels	[NO]
	R-2	houses and apartment buildings	YES
Group S – Storage	S-1	storage of light materials	YES
	S-2	storage of heavy or hazardous materials	NO
Group U – Utility	U	utilities, water supply systems, power generating plants	NO

6.1.1.2 Mixed occupancy

Buildings of mixed occupancy should be permitted to be designed using these provisions when all the types of occupancy in the building are permitted by Table 1.

6.1.2 Maximum number of stories

The maximum number of stories for a building designed in accordance with this International Standard should be [*five*]. This number of stories should include the floor at the level of the ground and any basement, and should not include the roof. The number of basements should not exceed one.

6.1.3 Maximum area per floor

The maximum area per floor should not exceed [500] m^2.

6.1.4 Maximum story height

The maximum story height, measured from the floor finish to the floor finish of the story immediately below, should not exceed [*4*] m.

6.1.5 Maximum span length

The maximum span length for girders and beams, measured centre-to-centre of the supports, should not exceed [10] m.

6.1.6 Maximum difference in span length

Spans shall be approximately equal, and the larger of two adjacent spans should not be greater than the shorter by more than 20 % of the larger span.

6.1.7 Minimum number of spans

The minimum number of spans in each of the two principal directions in the plan of the building should not be less than two. It should be permitted to use one span in buildings of one or two stories, but the span length should not exceed [*5*] m.

6.1.8 Maximum cantilever span

The maximum clear span length for girders, beams and slabs in cantilever should not exceed 1/3 of the span length of the first interior span of the element, in order to avoid cantilevers too long for the purposes of this International Standard.

6.1.9 Maximum slope for slabs, girders, beams and joists

It should be permitted to use sloping slabs, girders, beams and joists, but the slope of the structural element should not exceed 15°, except in members that are part of stairways.

6.1.10 Maximum slope of the terrain

The slope of the terrain where the building is located should not exceed, in any direction, a value that will produce a rise of the terrain, in the length of the building in that direction, of more than the story height of the first floor of the building, without exceeding a slope of [30]°.

6.1.11 Distance between centre of mass and centre of rigidity

The distance between the centre of mass and the centre of rigidity shall be kept small to reduce the risk of global torsion of the structure.

6.2 Limit states

The design approach for the purposes of this International Standard 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:

a) structural integrity limit state;

b) lateral load story drift limit state;

c) durability limit state;

d) fire limit state;

e) ultimate and serviceability limit states.

6.3 Ultimate limit state design format

6.3.1 General

For the ultimate limit state design, the structure and the structural members should be designed to have design strength at all sections at least equal to the required strengths calculated for the factored loads and forces in such combinations as are stipulated in this International Standard.

The basic requirement for ultimate limit state should be that the resistances are equal to or greater than the load effects.

To allow for the possibility that the resistances can be less than computed and the load effects can be larger than computed, strength reduction factors, ϕ, less than one, and load factors, γ, generally greater than one, should be employed as shown in Equation (1):

$$\phi \cdot R_n \geqslant \gamma_1 \cdot S_1 + \gamma_2 \cdot S_2 + \cdots \qquad (1)$$

where

R_n is the nominal strength;

S_1, S_2, are load effects based on the nominal loads specified by this International Standard.

Therefore, the ultimate limit state design format requires that the design strength is equal to or greater than the required factored strength, as specified in Equation (2):

$$\phi \cdot N_{st} \geqslant U \qquad (2)$$

where

U is the required factored strength, equal to $\gamma_1 \times S_1 + \gamma_2 \times S_2 + \ldots$;

N_{st} is the nominal strength.

6.3.2 Required factored strength

The required factored strength, U, should be computed by multiplying service loads or forces by load factors using the load factors and combinations specified in 7.2.2.

6.3.3 Design strength

The design strength provided by a member, its connections to other members and its cross-sections, in terms of flexure, axial load and shear, should be taken as the nominal strength calculated in accordance with the requirements and assumptions of this International Standard for each particular force effect in each of the element types at the critical sections defined by this International Standard, multiplied by the following strength reduction factors, ϕ:

a) flexure, without axial load: ϕ = [0,90];

b) axial tension, and axial tension with flexure: ϕ = [0,90];

c) axial compression and axial compression with flexure:

— columns with ties, and structural concrete walls: ϕ = [0,70],

— columns with spiral reinforcement ϕ = [0,75];

d) shear and torsion ϕ = [0,85];

e) bearing of concrete ϕ = [0,70].

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 accordance with the serviceability limit state for the purposes of this International Standard, should be obtained indirectly thorough the observance of the limiting dimensions, cover, detailing and construction requirements. These serviceability conditions include effects such as the following:

a) lack of durability due to long-term environmental effects, including exposure to an aggressive environment or corrosion of the reinforcement;

b) dimensional changes due to variations in temperature, relative humidity and other effects;

c) excessive cracking of the concrete;

d) excessive horizontal deflections;

e) excessive vertical deflections;

f) excessive vibration.

7 Specific provisions

7.1 Structural systems and layout

7.1.1 Description of the components of the structure

For the purposes of this International Standard, the building structure should be divided into components as specified in 7.1.1.1 to 7.1.1.5.

7.1.1.1 Floor system

The floor system consists of the structural elements that comprise the floor of a story in a building. The different types of floor systems covered by this International Standard are described in 7.4. The floor 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 columns, as in slab-column systems.

7.1.1.2 Vertical supporting elements

The vertical supporting elements hold up the floor system at each story and transmit the accumulated gravity loads all the way down to the foundation of the structure. For the purposes of this International Standard, they can be either columns or structural concrete walls.

7.1.1.3 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. Included are elements such as spread footings, combined footings, foundation mats, basement and retaining walls, grade beams and slabs on grade, among others. Deep foundations, such as piles and caissons, and their pile footings and caps, are beyond the scope of this International Standard and are not covered.

7.1.1.4 Lateral-load resisting system

The lateral-load-resisting system is composed of the structural elements that acting jointly support and transmit to the ground the lateral loads arising from earthquake motions, wind and lateral earth pressure. The floor system should act as a diaphragm that carries in its plane the lateral load from the point of application to the vertical elements of the lateral-load resisting system. The vertical elements of the lateral-load resisting system, in turn, collect the forces arising from all floors and transmit them down to the foundation and through the foundation to the underlying soil. Under this International Standard, the main vertical elements of the lateral-load-resisting system should be structural concrete walls.

7.1.1.5 Other structural elements

Other structural elements that are part of the structure of the building are stairways, ramps, water tanks and slabs on grade.

7.1.2 General program

7.1.2.1 Architectural program

A general architectural program of the building should be coordinated with the structural designer before actual structural design begins. The general architectural program should include, at least, the following items:

a) plan shape and dimensions of all the floors of the building;

b) elevation of the building and its relationship with the terrain, including the basement, if any;

c) type of roof, its shape and slopes, the type of water-proofing, the means to facilitate the runoff of water from rain and melting snow or hail and the location of drainage gutters;

d) use of the internal space of the building, its subdivision, and means of separation, in all stories;

e) minimum architectural clear height in all floors;

f) location of stairways, ramps, and elevators;

g) type of building enclosure, internal partitions, architectural, and non-structural elements;

h) location of ducts and shafts for utilities, such as power supply, lighting, thermal control, ventilation, water supply and waste water, including enough information to detect interference with the structural elements.

7.1.2.2 General structural provisions

Based on the general architectural program information, the structural designer should define the general structural requirements for the structure being designed under this International Standard. These general structural requirements should include, at least, the following items:

a) intended use of the building;

b) nominal loads related to the use of the building;

c) special loads required by the owner;

d) [design earthquake motions, if the building is located in a seismic zone];

e) wind requirements for the site;

f) requirements for [snow], [hail] or rain;

g) fire requirements;

h) type of roof and appropriate loads when not built from reinforced concrete;

i) site information related to slopes and site drainage;

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

k) environmental requirements derived from local seasonal and daily temperature variations, humidity, presence of deleterious chemicals and salt;

l) availability, type and quality of materials such as reinforcing bars, cement, aggregates;

m) availability of materials for formwork;

n) availability of a testing lab for concrete mix design and quality control during construction;

o) availability of qualified workmanship.

7.1.3 Structural layout

7.1.3.1 General structural layout

The structural designer should define a general structural layout in plan. This general layout should include all information, in plan, that is common to all levels of the structure; see Figure 2. The general structural layout in plan should include the following:

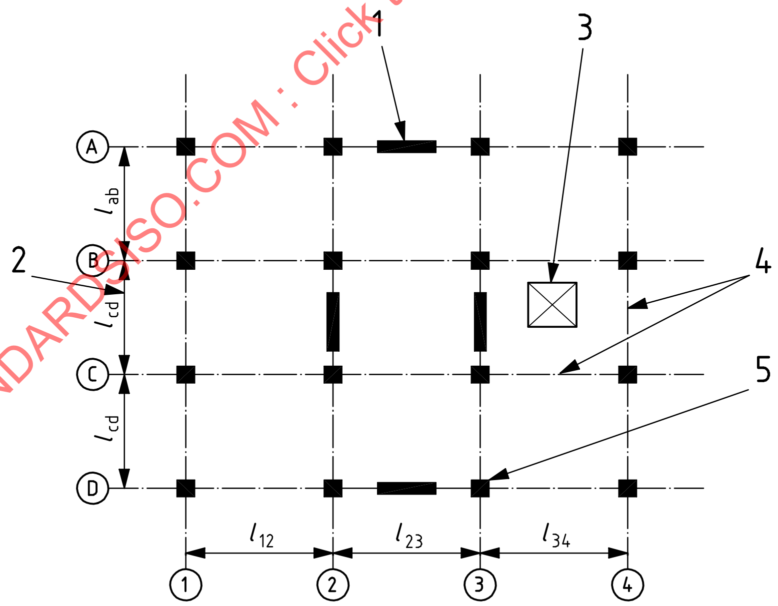
a) Dimensioned grid of axes, or centrelines, in both principal directions in plan. These axes should intersect at the location of the vertical supporting elements (columns and structural concrete walls).

b) Location in plan of all vertical supporting elements, columns and structural concrete walls. These vertical supporting elements should be aligned vertically and should be continuous all the way down to the foundation. Walls that separate spaces, built of reinforced concrete, can be made into structural concrete walls if they are continuous all the way down to the foundation and have no openings for windows or doors.

c) Location of all ducts, shafts, elevators and stairways that are continuous from floor to floor;

d) Horizontal distances, l, between centrelines, which correspond to the centre-to-centre span lengths of the floor system;

e) In seismic zones, the location and distribution of all structural concrete walls.



Key

1 wall
2 span length
3 shaft
4 centreline grid
5 column

Figure 2 — General structural layout in plan

7.1.3.2 Floor layout

For each typical floor, the structural designer should develop a structural floor layout (see Figure 3) that should contain the following:

a) superposition of the floor perimeter on the general grid of axis;

b) girder and beam location, or column and middle strips for slab-column systems;

c) additional information, including all substantial architectural openings in the floor;

d) approximate load path from all floor areas to the supporting beams and girders.

7.1.3.3 Vertical layout

The structural designer should define a general structural vertical layout (see Figure 4) that should include all relevant information relating to the height of the structure, including the following:

a) number of stories;

b) for all floors, the story height, defined as the vertical distance from the floor finish to the floor finish of the floor immediately below;

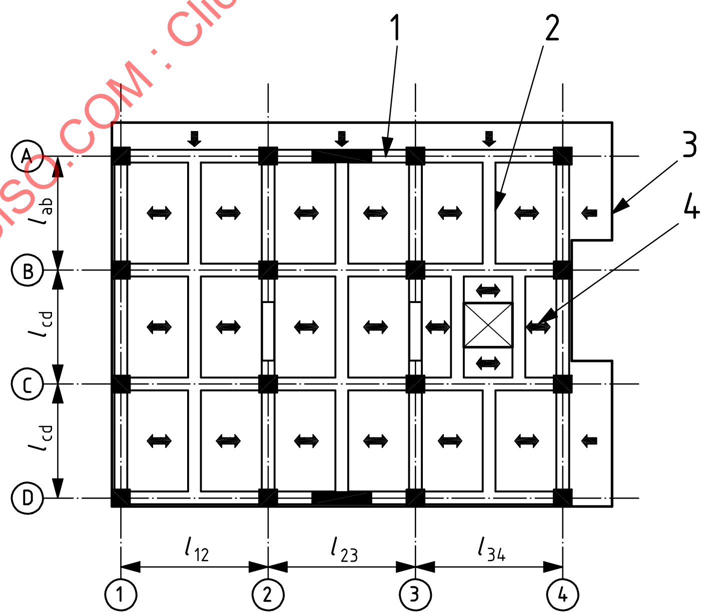
c) slope and shape of the roof;

d) architectural vertical clearance from floor finish to ceiling, as required by the use of the building;

e) vertical space necessary to accommodate horizontal and vertical service elements for power, water supply and drainage, heating, ventilation and air conditioning;

f) slope of the terrain and its relationship to the ground floor or basement, if any;

g) supporting soil-stratum depth and water-table depth.



Key

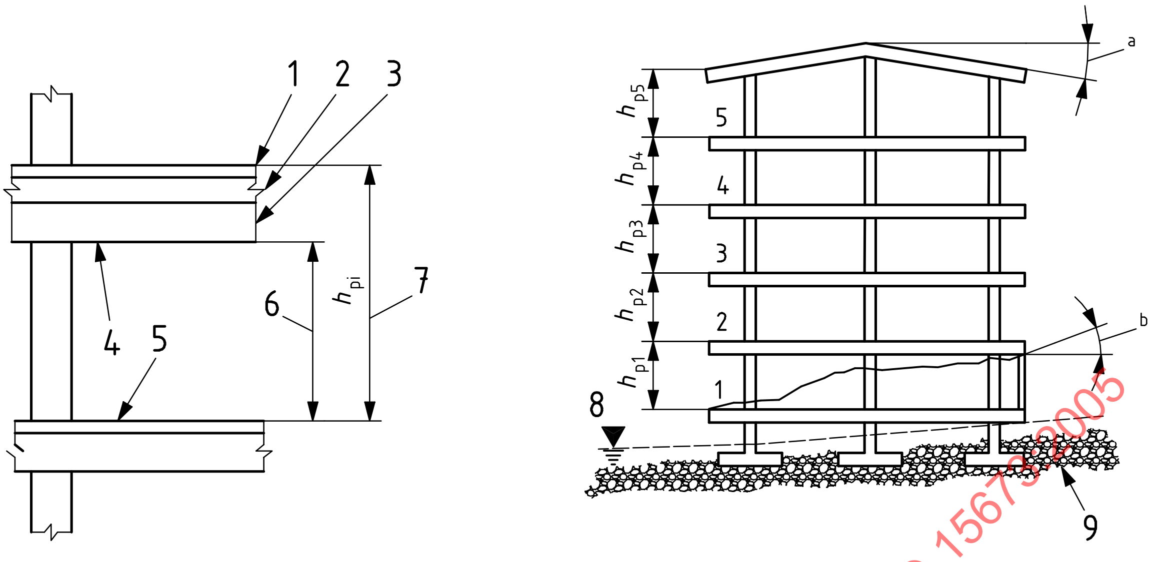
1 girder

2 beam

3 structural perimeter

4 load path

Figure 3 — Typical floor structural layout



a) floor section details

b) building elevation

Key

1 floor finish
2 slab
3 service elements
4 ceiling
5 floor finish
6 architectural vertical clearance
7 story height, h_{pi}
8 water-table
9 bearing soil stratum

a Maximum slope, 15°.
b Maximum slope, 30°.

Figure 4 — Vertical layout of the building

7.1.4 Feasibility

Based on the layout information, the structural designer should verify the feasibility of performing the structural design in accordance with this International Standard. The compliance with the following limitations should be verified.

a) The use of the building should be within the accepted occupancies as specified in 6.1.1 and, if of mixed use, all types of intended occupancies should be within those permitted.

b) The number of stories should not exceed the maximum permissible, as specified in 6.1.2.

c) The area of the largest floor should not exceed the maximum permissible area as specified in 6.1.3.

d) The story height of the tallest story, measured from finish-to-finish, should not exceed the maximum permissible story height as specified in 6.1.4.

e) The span lengths should be within the maximum span length as specified in 6.1.5.

f) The difference between adjacent spans should not exceed the limits as specified in 6.1.6.

g) The number of spans in both directions and in all floors should not be fewer than two, as specified in 6.1.7, or within the exceptions stated there.

h) No girder, beam or slab cantilever should exceed the limits as specified in 6.1.8.

i) No sloping girder, beam, joist or slab should have a slope greater than the maximum permissible value as specified in 6.1.9.

j) The slope of the terrain at the building site should not exceed the maximum as specified in 6.1.10.

7.2 Actions (loads)

7.2.1 General

Subclause 7.2 provides the minimum load provisions for the design of buildings under this International Standard. Loads and the appropriate load combinations should be used together.

7.2.2 Load factors and load combinations

The load factors and combinations specified in 7.2.2.1 to 7.2.2.7 should be employed to obtain the required factored strength, U, of the structural member or element as specified in 6.3.1. In the following load combinations set forth to obtain the required factored strength U, the symbol ± for alternating forces that can act in one direction or the opposite should be interpreted as the force with the sign that leads to the maximum (positive) or minimum (negative) value of U.

7.2.2.1 Dead and live load

The required factored strength, U, to resist dead load, D, and live load, L, should be equal to at least the greater of the results of Equations (3) and (4):

$$U = [1{,}6] \cdot D \tag{3}$$

and

$$U = [1{,}4] \cdot D + [1{,}7] \cdot L \tag{4}$$

7.2.2.2 Rain load, snow load and sloping roof live load

If resistance to structural effects of a specified rain load, R_a, snow load, S, or sloping roof live load, L_r, is required to be included by the specifications of this International Standard, the combinations of D, L and (R_a, S or L_r) as specified in Equations (5) and (6) should be investigated to determine the greatest required factored strength, U:

$$U = [1{,}4] \cdot D + [1{,}7] \cdot L + [0{,}6] \cdot (R_a \text{ or } S \text{ or } L_r) \tag{5}$$

$$U = [1{,}4] \cdot D + [0{,}6] \cdot L + [1{,}7] \cdot (R_a \text{ or } S \text{ or } L_r) \tag{6}$$

However, for any combination of D, L and (R_a, S or L_r), the required factored strength, U, should not be less than the values derived from Equations (3) and (4).

7.2.2.3 Wind

If resistance to structural effects of a specified wind load, W, is required to be included by the specifications of this International Standard, the combinations of D, L and W as specified in Equations (7) and (8) should be investigated to determine the greatest required factored strength, U:

$$U = [0{,}75] \cdot ([1{,}4] \cdot D + [1{,}7] \cdot L) + [1{,}3] \cdot W = [1{,}1] \cdot D + [1{,}3] \cdot L + [1{,}3] \cdot W \tag{7}$$

where the load combinations should include both the full value and a 0 value of L to determine the more severe condition, and

$$U = [0{,}9] \cdot D + [1{,}3] \cdot W \tag{8}$$

However, for any combination of D, L and W, the required factored strength, U, should not be less than the values derived from Equations (3) and (4).

7.2.2.4 Earthquake forces

If resistance to specified earthquake forces, E, is required to be included by the specifications of this International Standard, the combinations of D, L and E as specified in Equations (9) and (10) should be investigated to determine the greatest required factored strength U:

$$U = [0{,}75] \cdot ([1{,}4] \cdot D + [1{,}7] \cdot L) \pm [1{,}0] \cdot E = [1{,}1] \cdot D + [1{,}3] \cdot L \pm [1{,}0] \cdot E \tag{9}$$

where the load combinations should include both the full value and a 0 value of L to determine the more severe condition, and

$$U = [0{,}9] \cdot D \pm [1{,}0] \cdot E \tag{10}$$

However, for any combination of D, L and E, the required factored strength, U, should not be less than the value derived from Equations (3) and (4).

7.2.2.5 Earth pressure

If resistance to earth pressure, H, is required by the design procedure of this International Standard, the required factored strength, U, should be at least equal to the value derived from Equation (11):

$$U = [1{,}4] \cdot D + [1{,}7] \cdot L + [1{,}7] \cdot H \tag{11}$$

except where D or L reduces the effect of H, in which case Equation (12) should be employed:

$$U = [0{,}9] \cdot D + [1{,}7] \cdot H \tag{12}$$

For any combination of D, L and H, the required factored strength, U, should not be less than the values derived from Equations (3) and (4). When the building structure as a whole should resist permanent uncompensated horizontal loads due to lateral soil pressure, ($[1{,}7] \cdot H$) should be added to the right side of Equations (3), (4), (8), (9), (10), (11) and (12).

7.2.2.6 Weight and pressure of fluids

If resistance to loadings due to the weight and pressure of fluids with well-defined densities and controllable maximum heights, F, is required by the design procedure of this International Standard, ($[1{,}7] \cdot F$) should be added to the right side of Equations (3), (4), (9) and (11).

7.2.2.7 Other effects

Where structural effects, T, of differential settlement, shrinkage or temperature change are significant to the design, the design should not be preformed using this International Standard, and the appropriate standard of each country should be employed.

7.2.3 Mass of materials

For defining the mass of materials, the requirements of the applicable national standard should be met. When no national standard is available, the requirements of ISO 9194 should be used.

7.2.4 Dead loads

Dead loads consist of the mass of all material of construction incorporated into the building, including, but not limited to structure, walls and partitions, floors, roofs, ceilings, stairways, ramps, finishes, cladding and other incorporated architectural and structural systems, and fixed service equipment. In determining dead loads for the purposes of design, the actual masses of materials and constructions should be used. In determining dead loads for purposes of design, the mass of fixed service equipment, such as plumbing stacks and risers, electrical feeders, and heating, ventilating and air conditioning systems, should be included.

7.2.5 Live loads

For live loads, the requirements of the applicable national standard should be met. When no national standard is available, the requirements of ISO 2103 should be used. For industrial buildings and storage facilities, the requirements of ISO 2633 should be consulted for the determination of realistic live loads.

7.2.6 Specified snow load

When snow is expected to fall due to geographical latitude, altitude, or both, the loads caused by its accumulation should be taken into account in the design of the roof. The requirements of the applicable national standard should be met, and when no national standard is available, it should be permitted to employ the requirements ISO 4355.

7.2.7 Specified wind forces

For wind loading, the requirements of the applicable national standard should be met. When no national standard is available, the requirements of ISO 4354 should be employed.

7.2.8 Specified earthquake forces

For earthquake loading, the requirements of the applicable national standard should be met. When no national standard is available, the requirements of ISO 3010 should be used.

7.3 General reinforced concrete requirements

7.3.1 General

7.3.1.1 Scope

Subclause 7.3 contains the provisions that are common to the reinforced-concrete structural elements covered by this International Standard. They include provisions 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.

7.3.1.2 Additional requirements

The designer should be in accordance with the additional requirements for each individual element type as specified in 7.4 to 7.10 of this International Standard.

7.3.2 Materials for reinforced concrete

7.3.2.1 General

All materials employed in the construction of a structure designed in accordance with this International Standard should conform to the following ISO standards:

7.3.2.2 Cement

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

— ISO 679;

— ISO 680;

— ISO 863.

7.3.2.3 Aggregates

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

- ISO 6274;
- ISO 6782;
- ISO 6783;
- ISO 7033.

7.3.2.4 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 the corresponding national mixing-water standard.

7.3.2.5 Steel reinforcement

Steel reinforcement should be deformed reinforcement, with the exceptions noted in 7.3.2.5.3 and should be in accordance with the following limitations and with the applicable ISO standards, specifically ISO 10144. Welded-wire fabric should be considered deformed reinforcement for the purposes of this International Standard.

7.3.2.5.1 Deformed reinforcement

The maximum specified yield strength for deformed reinforcement should be 400 MPa. Deformed reinforcing bars should conform to ISO 6935-2 or the corresponding national deformed reinforcement standard. ISO 6935-2 covers grades RB 300, RB 400 and RB 500 (with a characteristic upper yield stress of 300 MPa, 400 MPa and 500 MPa, respectively) and nominal diameters of 6 mm, 8 mm, 10 mm, 12 mm, 16 mm, 20 mm, 25 mm, 32 mm and 40 mm, although for the purposes of this International Standard, the nominal diameter of deformed reinforcement bars is limited to 25 mm (see 7.3.3).

7.3.2.5.2 Welded-wire fabric

The maximum specified yield strength for the wire in welded-wire fabric should be 400 MPa. Welded-wire fabric should conform to ISO 6935-3 or corresponding national standard for welded-wire fabric. For the purposes of this International Standard, the nominal diameter of wire for welded-wire fabric is limited to 10 mm (see 7.3.3).

7.3.2.5.3 Plain reinforcement

Plain reinforcement should be permitted only for stirrups, ties, spirals and components of 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 the corresponding national standard for plain reinforcement. ISO 6935-1 covers grades PB 240 and PB 300 (with a characteristic upper yield stress of 240 MPa and 300 MPa, respectively) and nominal diameters of 6 mm, 8 mm, 10 mm, 12 mm, 16 mm and 20 mm, although for the purposes of this International Standard, the nominal diameter of plain reinforcement bars is limited to 16 mm (see 7.3.3).

7.3.2.6 Admixtures

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

7.3.2.7 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.

7.3.3 Minimum and maximum reinforcement bar diameter

Reinforcement employed in structures designed in accordance with this International Standard should not have a nominal diameter, d_b, less than the minimum diameter, nor larger than the maximum diameter as specified in Table 2.

Table 2 — Minimum and maximum nominal diameters for reinforcement bar

Application	Minimum bar diameter d_b mm	Maximum bar diameter d_b mm
Deformed reinforcing bars (see 7.3.2.5.1)	6	25
Wire for welded-wire fabric (see 7.3.2.5.2)	4	10
Stirrups and ties	6	16
Plain reinforcing bars (see 7.3.2.5.3)	6	16

7.3.4 Concrete cover of reinforcement

7.3.4.1 Minimum concrete cover

The minimum concrete cover as specified in Figures 5 to 8 should be provided for reinforcement.

Dimension in millimetres

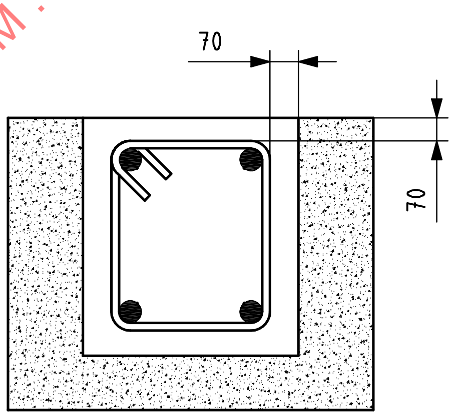


Figure 5 — All types of reinforcement of elements cast and permanently exposed to earth — Minimum concrete cover 70 mm

Dimension in millimetres

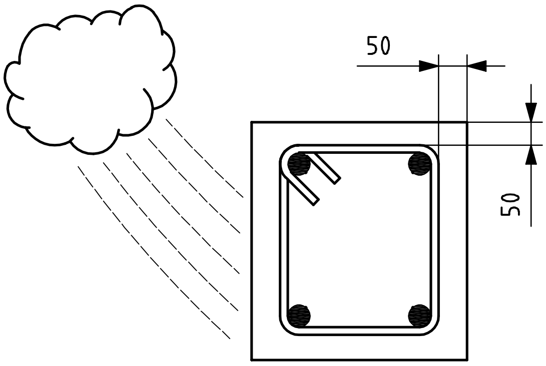


Figure 6 — All types of reinforcement of elements exposed to weather — Minimum concrete cover 50 mm

Dimension in millimetres

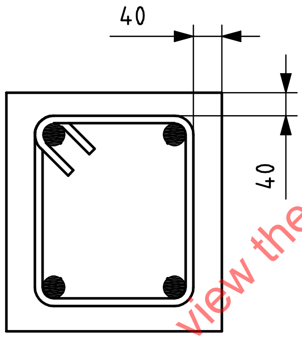


Figure 7 — All types of reinforcement of girders, beams, or columns, when not exposed to weather or in contact with ground — Minimum concrete cover 40 mm

Dimension in millimetres

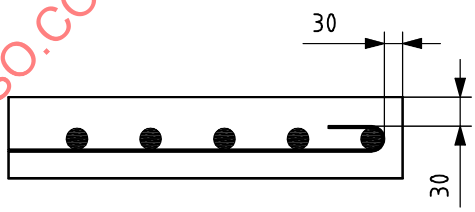


Figure 8 — All types of reinforcement of solid slabs, structural concrete walls or joists, when not exposed to weather or in contact with ground — Minimum concrete cover 30 mm

7.3.4.2 Special fire protection

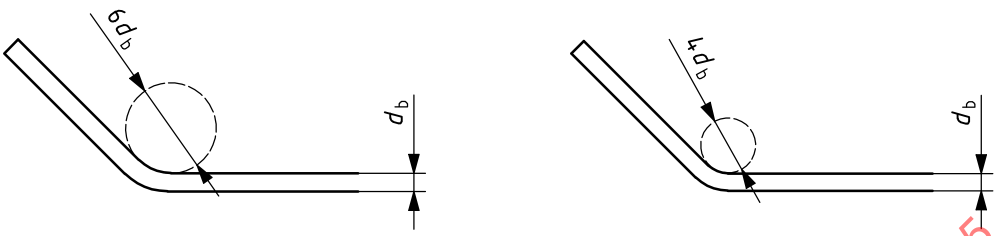
When the designated fire protection for a building is greater than 1 h, the concrete cover provisions of 7.3.4.1 should be increased by 12 mm per each additional hour of fire protection after the first hour. The structural designer should consult the requirements of ISO/TR 3956.

7.3.4.3 Special corrosion protection

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

7.3.5 Minimum reinforcement bend diameter

The diameter of bends of the reinforcement, measured on the inside of the bar, should not be less than the values specified in Figure 9.



For deformed reinforcing bars: bend diameter of $6d_b$.

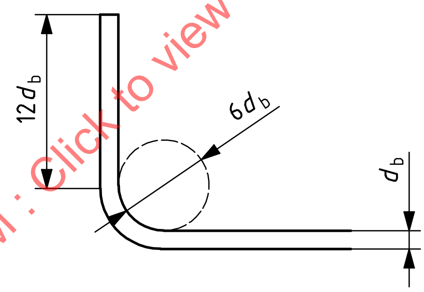
For plain reinforcing bars: bend diameter of $6d_b$.

For stirrups and ties: bend diameter of $4d_b$.

Figure 9 — Minimum reinforcement bend diameter

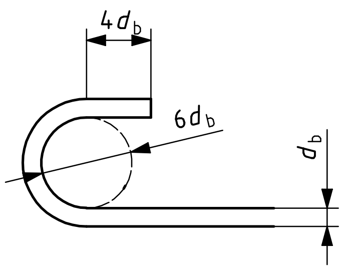
7.3.6 Standard hook dimensions

The term “standard hook” as used in this International Standard means one of the configurations shown in Figures 11 to 14.



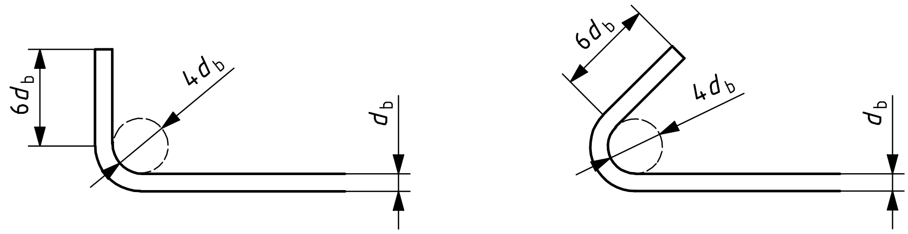
Description: a 90° bend plus a $12d_b$ extension at the free end of the bar.

Figure 10 — 90° hook



Description: a 180° bend plus a $4d_b$ extension at the free end of the bar.

Figure 11 — 180° hook



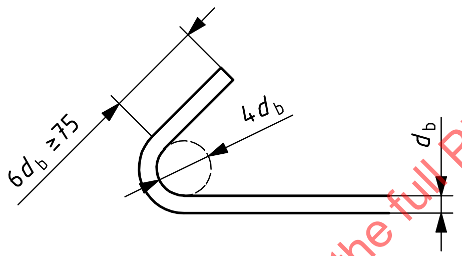
a) b)

Description a): a 90° bend plus $6d_b$ extension at free end of bar.

Description b): a 135° bend plus $6d_b$ extension at free end of bar.

Figure 12 — Stirrup and tie hooks

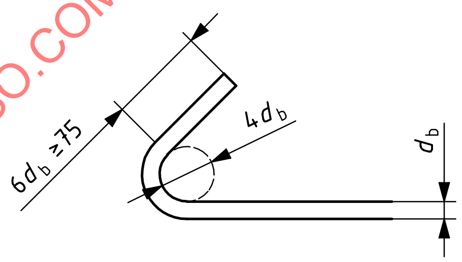
Dimension in millimetres



Description: a 135° bend plus a $6d_b$ extension at the free end of the bar, but not less than 75 mm.

Figure 13 — Confinement stirrups and ties in seismic zones

Dimension in millimetres



Description: a 135° bend plus a $6d_b$ extension at the free end of the bar, but not less than 75 mm.

Figure 14 — Crossties in seismic zones

7.3.7 Bar separation and maximum aggregate size

The clear spacing between parallel bars in a layer and the maximum coarse aggregate size should be interrelated as specified in 7.3.7.1.

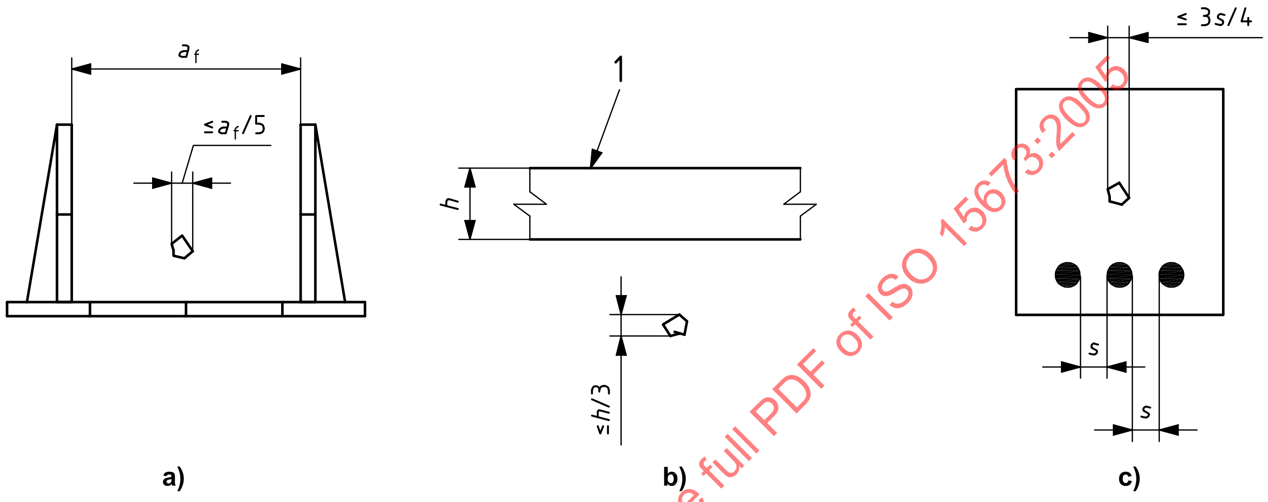
7.3.7.1 Maximum nominal coarse aggregate size

Maximum nominal coarse aggregate size should be not larger than the following; see Figure 15:

a) 1/5 of the narrowest dimension between sides of the forms;

b) 1/3 of the depth of the slab;

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



Key

1 slab

Figure 15 — Maximum nominal coarse aggregate size

7.3.7.2 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 16. This provision should apply also to the separation between parallel stirrups or ties.

7.3.7.3 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 a clear distance between layers of not less than 25 mm; see Figure 16.

Dimensions in millimetres

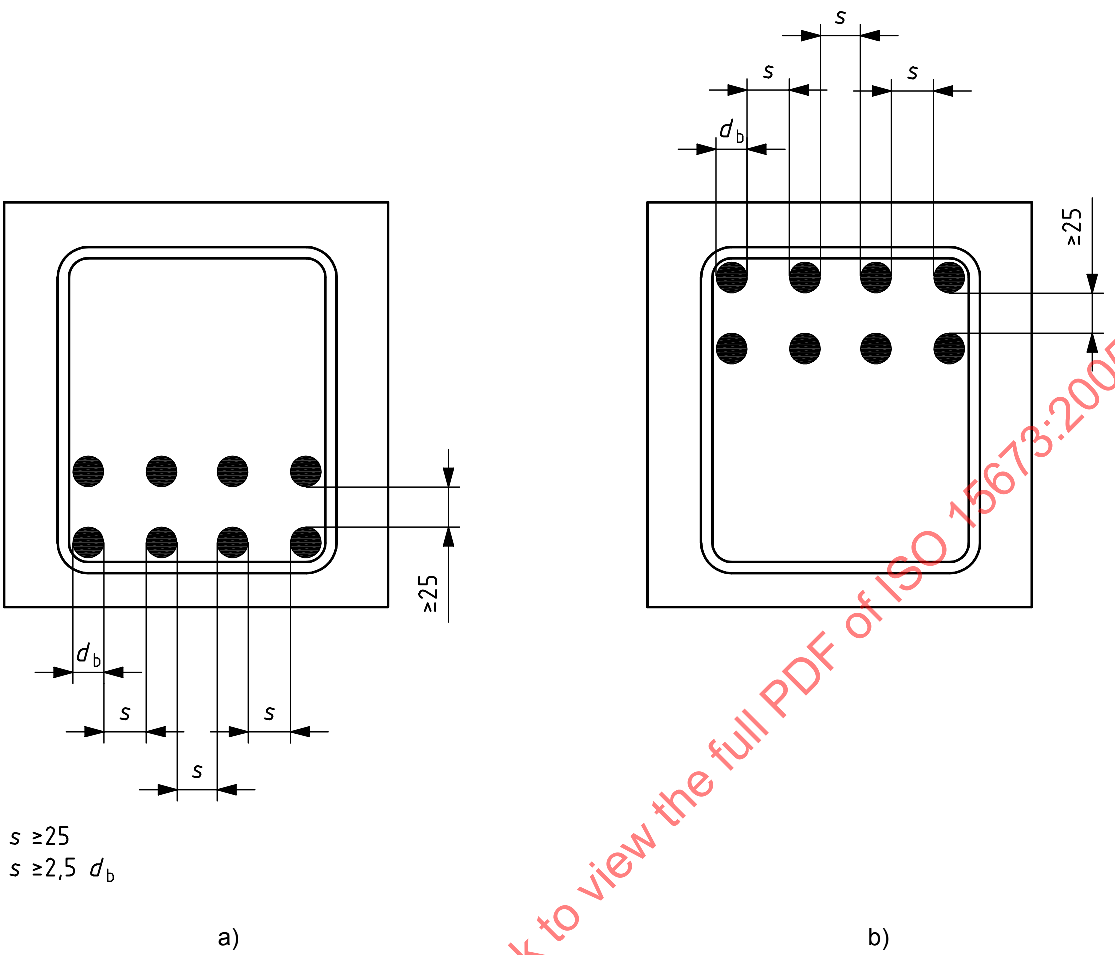
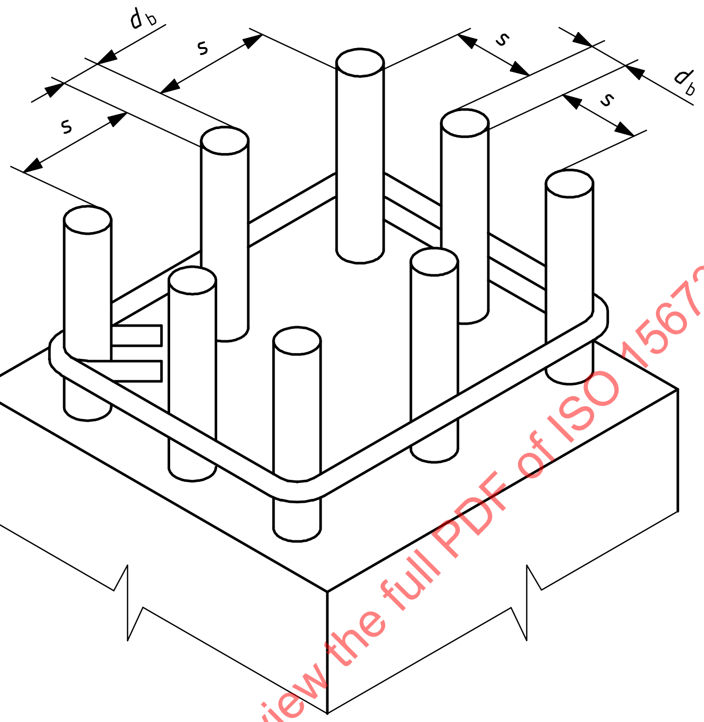


Figure 16 — Minimum clear spacing between parallel bars in a layer (a) and clear distance between parallel layers of reinforcement (b)

7.3.7.4 Minimum clear spacing between longitudinal bars in columns

In columns, the clear distance between longitudinal bars should not be less than 1,5d_b or 40 mm; see Figure 17.

Dimension in millimetres



$s \geq 40$
$s \geq 1{,}5\ d_b$

Figure 17 — Clear distance between longitudinal bars in columns

7.3.7.5 Clear spacing between parallel lap splices

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

7.3.7.6 Maximum flexural reinforcement spacing in solid slabs

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

Dimension in millimetres

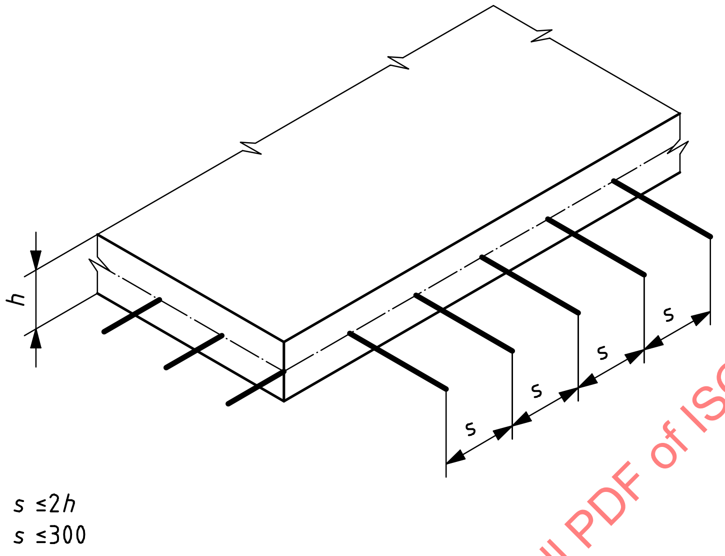


Figure 18 — Spacing between flexural reinforcement in solid slabs

7.3.7.7 Maximum shrinkage and temperature reinforcement spacing in solid slabs

Shrinkage and temperature reinforcement in slabs should be spaced no farther apart than three times the slab thickness, nor more than 300 mm; see Figure 19.

Dimension in millimetres

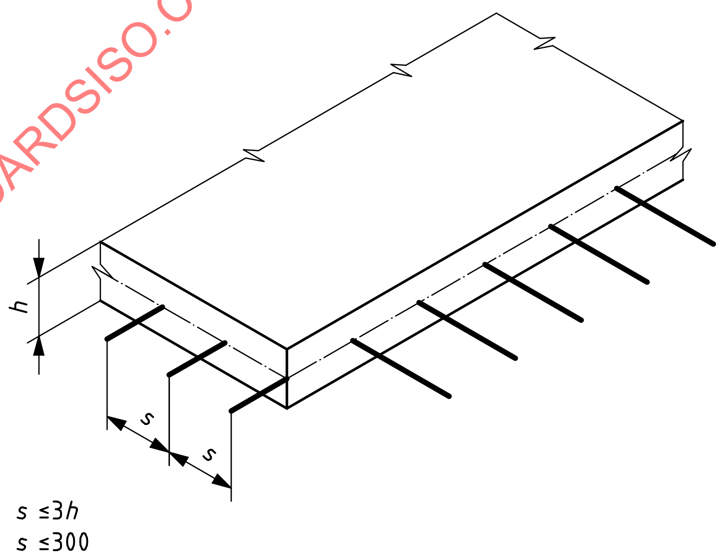


Figure 19 — Spacing between shrinkage and temperature reinforcement in slabs

7.3.7.8 Maximum reinforcement spacing in structural concrete walls

7.3.7.8.1 Vertical and horizontal reinforcement

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

7.3.7.8.2 Number of layers of reinforcement

Structural concrete walls more than 250 mm thick should have vertical and horizontal reinforcement placed in two layers parallel with faces of wall. Each layer should have approximately half of the reinforcement in that direction. The layers should be placed not 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 specified.

Dimension in millimetres

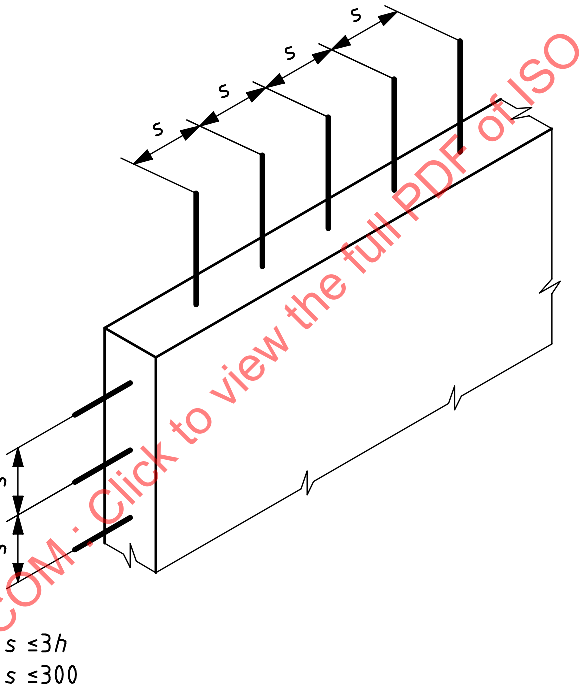


Figure 20 — Spacing between reinforcement in structural concrete walls

7.3.7.9 Special details per element type

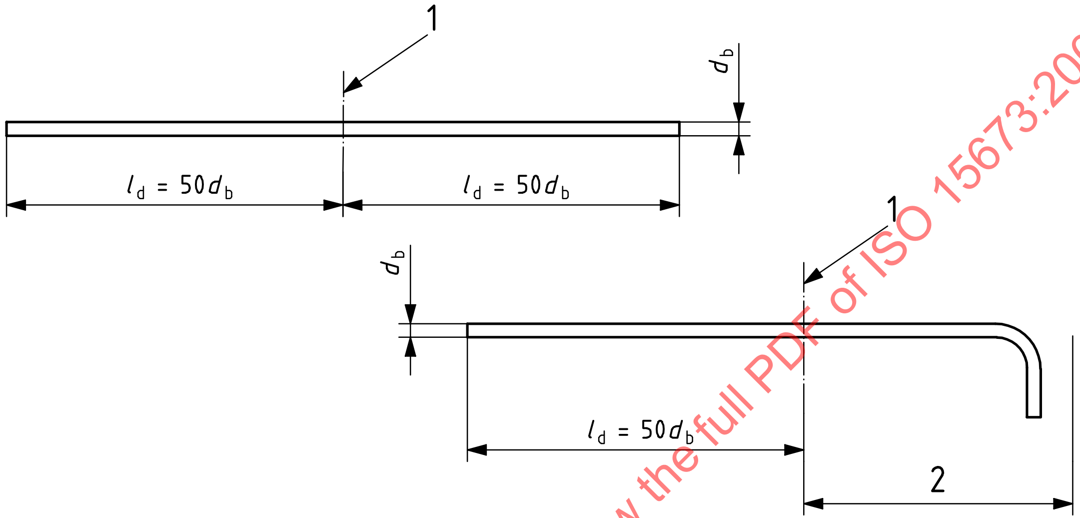
The designer should comply with the additional reinforcement details required for each individual element type, as specified in 7.3 to 7.10 of this International Standard.

7.3.8 Development length, lap splicing and anchorage of reinforcement

7.3.8.1 Development length

7.3.8.1.1 Reinforcing bars

The minimum length, l_d, of embedment required on each side of a critical section for a reinforcing bar to develop its full strength should be $50d_b$, for the bar diameters permitted by this International Standard in 7.3.3. It should be permitted to replace development length on one side of the critical section by a length of bar ending in a standard hook in accordance with the minimum anchorage distance as specified in 7.3.8.3; see Figure 21.



Key

1 critical section

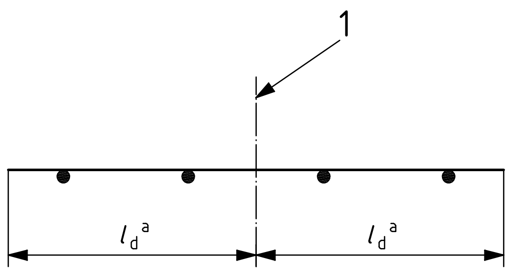
2 anchorage distance; see 7.3.8.3

Figure 21 — Required development length for reinforcing bars

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

7.3.8.1.2 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 this International Standard in 7.3.3; see Figure 22.



Key

1 critical section

a l_d = 2 cross-wires and/or ⩾ 200 mm.

Figure 22 — Required development length for welded-wire fabric

7.3.8.2 Lap splice dimensions

7.3.8.2.1 Reinforcing bars

The minimum length of a lap for the splicing of reinforcing bars should be $50d_b$ for the bar diameters permitted by this International Standard in 7.3.3; see Figure 23.

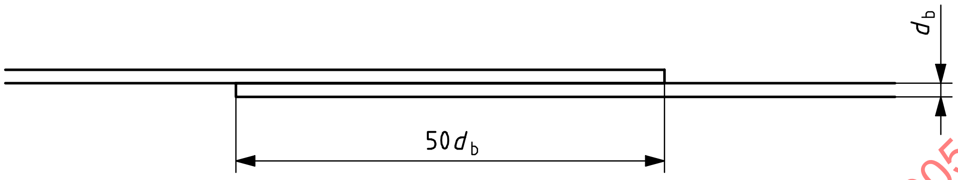
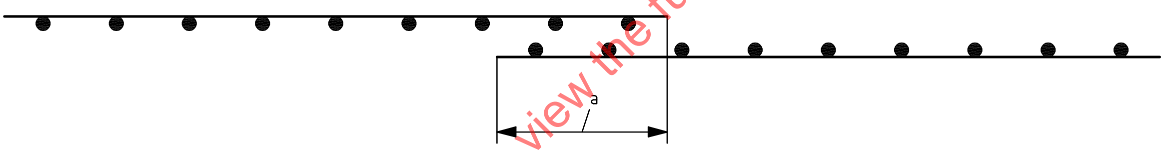


Figure 23 — Minimum lap splice length for reinforcing bars

7.3.8.2.2 Welded-wire fabric

Welded-wire fabric splicing should be attained by superimposing two cross-wires; the distance between the edge cross-wires should not be less than 250 mm for the wire diameters permitted by this International Standard in 7.3.3; see Figure 24.



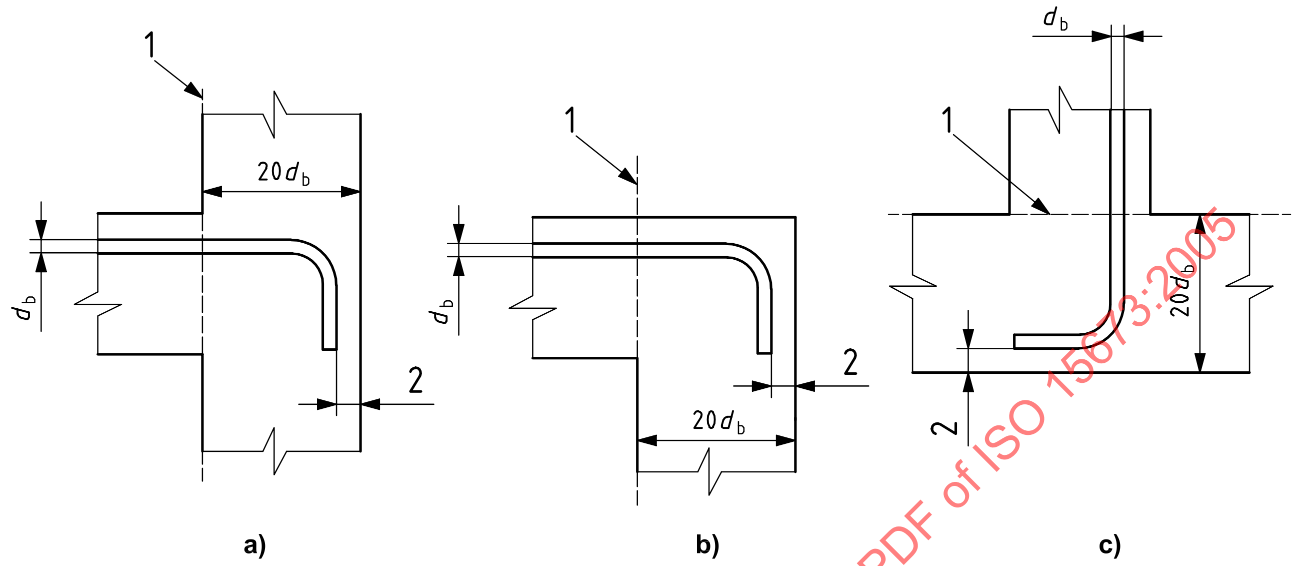
Key

a 2 cross-wires ⩾ 250 mm.

Figure 24 — Minimum lap splice length for welded-wire fabric

7.3.8.3 Minimum standard hook-anchorage distance

The minimum distance between the outer face of concrete and the critical section where the hooked bar develops its full strength should not be less than $20d_b$; see Figure 25.



Key

1 critical section

2 cover requirement

Figure 25 — Minimum standard hook-anchorage distance

7.3.9 Limits for longitudinal reinforcement

7.3.9.1 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 this International Standard should be that required to resist the factored loads and forces, but should be not less than the minimum values specified in 7.3.9. 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 7.3.9.

7.3.9.2 Solid slabs and footings

7.3.9.2.1 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 only one direction; see Figure 26. The maximum spacing for this reinforcement should be in accordance with 7.3.7.7. The following minimum ratios, ρ_t, of reinforcement area to gross concrete area should be provided for shrinkage and temperature:

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

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

7.3.9.2.2 Minimum area of tension flexural reinforcement

The minimum area, $A_{s,min}$, of tension flexural reinforcement in structural solid slabs and footings should be equal to or greater than the reinforcement area required for shrinkage and temperature stresses as specified by 7.3.9.2.1: ($A_{s,min} \geqslant \rho_t \cdot b \cdot h$); see Figure 26. The maximum spacing of this reinforcement should be in accordance with 7.3.7.6.

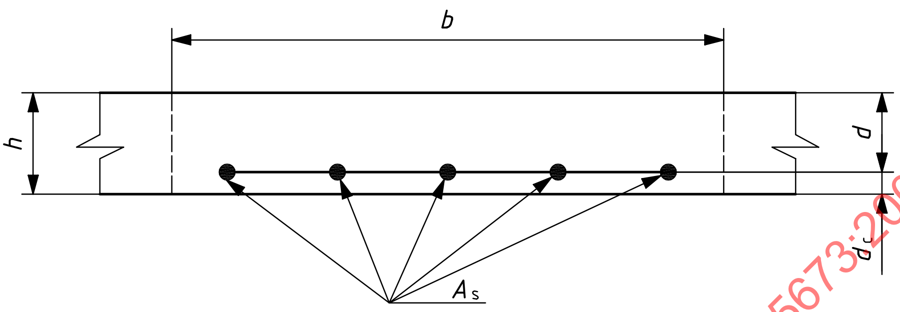


Figure 26 — Slab or footing section

7.3.9.2.3 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}, specified in Table 3. In solid slabs and footings, flexural reinforcement in compression should not be taken into account in the computation of design moment strength.

Table 3 — Maximum flexural reinforcement ratio, ρ_{max}, for solid slabs and footings

f'_c MPa	f_y MPa		
	240	**300**	**400**
15	0,016 0	0,012 0	0,008 0
20	0,022 0	0,016 0	0,011 0
25	0,027 0	0,020 0	0,014 0
30	0,032 0	0,024 0	0,016 0
It should be permitted to interpolate for different values of f_y and f'_c.			

7.3.9.3 Girders, beams and joists

7.3.9.3.1 Minimum area of tension flexural reinforcement

At every section of a girder, beam or joist where tension flexural reinforcement is required by 7.7, the minimum area, $A_{s,min}$, of tension flexural reinforcement should be greater or equal to the values calculated in Equations (13) to (15):

a) for rectangular sections and for T-sections where the flange is in compression; see Figure 27:

$$A_{s,min} = \rho_{min} \cdot d \cdot b_w \tag{13}$$

where ρ_{min}, is the value specified in Table 3.

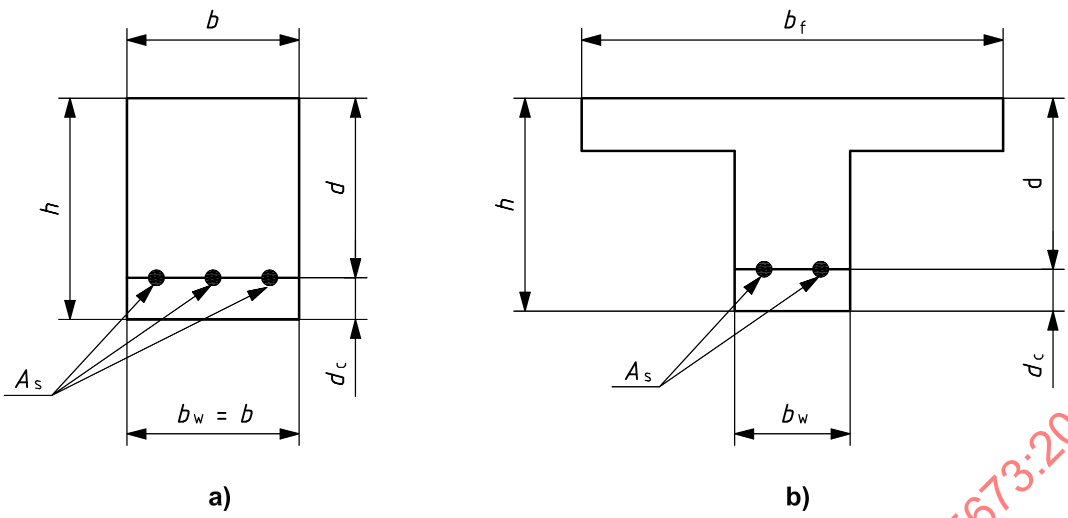


Figure 27 — Rectangular section (a) and T-shaped section (b) with flange in compression

b) for T-sections where the flange is in tension (see Figure 28), it should be equal to or greater than the smaller of the values calculated in accordance with Equations (14) and (15):

$$A_{s,min} = 2 \cdot \rho_{min} \cdot d \cdot b_w \tag{14}$$

$$A_{s,min} = \rho_{min} \cdot d \cdot b_f \tag{15}$$

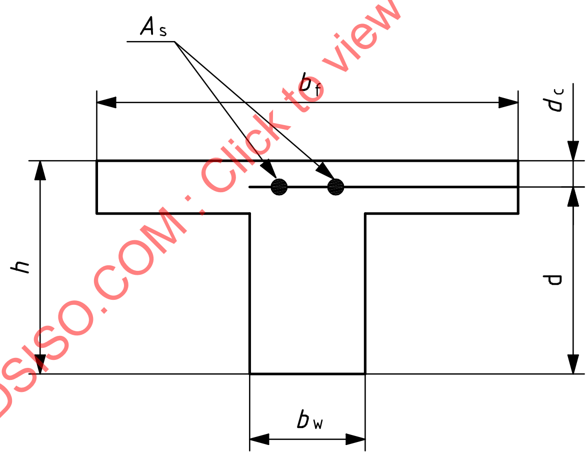


Figure 28 — T-shaped section with flange in tension

Table 4 — Minimum flexural reinforcement ratio, ρ_{min}, for girders, beams and joists

f'_c MPa	f_y MPa		
	240	**300**	**400**
15	0,004 0	0,003 2	0,002 4
20	0,004 7	0,003 7	0,002 8
25	0,005 2	0,004 2	0,003 1
30	0,005 7	0,004 6	0,003 4
It should be permitted to interpolate for different values of f_y and f'_c or use the following equation: $\rho_{min} \geqslant 0{,}25 \frac{\sqrt{f'_c}}{\sqrt{f_y}} \geqslant \frac{1{,}4}{f_y}$			

7.3.9.3.2 Maximum flexural reinforcement ratios

The ratio, ρ, of tension flexural reinforcement should not exceed the values expressed as a function of ρ_{max} as specified in Table 5 and calculated in accordance with Equations (16) and (17):

a) in girders, beams and joists having only tension flexural reinforcement:

$$\rho = \frac{A_s}{b \cdot d} \leqslant \rho_{max} \tag{16}$$

b) in girders, beams and joists having tension and compression flexural reinforcement; see Figure 29:

$$\rho - \rho' = \frac{A_s - A'_s}{b \cdot d} \leqslant \rho_{max} \tag{17}$$

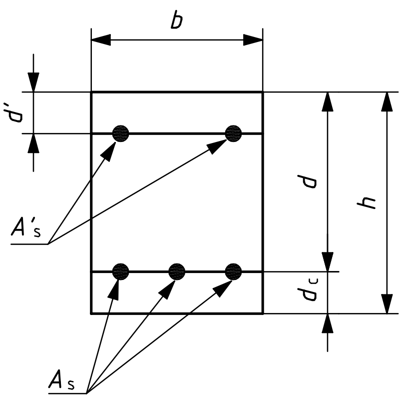


Figure 29 — Section with tension and compression reinforcement

Table 5 — Maximum flexural reinforcement ratio, ρ_{max}, for girders, beams and joists

f'_c MPa	f_y MPa		
	240	**300**	**400**
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
It should be permitted to interpolate for different values of f_y and f'_c or use the following equation: $\rho_{max} \geqslant 0{,}55 \frac{f'_c}{f_y} \cdot \frac{600}{600 + f_y}$			

7.3.9.4 Columns

7.3.9.4.1 Minimum and maximum area of longitudinal reinforcement

The total area, A_{st}, of longitudinal reinforcement for columns should not be less than 0,01 nor more than 0,06 times the gross area, A_g, of the section, in accordance with Equation (18):

$$0{,}01 \leqslant \rho_t = \left(\frac{A_{st}}{A_g} \right) \leqslant 0{,}06 \tag{18}$$

7.3.9.4.2 Minimum diameter of longitudinal bars

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

7.3.9.4.3 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.

7.3.9.4.4 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.

7.3.9.5 Structural concrete walls

7.3.9.5.1 Minimum area of vertical reinforcement

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

7.3.9.5.2 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 specified for columns in 7.3.10.4.1 and in accordance with Equation (19):

$$0{,}002\,5 \leqslant \rho_v = \left(\frac{A_{st}}{b_w \cdot l_w} \right) \leqslant 0{,}06 \tag{19}$$

7.3.10 Minimum amounts of transverse reinforcement

7.3.10.1 General

Transverse reinforcement in reinforced-concrete structural elements should be provided to resist shear, diagonal tension and torsion stresses. It should be provided also to counteract the tendency of compression-loaded bars to buckle out of the concrete by bursting the thin outer concrete cover and to prevent displacement of the longitudinal reinforcement during construction operations. In seismic zones, it should be placed in special regions of the structural elements to provide confinement of concrete subjected to stresses in the non-linear range. The amount of transverse reinforcement employed in the structural elements covered by this International Standard should be that required to resist the factored loads, forces and stresses, but should be not less than the minimum values specified by 7.3.10. The dimensions of the structural element should be appropriately modified when the calculated amount of reinforcement required to resist the factored loads, forces and stresses exceeds the maximum amounts permitted by 7.3.10.

7.3.10.2 Slabs

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

7.3.10.3 Girders, beams and joists

7.3.10.3.1 Minimum transverse reinforcement

The minimum transverse reinforcement in girders, beams and joist should be that required for shear, as specified in 7.3.13.4.3 and 7.3.13.4.4, with the exceptions noted in 7.3.10.3.2.

7.3.10.3.2 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 specified in 7.8.

7.3.10.4 Columns

All columns should have transverse reinforcement in the form of either tie reinforcement or spiral reinforcement in accordance with 7.3.10.4.1 or 7.3.10.4.2, respectively.

7.3.10.4.1 Ties

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

a) All longitudinal columns bars should be enclosed by lateral ties made with bars at least 8 mm in diameter ($d_b \geqslant 8$ 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 30.

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 30.

d) The vertical spacing, s, of ties should not exceed 16 longitudinal bar diameters, 48 tie-bar diameters, or the smallest dimension of the column section; see Figure 31.

e) The first tie should be located a distance of one-half the 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 the tie spacing below the lowest horizontal reinforcement of shallowest member supported above.

Dimensions in millimetres

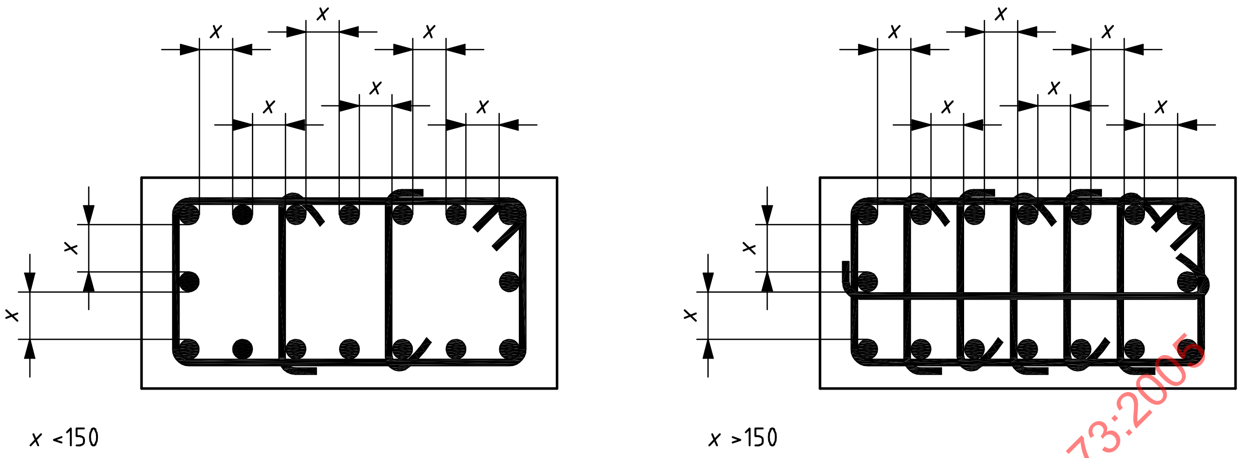
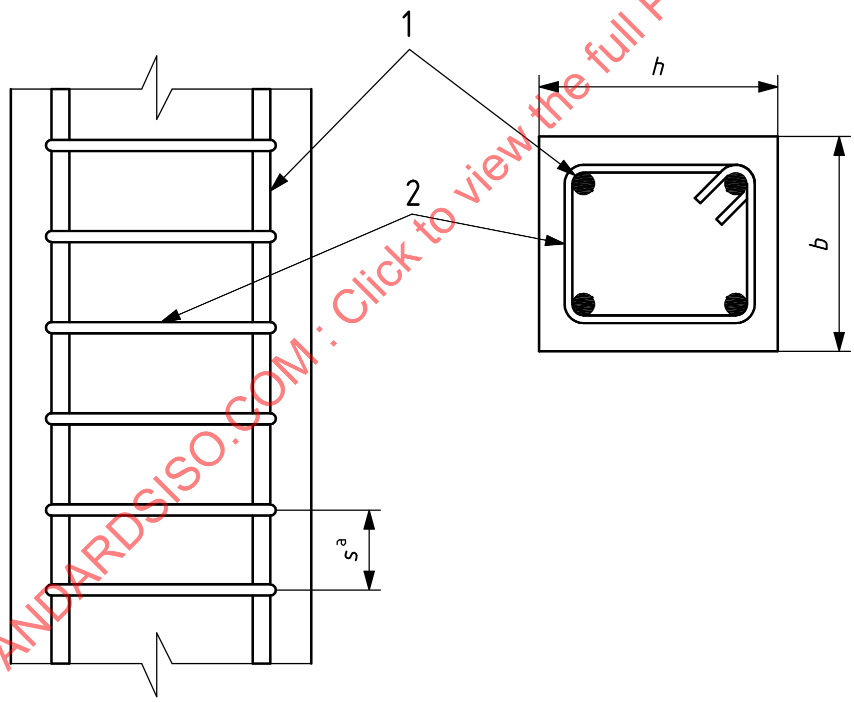


Figure 30 — Arrangement of ties in a tied column section



Key

1 longitudinal bars

2 tie

a Restrictions: s does not exceed 16 d_b longitudinal bar, 48 d_b tie bar, or b.

Figure 31 — Vertical spacing of ties in a tied column

7.3.10.4.2 Spirals

Columns with spiral reinforcement should be in accordance with the following provisions.

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

b) Clear spacing between spirals should not exceed 80 mm nor be less than 25 mm, and should be in accordance with 7.3.7.

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

d) Splices in spiral reinforcement should be in accordance with 7.3.8.

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) The ratio, ρ_s, of spiral reinforcement is defined as the 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 and should be not less than either of the values calculated in accordance with Equations (20) and (21); see Figure 32:

$$\rho_s = \frac{A_b \cdot \pi \cdot d_c}{A_c \cdot s} \geqslant 0{,}12 \cdot \frac{f'_c}{f_{ys}} \tag{20}$$

where

A_b is the area of the bar of spiral;

d_c is the centre-to-centre diameter of the spiral;

s is the vertical spacing of the spiral;

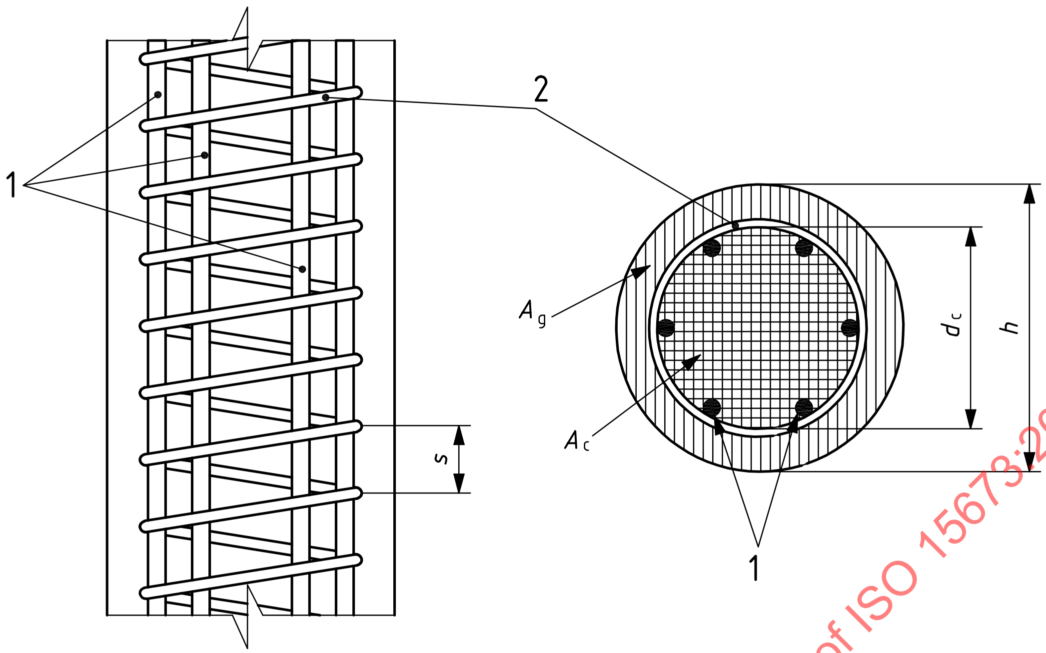
A_c is the area of the confined column core measured centre-to-centre of the spiral ($A_c = \frac{\pi \cdot d_c^2}{4}$);

f'_c is the specified concrete strength of the column;

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

$$\rho_s = \frac{A_b \cdot \pi \cdot d_c}{A_c \cdot s} \geqslant 0{,}45 \cdot \left(\frac{A_g}{A_c} - 1\right) \cdot \frac{f'_c}{f_{ys}} \tag{21}$$

where A_g is the gross column section area and the other terms are the same as for Equation (20).



Key

1 longitudinal bars

2 spiral

Figure 32 — Spiral reinforcement of column

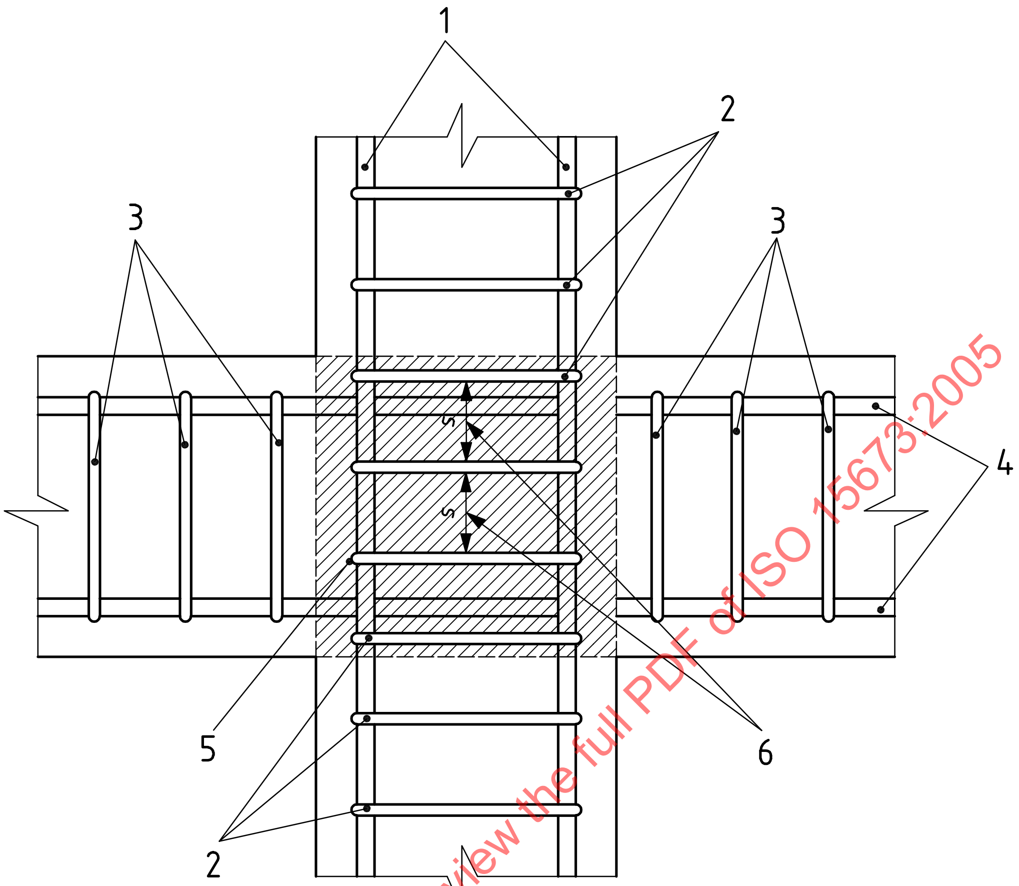
7.3.10.4.3 Column-girder joints

At joints of frames where columns and girders meet, a minimum of three column ties, in accordance with 7.3.10.4.2 a) to 7.3.10.4.2 c), should be provided within the joint and the maximum vertical spacing between ties should be 150 mm. As many ties as necessary to be in accordance with the maximum spacing should be provided; see Figure 33.

7.3.10.5 Structural concrete walls

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

Dimension in millimetres



Key

1 column longitudinal reinforcement
2 column ties
3 girder stirrups
4 girder longitudinal reinforcement
5 joint
6 joint ties
s $\leqslant$ 150

Figure 33 — Column ties in column-girder joints

7.3.11 Strength of members subjected to flexural moments

7.3.11.1 General

Calculation of the design strength of member sections subjected to flexural moments should be performed in accordance with 7.3.11. If the factored axial compressive load, P_u, on the member exceeds ($0{,}10 \cdot f'_c \cdot A_g$), the calculation of the design strength should be performed in accordance with 7.3.12.

7.3.11.2 Factored flexural moment at section

The factored flexural moment, M_u, for a section caused by the factored loads applied to the structure should be determined for the particular element type in accordance with 7.5 to 7.10.

7.3.11.3 Minimum design flexural moment strength

The design flexural moment strength, ($\phi \cdot M_n$), of the section should be equal to or greater than the factored flexural moment, M_u, at that section in accordance with Equation (22):

$$\phi \cdot M_n \geqslant M_u \tag{22}$$

7.3.11.4 Design moment strength for rectangular sections with only tension reinforcement

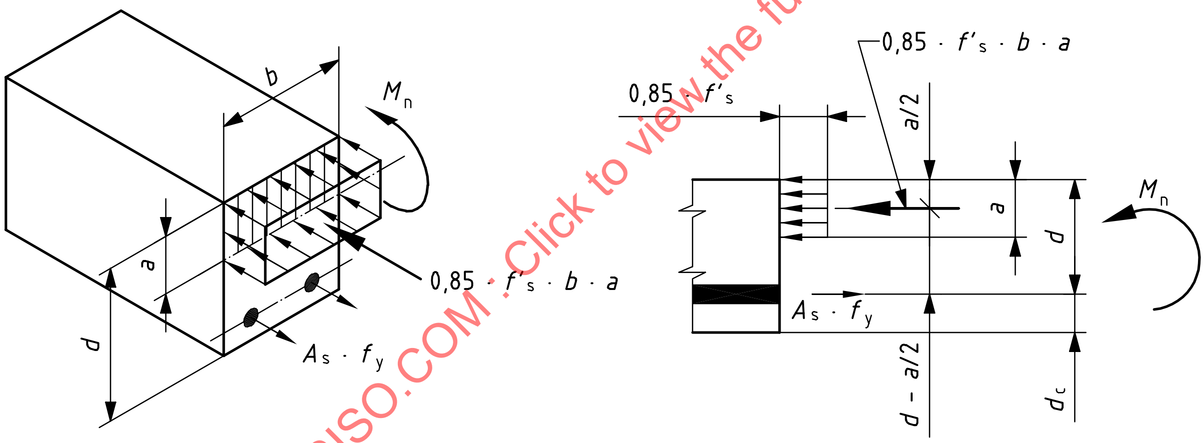
7.3.11.4.1 Design moment strength

For a section with only tension reinforcement, the design moment strength at the section should be calculated in accordance with Equation (23):

$$\phi \cdot M_n = \phi \cdot A_s \cdot f_y \left(d - \frac{a}{2} \right) \tag{23}$$

where the depth, a, of the equivalent uniform stress block should be calculated in accordance with Equation (24); see Figure 34:

$$a = \frac{A_s \cdot f_y}{0{,}85 \cdot f_c' \cdot b} \tag{24}$$



a) b)

Figure 34 — Flexural nominal moment strength

It should be permitted to use Equation (25), analogous to Equation (23) but where the value of a has been replaced by the expression given in Equation (24):

$$\phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot d \cdot \left(1 - 0{,}59 \cdot \frac{A_s \cdot f_y}{b \cdot d \cdot f_c'} \right) \tag{25}$$

For the purposes of this International Standard, it should be permitted to approximate the design moment strength in slabs, and also in girders, beams and joists where $\rho < \frac{\rho_{max}}{2}$, with ρ_{max} from Table 5 in accordance with Equation (26):

$$\phi \cdot M_n \approx \phi \cdot A_s \cdot f_y \cdot 0{,}85 \cdot d \tag{26}$$

7.3.11.4.2 Obtaining the flexural tension reinforcement area

The required ratio, $\rho = \frac{A_s}{(b \cdot d)}$, of flexural reinforcement should be derived by combining Equation (22) with Equation (23), and using the factored flexural moment, M_u, in accordance with Equation (27):

$$\rho = \frac{A_s}{b \cdot d} = \alpha - \sqrt{\alpha^2 - \left(\frac{M_u}{\phi \cdot b \cdot d^2} \cdot \frac{2 \cdot \alpha}{f_y} \right)} \quad (27)$$

where

$$\alpha = \frac{f'_c}{1{,}18 \cdot f_y} \quad (28)$$

or by using the approximate Equation (26), in slabs where $\rho < \rho_{max}$, with ρ_{max} from Table 3, and in girders, beams and joists where $\rho < \frac{\rho_{max}}{2}$, with ρ_{max} from Table 5, in accordance with Equation (29):

$$\rho = \frac{A_s}{b \cdot d} \approx \frac{M_u}{\phi \cdot b \cdot d^2 \cdot 0{,}85 \cdot f_y} \quad (29)$$

In Equations (27) and (29), $\phi = [0{,}90]$; see 6.3.3 a). If the value derived from Equation (27) or (29) is smaller than ρ_{min} from 7.3.9.3.1, ρ should be increased to that value. If the value of ρ derived for slabs is greater than ρ_{max} from Table 3, the slab depth, h, should be increased, correcting the selfweight of the slab. If the value of ρ derived for girders, beams and joists is greater than ρ_{max} from Table 5, the possibility of either using compression reinforcement (see 7.3.11.5) or changing the dimensions and making the appropriate correction for the selfweight should be investigated.

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

7.3.11.5.1 Tension reinforcement less than maximum

If the ratio, ρ, of tension reinforcement is less than ρ_{max} as specified in 7.3.9.3.2, it should be permitted to disregard the effect of the reinforcement in the compression face of the element.

7.3.11.5.2 Shallow doubly reinforced sections

If the ratio of $\frac{d'}{d}$ is greater than the values specified in Table 6, the compression reinforcement should be considered not to be effective.

Table 6 — Maximum values of $\frac{d'}{d}$ for compression reinforcement to be effective

f_y MPa	$\frac{d'}{d}$
240	0,320
300	0,250
400	0,150
It should be permitted to interpolate for different values of f_y	

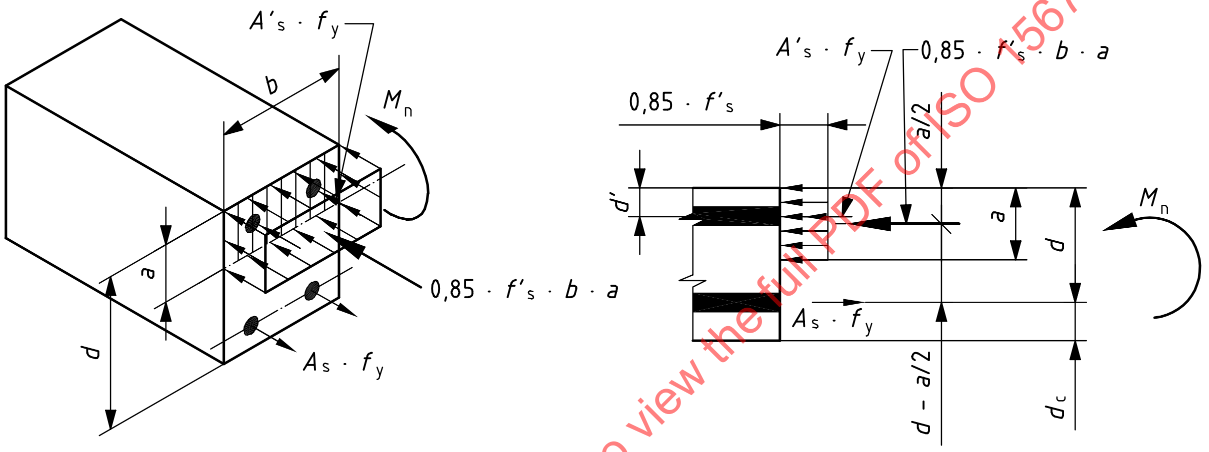
7.3.11.5.3 Design moment strength of sections with compression reinforcement

When the condition of $\frac{d'}{d}$ is met, the design moment strength at the section should be in accordance with Equation (30); see Figure 35:

$$\phi \cdot M_{\text{n}} = \phi \cdot \left[\left(A_{\text{s}} - A'_{\text{s}} \right) \cdot f_{\text{y}} \left(d - \frac{a}{2} \right) + A'_{\text{s}} \cdot f_{\text{y}} \cdot \left(d - d' \right) \right] \tag{30}$$

where the depth, a, of the equivalent uniform stress block should be in accordance with Equation (31):

$$a = \frac{\left(A_{\text{s}} - A'_{\text{s}} \right) \cdot f_{\text{y}}}{0{,}85 \cdot f'_{\text{c}} \cdot b} \tag{31}$$



a) b)

Figure 35 — Flexural nominal moment strength for doubly reinforced sections

7.3.11.5.4 Obtaining the flexural tension and compression reinforcement area

The required area, A_{s}, of flexural tension reinforcement and area, A'_{s}, of compression reinforcement should be calculated by combining Equation (22) with Equation (30), and using the factored flexural moment, M_{u}, in order to derive Equations (32) and (33):

$$A'_{\text{s}} = \frac{M_{\text{u}}}{\phi \cdot f_{\text{y}} \cdot \left(d - d' \right)} - \left(b \cdot d^2 \cdot \rho_{\text{max}} \cdot f_{\text{y}} \cdot 0{,}8 \right) \tag{32}$$

$$A_{\text{s}} = A'_{\text{s}} + \rho_{\text{max}} \cdot b \cdot d \tag{33}$$

In Equations (32) and (33), $\phi = [0{,}90]$; see 6.3.3 a). The steel ratio, ρ_{max}, should be derived from Table 5. This procedure should be used only when the condition of $\frac{d'}{d}$ is in accordance with 7.3.11.5.2. Compression reinforcement should be enclosed by ties as specified by 7.3.10.3.2.

7.3.11.6 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 for the beam and the flexural design should be in accordance with the requirements of 7.3.11.6.1 to 7.3.11.6.5.

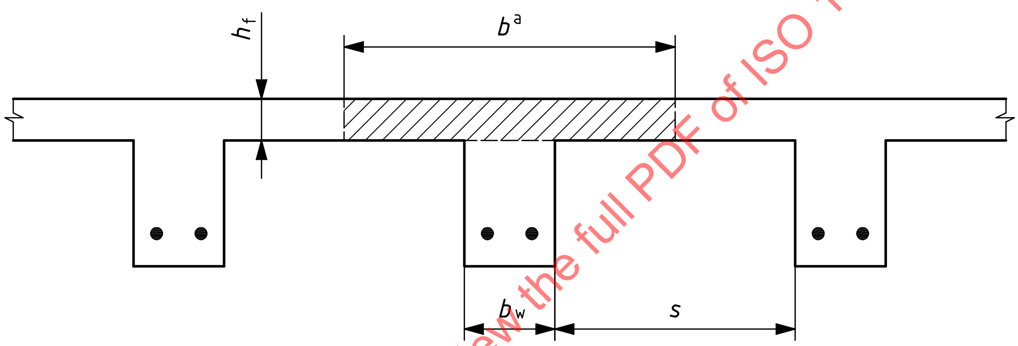
7.3.11.6.1 Effective flange width for beams with a slab on both sides

The width, b, of a slab functioning as a T-beam flange should not exceed the following dimensions; see Figure 36:

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

b) 16 times the slab thickness, h_f, plus the web thickness, b_w;

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



Key

a Restrictions: b is the lesser of 1/4 the span length; $16h_f + b_w$; or $s + b_w$.

Figure 36 — Effective flange width for T-beams with a slab on both sides

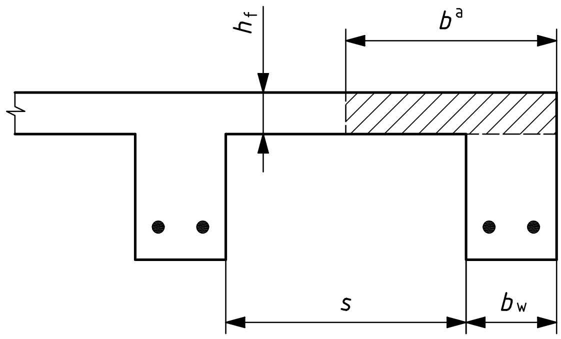
7.3.11.6.2 Effective flange width for beams with a slab on only one side

The width, b, of slab functioning as a T-beam flange should not exceed the following dimensions; see Figure 37):

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

b) six times the slab thickness, h_f, plus the web thickness, b_w;

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



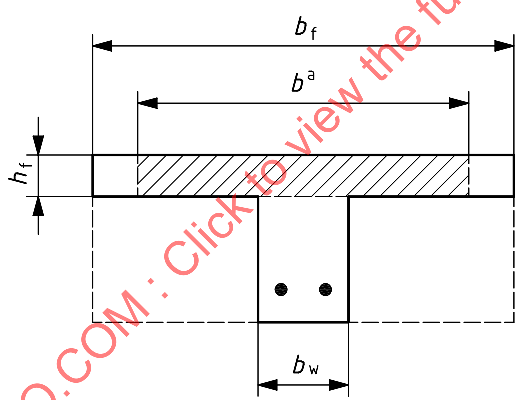
Key

a The value of b does not exceed 1/12 the span length + b_w; $6h_f + b_w$; or $s/2 + b_w$.

Figure 37 — Effective flange width for T-beams with a slab on only one side

7.3.11.6.3 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 $4b_w$ or b_f; see Figure 38.



Key

a The value of b does not exceed $4b_w$ or b_f.

Figure 38 — Effective flange width for isolated T-beams

7.3.11.6.4 Design moment strength of T-beams

When the flange is in compression, the moment strength should be calculated as for a rectangular beam in accordance with 7.3.11.4.1, as long as the depth, a, of the equivalent uniform stress block lies within the flange thickness, h_f; see Figure 29. The last condition should be verified using Equation (34).

$$h_f \geqslant a \tag{34}$$

where $a = \dfrac{A_s \cdot f_y}{0{,}85 \cdot f'_c \cdot b}$.

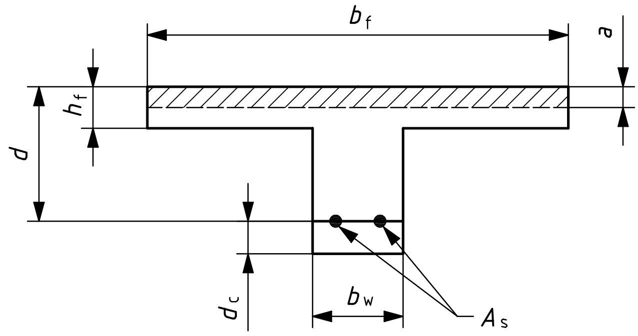


Figure 39 — Effective cross section for moment strength calculation of T-beams

7.3.11.6.5 Obtaining the flexural tension reinforcement area

The required ratio, $\rho < \frac{A_s}{(b \cdot d)}$, of flexural reinforcement for T-beams should be calculated in accordance with Equation (27) or (29), and the flexural reinforcement ratio, ρ, should not exceed the value calculated in accordance with Equation (35) in order for the depth, a, of the equivalent uniform stress block to lie within the flange thickness, h_f.

$$\rho \geqslant \frac{0{,}85 \cdot f_c' \cdot h_f}{f_y \cdot d} \tag{35}$$

If the value derived from Equation (27) or (29) is smaller than ρ_{min} derived in accordance with 7.3.9.3.1, ρ should be increased to that value. If the derived value of ρ is greater than ρ_{max} from Table 5, the dimensions should be changed, making the appropriate correction for the selfweight.

7.3.12 Strength of members subjected to axial loads with or without flexure

7.3.12.1 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 in accordance with the requirements of 8.3.12.

7.3.12.2 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 in accordance with the provisions of 7.5 to 7.10.

7.3.12.3 Design strength for axial compression

7.3.12.3.1 Design strength for axial compression without flexure

Equation (36) should be used to determine the design axial strength, $\phi \cdot P_{0n}$, for axial compression without flexure:

$$\phi \cdot P_{0n} = \phi \cdot \left[0{,}85 \cdot f_c' \cdot \left(A_g - A_{st} \right) + A_{st} \cdot f_y \right] \tag{36}$$

In Equation (36), $\phi = [0{,}70]$ for columns with ties and structural concrete walls and $\phi = [0{,}75]$ for columns with spiral reinforcement; see 6.3.3 c).

7.3.12.3.2 Maximum design axial load strength

The design strength, $\phi \cdot P_n$, for axial load in columns and structural concrete walls subjected to compression, with or without flexure, should not be taken greater than that calculated in accordance with Equations (37) and (38):

a) columns with ties and structural concrete walls:

$$\phi \cdot P_{n(max)} \leqslant 0{,}80 \cdot \phi \cdot P_{0n} \tag{37}$$

where $\phi = [0{,}70]$;

b) columns with spiral reinforcement:

$$\phi \cdot P_{n(max)} \leqslant 0{,}85 \cdot \phi \cdot P_{0n} \tag{38}$$

where $\phi = [0{,}75]$.

7.3.12.4 Balanced strength for axial compression with flexure

7.3.12.4.1 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 in accordance with Equations (39) and (40), respectively. However, these equations only apply to rectangular columns with symmetrical reinforcement.

$$\phi \cdot P_{bn} = \phi \cdot 0{,}42 \cdot f'_c \cdot h \cdot b \tag{39}$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0{,}32 \cdot h + \phi \cdot \left(0{,}6 \cdot A_{se} + 0{,}15 \cdot A_{ss}\right) \cdot f_y \cdot \left(\frac{h}{2} - d'\right) \tag{40}$$

For Equation (40), 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 40. In Equations (39) and (40), $\phi = [0{,}70]$; see 6.3.3 c).

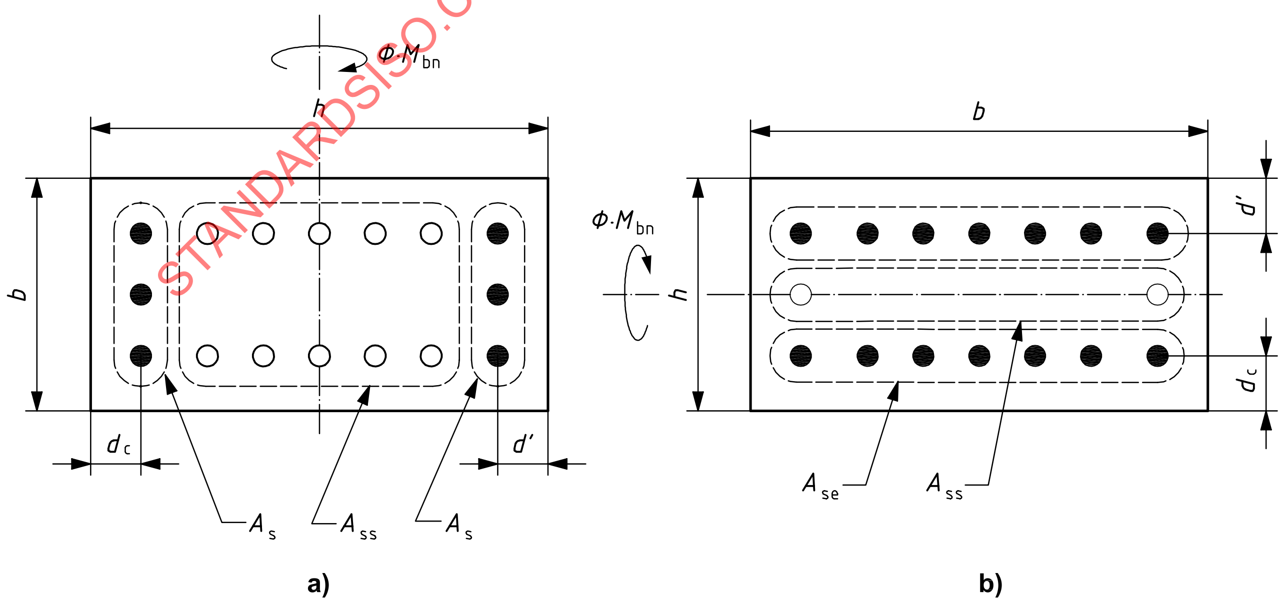


Figure 40 — Dimensions for calculation of balanced moment design strength

7.3.12.4.2 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 calculated in accordance with Equations (41) and (42), respectively:

$$\phi \cdot P_{bn} = \phi \cdot 0{,}5 \cdot f'_c \cdot A_c \tag{41}$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0{,}2 \cdot h + \phi \cdot 0{,}6 \cdot A_{st} \cdot f_y \cdot \left(\frac{h}{2} - d'\right) \tag{42}$$

For Equation (41), h should be taken as the diameter of the section of the column. In Equations (41) and (42), $\phi = [0{,}75]$; see 6.3.3 c).

7.3.12.5 Design strength for axial tension without flexure

The design strength, $\phi \cdot P_{tn}$, for axial tension without flexure should be determined in accordance with Equation (43):

$$\phi \cdot P_{tn} = \phi \cdot A_{st} \cdot f_y \tag{43}$$

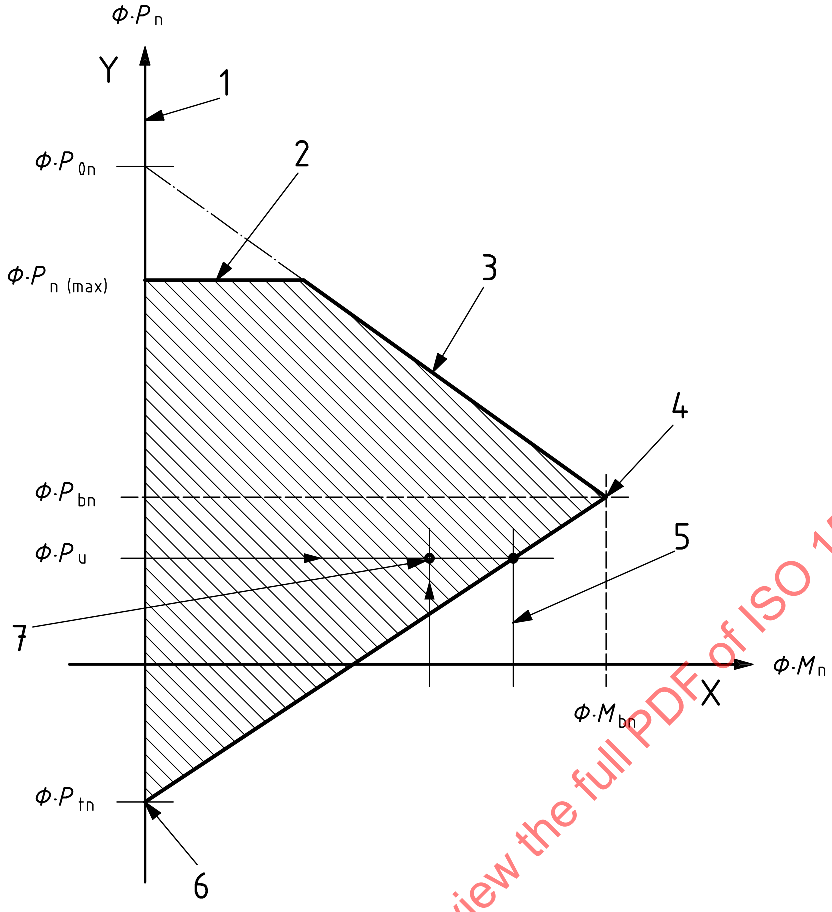
In Equation (43), $\phi = [0{,}90]$; see 6.3.3 b).

7.3.12.6 Minimum design combined axial load and moment strength

The design moment strength, $\phi \cdot M_n$, at the section of the element at the level of applied factored axial load, P_u, should be equal to or greater than the greater factored flexural moment, M_u, that can accompany the factored axial load, P_u, in accordance with Equation (44):

$$\phi \cdot M_n \geqslant M_u \tag{44}$$

Compliance with Equation (44) should be established by proving that the coordinates of (M_u, P_u) are inside the interaction design-strength surface of a moment vs. axial load interaction diagram relating $\phi \cdot M_n$ and $\phi \cdot P_n$, as indicated by the hatched area in Figure 41.



Key

X moment

Y axial load

1 design strength for axial compression

2 maximum allowable axial compression load

3 interaction design strength point

4 balance design strength point

5 design moment strength at factored axial load level, P_u

6 design strength for axial tension

7 required factored axial load and moment

Figure 41 — Interaction diagram for ($\phi \cdot M_n$, $\phi \cdot P_n$)

The conditions specified in Equations (45) to (48) should be met for all couples of P_u and M_u that act on the column section:

$$P_u \leqslant \phi \cdot P_{n(max)} \tag{45}$$

$$P_u \geqslant -(\phi \cdot P_{tn}) \tag{46}$$

For values of $P_u \geqslant \phi \cdot P_{bn}$:

$$M_u \leqslant \phi \cdot M_n = \frac{(\phi \cdot P_{0n}) - P_u}{(\phi \cdot P_{0n}) - (\phi \cdot P_{bn})} \cdot (\phi \cdot M_{bn}) \tag{47}$$

For values of $P_u \geqslant \phi \cdot P_{bn}$:

$$M_u \leqslant \phi \cdot M_n = \frac{P_u + (\phi \cdot P_{tn})}{(\phi \cdot P_{bn}) + (\phi \cdot P_{tn})} \cdot (\phi \cdot M_{bn}) \tag{48}$$

7.3.12.7 Use of interaction diagrams

It should be permitted to use interaction diagrams for columns from authoritative sources, if the employment of the strength reduction factors, ϕ, as set forth in this International Standard is warranted.

7.3.12.8 Biaxial moment strength

Corner columns and other columns subjected to moments about each axis simultaneously should be in accordance with Equation (49):

$$\frac{(M_u)_x}{(\phi \cdot M_n)_x} + \frac{(M_u)_y}{(\phi \cdot M_n)_y} \leqslant 1{,}0 \tag{49}$$

where $(M_u)_x$ and $(M_u)_y$ correspond to the factored moments that act about the x and y axes, simultaneously with the factored axial load $P_u \cdot (\phi \cdot M_n)_x$ and $(\phi \cdot M_n)_y$ correspond to the values of the design moment strength derived from Equation (47) or (48) for the factored axial load value, P_u, and for the appropriate direction x or y.

7.3.13 Strength of members subjected to shear stresses

7.3.13.1 General

Calculation of the design strength of member sections subjected to diagonal tension or shear stresses should be performed in accordance with the requirements of 7.3.13. Two types of shear stress effects are covered by this International Standard:

a) beam-action shear that accompanies flexural moments and occurs in girders, beams, joists, solid slabs and structural concrete walls in the vicinity of supports and concentrated loads;

b) punching-shear or two-way action shear that occurs in solid slabs and footings, 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 this International Standard.

7.3.13.2 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 7.5 to 7.10.

7.3.13.3 Design shear strength

The design shear strength, $\phi \cdot V_n$, at the section of the element should be equal to or greater than the factored shear, V_u, in accordance with Equation (50):

$$\phi \cdot V_n \geqslant V_u \tag{50}$$

In Equation (50), $\phi = [0{,}85]$; see 6.3.3 d).

7.3.13.4 Beam-action shear

7.3.13.4.1 General

Members for beam-action shear should be designed in accordance with 7.3.13.4. The general considerations are specified in 7.3.13.4.1 a) and 7.3.13.4.1 b).

a) Where shear reinforcement is used, the design shear strength, $\phi \cdot V_n$, should be computed in accordance with Equation (51):

$$\phi \cdot V_n = \phi \cdot (V_c + V_s) \tag{51}$$

In Equation (51), $\phi \cdot V_c$ 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 (51) $\phi = [0,85]$; see 6.3.3 d).

b) Where support reaction in the 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, d, from the support for girders, beams, joists, columns, slabs and footings, it should be permitted to design the sections in-between for the same factored shear, V_u, as computed at d.

7.3.13.4.2 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 42, and it should be computed using Equation (52) with $\phi = [0,85]$; see 6.3.3 d).

$$\phi \cdot V_c = \phi \cdot 2 \cdot \left(\frac{\sqrt{f'_c}}{6} \right) \cdot b_w \cdot d \tag{52}$$

In Equation (52) for solid slabs and footings, b_w should be taken as the width of the section, b; see Figure 43.

7.3.13.4.3 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 computed using Equation (53):

$$\phi \cdot V_s = \phi \cdot \left(\frac{A_v \cdot f_{ys} \cdot d}{s} \right) \tag{53}$$

where

A_v is the area of shear reinforcement perpendicular to the axis of the element within a distance, s;

f_{ys} is the yield strength of the steel of the shear reinforcement.

In Equation (53) $\phi = [0,85]$; see 6.3.3 d).

The contribution of the shear reinforcement to the design shear strength should not be taken greater than that specified in Equation (54):

$$\phi \cdot V_s \leqslant \phi \cdot \left(\frac{2}{3} \cdot \sqrt{f'_c} \cdot b_w \cdot d \right) = 4 \cdot \phi \cdot V_c \tag{54}$$

Shear reinforcement for solid slabs and footings is beyond the scope of this International Standard.

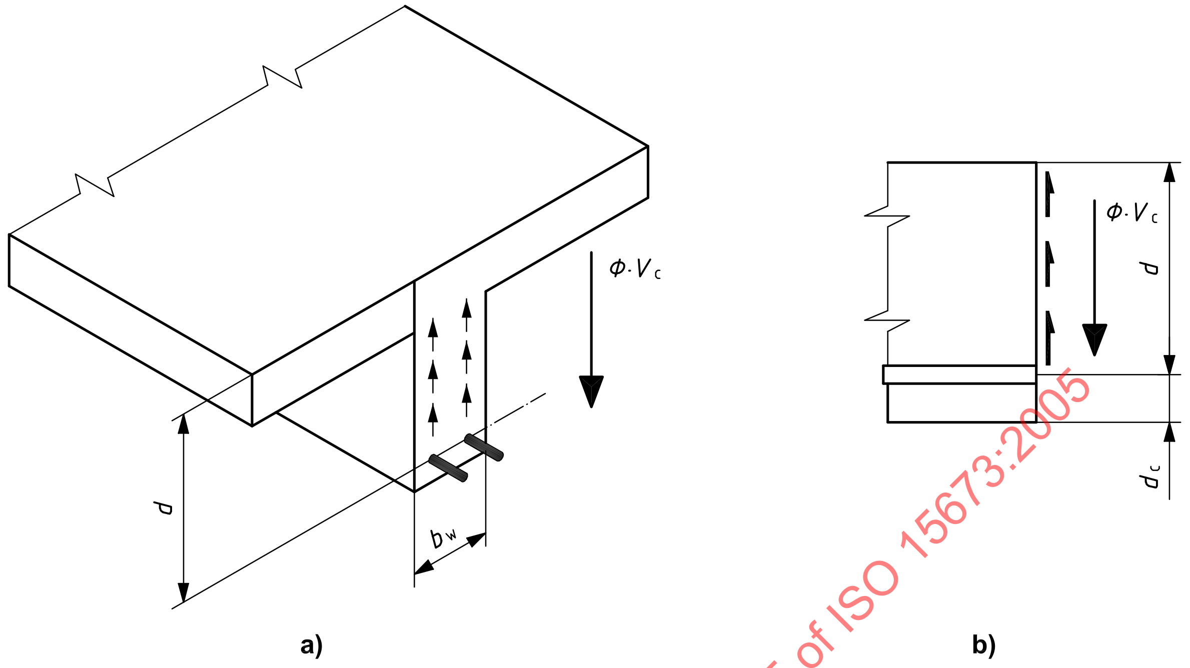


Figure 42 — Contribution of concrete to beam-action shear strength in girders, beams and joists

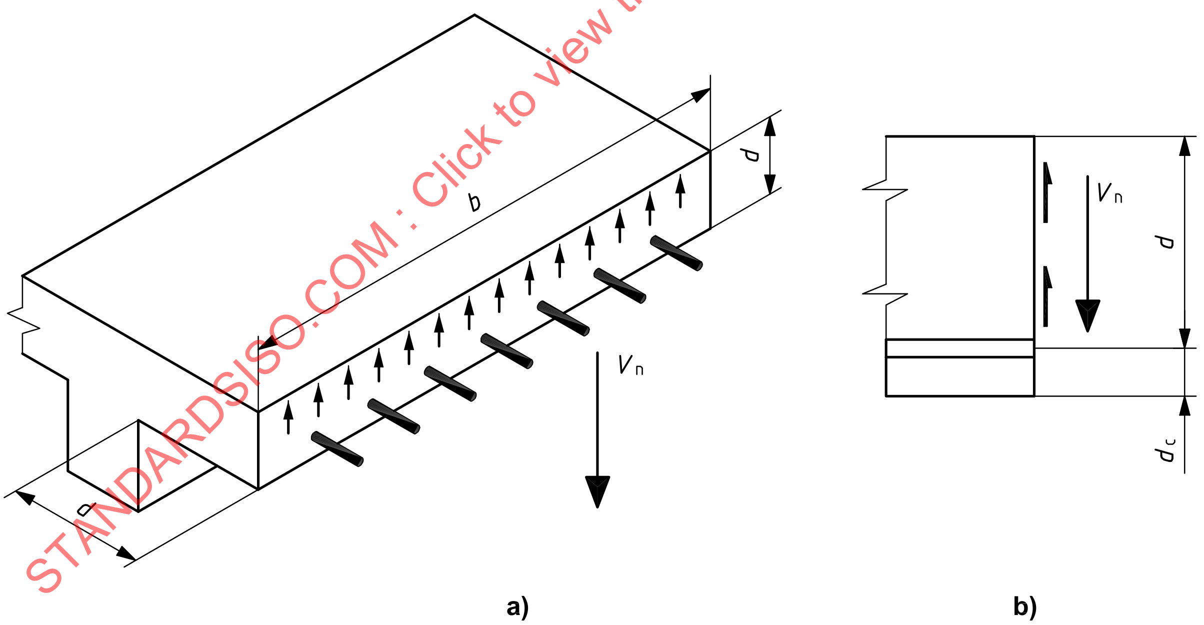


Figure 43 — Contribution of concrete to beam-action shear strength in solid slabs

7.3.13.4.4 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 as follows.

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 (55). The maximum spacing, s, along the axis of the element should not exceed d/2 or 600 mm; see Figure 44.

$$A_v = \frac{1}{16}\sqrt{f'_c}\,\frac{b_w \cdot s}{f_{ys}} \geqslant \frac{b_w \cdot s}{3 \cdot f_{ys}} \qquad (55)$$

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

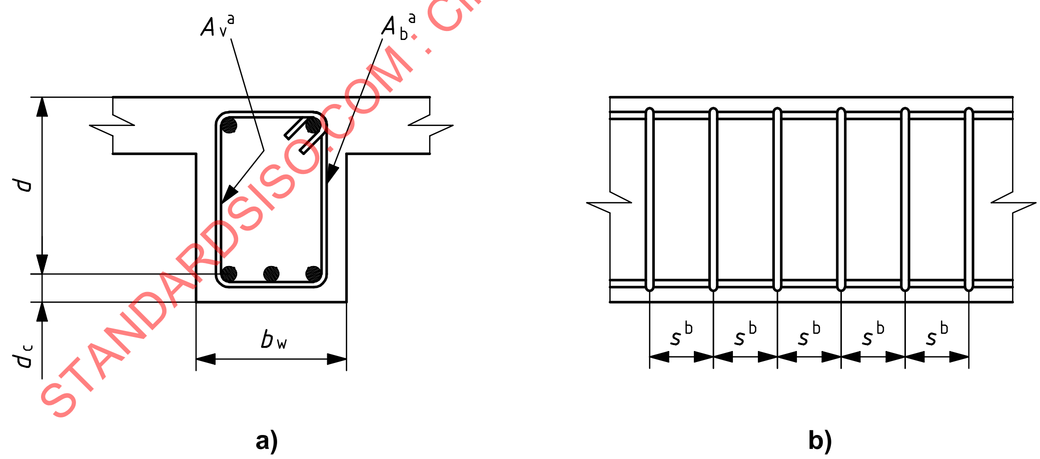
c) Where the factored shear, V_u, exceeds $\phi \cdot V_c$, the difference $(V_u - \phi \cdot V_c)$ should be provided for by shear reinforcement, using Equations (51), (52) and (53), and the following limitations should be observed; see Table 7.

1) The amount of shear reinforcement should not be less than that determined using Equation (55).

2) If the value of $\phi \cdot V_s$ calculated using Equation (53) is less than $(2 \cdot \phi \cdot V_c)$, the spacing limits of 7.3.13.4.4 b) should be employed.

3) If the value of $\phi \cdot V_s$ calculated using Equation (53) is greater than $(2 \cdot \phi \cdot V_c)$, the spacing limits should be half of the values of 7.3.13.4.4 b).

4) The value of $\phi \cdot V_s$ calculated using Equation (53) should not be greater than $(4 \cdot \phi \cdot V_c)$.



Key

a A_v is equal to the number of legs multiplied by A_b.

b $s \leqslant$ minimum is given in 7.3.13.4.4 b)

Figure 44 — Minimum shear reinforcement in girders, beams, and joists when $(\phi \times V_c/2 \leqslant V_u < \phi \times V_c)$

Table 7 — Shear reinforcement in girders, beams, and joists, maximum spacing

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

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

7.3.13.5.1 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.

7.3.13.5.2 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.

7.3.13.5.3 Two-way action shear design strength

The design shear strength should be the smallest of the values calculated from Equations (56), (57) and (58), with $\phi = [0{,}85]$; see 6.3.3 d):

$$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left(1 + \frac{2}{\beta_c}\right) \cdot \left(\frac{\sqrt{f'_c}}{6}\right) \cdot b_0 \cdot d \tag{56}$$

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

$$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left(2 + \frac{\alpha_s \cdot d}{b_0}\right) \cdot \left(\frac{\sqrt{f'_c}}{12}\right) \cdot b_0 \cdot d \tag{57}$$

where α_s is 40 for interior columns, 30 for edge columns and 20 for corner columns.

$$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left(\frac{\sqrt{f'_c}}{3}\right) \cdot b_0 \cdot d \tag{58}$$

7.3.13.6 Shear in structural concrete walls

7.3.13.6.1 General

The provisions in 7.3.13.6 should be applied to the design of structural concrete walls for shear. The following general provisions should be employed.

a) The design for shear forces perpendicular to the face of the structural concrete wall should be in accordance with the provisions in 7.3.13.4 for solid slabs. The design for shear forces in the plane of the structural concrete wall should be performed in accordance with 7.3.13.6.

b) The structural concrete wall should be continuous from the roof 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 directions, not less than the minimum values of 7.3.9.5 and 7.3.10.5, and in accordance with the maximum spacing specified in 7.3.7.8.

d) Where shear reinforcement is used, the design shear strength, $\phi \cdot V_n$, should be computed using Equation (59):

$$\phi \cdot V_n = \phi \cdot (V_c + V_s) \tag{59}$$

where $\phi \cdot V_c$ 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 (59), $\phi = [0,85]$; see 6.3.3 d).

7.3.13.6.2 Contribution of concrete to the 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 (60) with $\phi = [0,85]$; see 6.3.3 d):

$$\phi \cdot V_c = \phi \cdot \left(\frac{\sqrt{f'_c}}{6} \right) \cdot b_w \cdot l_w \tag{60}$$

where

b_w is the thickness of the web of the structural concrete wall;

l_w its horizontal length.

In Equation (61), $\phi = [0,85]$; see 6.3.3 d).

7.3.13.6.3 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 in accordance with Equation (61):

$$\phi \cdot V_s = \phi \cdot \left(\rho_h \cdot f_y \cdot b_w \cdot l_w \right) \tag{61}$$

where

ρ_h is the ratio of horizontal reinforcement;

f_y is its yield strength.

In Equation (61), $\phi = [0,85]$; see 6.3.3 d).

7.3.13.6.4 Design of shear reinforcement

Where the factored shear, V_u exceeds $\phi \cdot V_c$, the ratio of the horizontal reinforcement should not be less than the amount determined from Equation (62), with $\phi = [0,85]$; see 6.3.3 d):

$$\rho_h \geqslant \frac{V_u - \phi \cdot V_c}{\phi \cdot f_y \cdot b_w \cdot l_w} \tag{62}$$

In addition, the following requirements should be met.

a) Two curtains of reinforcement should be employed, both in the vertical and the 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 $\phi \cdot V_n$ should not exceed that calculated in accordance with Equation (63):

$$\phi \cdot V_n = \phi \cdot (V_c + V_s) \leqslant \phi \cdot \left(\frac{5}{6}\right) \cdot \sqrt{f_c'} \cdot b_w \cdot l_w \tag{63}$$

7.3.13.7 Torsion

Design for torsion is beyond the scope of this International Standard, and it should be permitted to neglect torsion effects when the calculated factored torsion, T_u, is less than the value derived from Equation (64):

$$T_u \leqslant \phi \cdot \left(\frac{\sqrt{f_c'}}{24}\right) \cdot \left(\frac{h^2 \cdot b^2}{h + b}\right) \tag{64}$$

Notwithstanding, in members where torsion smaller than the value given by Equation (64) 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 $d/4$ of the smaller element, for a distance equal to 1/4 of the clear span of the element measured from the internal face of each support. In Equation (64), $\phi = [0,85]$; see 6.3.3 d).

7.3.14 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 ($\phi \cdot P_n$) calculated in accordance with Equation (65):

$$\phi \cdot P_n = \phi \cdot 0{,}85 \cdot f_c' \cdot A_c \tag{65}$$

where A_c corresponds to the contact area, expressed in square millimetres and $\phi = [0,70]$; see 6.3.3 e).

7.4 Floor system

7.4.1 Types of floor systems

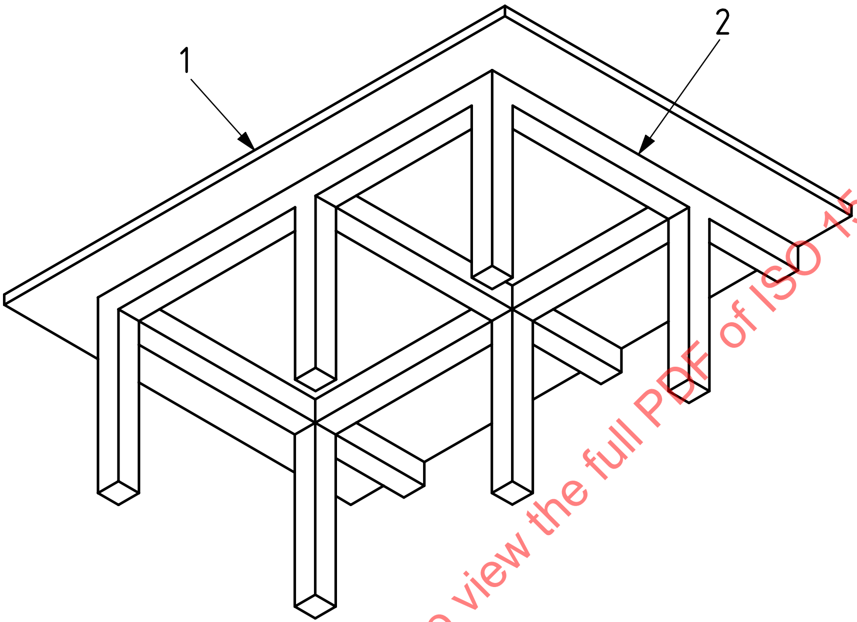
7.4.1.1 General

The floor system employed by a building designed in accordance with this International Standard should be one of the systems covered or their permitted variations. The selection of an appropriate floor system should be made after studying several alternatives.

7.4.1.2 Slab-on-girder system

7.4.1.2.1 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 axes, spanning the distance between the columns. A solid slab is supported by the girders. The slab can be cantilevered beyond the edge beam. In this system, the slab has a shallower depth than the girders; see Figure 45. This system should comply with the provisions for structural integrity specified in 7.4.3.



Key

1 slab

2 girder

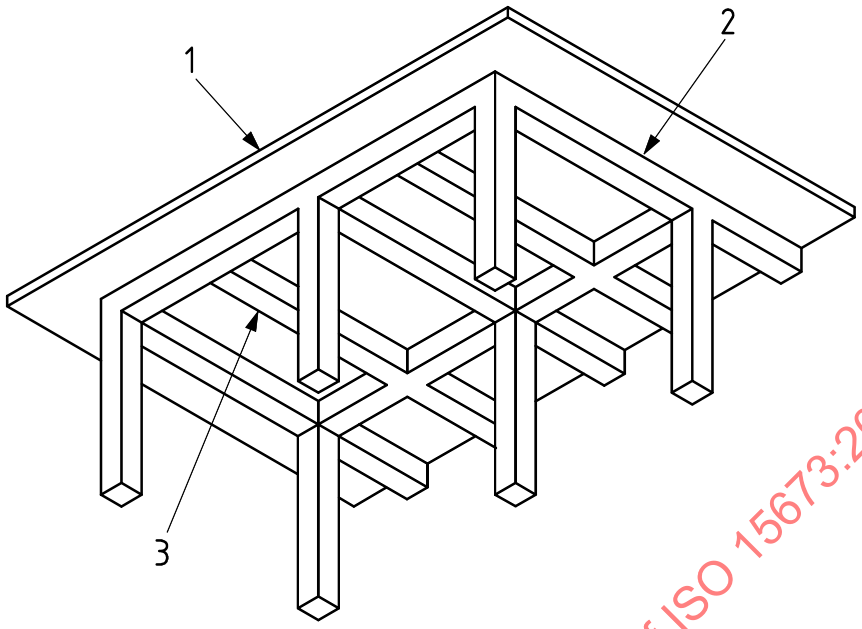
Figure 45 — Slab-on-girder floor system

7.4.1.2.2 Use of intermediate beams

One of the main variations of this 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 as the girders or shallower. These intermediate beams can be used in one direction, as shown in Figure 46, or in two directions, as shown in Figure 47. The use of too many intermediate beams will make the system gravitate to the joist system described in 7.4.1.3.

7.4.1.2.3 Advantages of slab-on-girder system

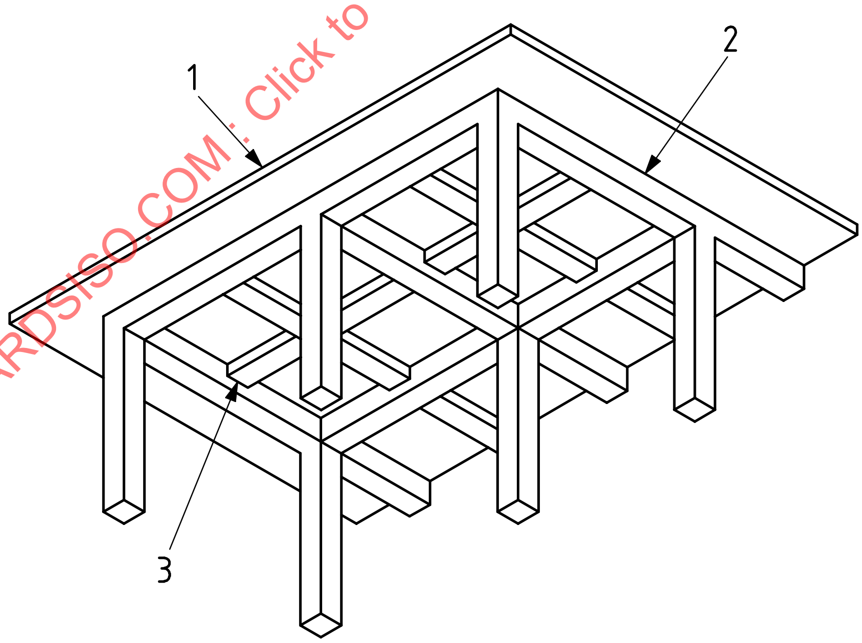
For the slab-on-girder system, each component has the appropriate minimum depth and width to comply with the strength or serviceability provisions therefore having a relatively low selfweight. The system can accommodate spans of any size, can easily be adapted to any plan shape, and large perforations,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



Key

1 slab
2 girder
3 intermediate one-directional beam

Figure 46 — Use of one-direction intermediate beams in the slab-on-girder floor system



Key

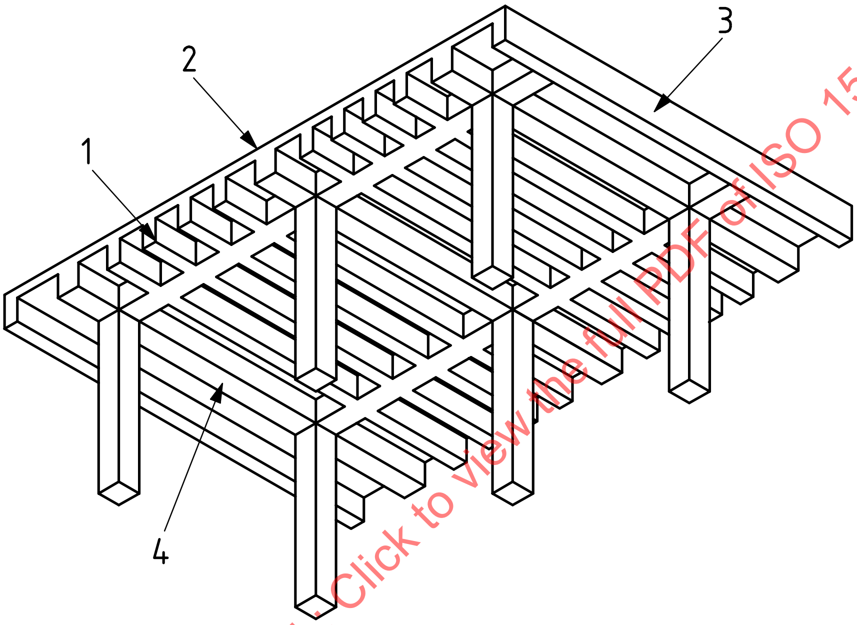
1 slab
2 girder
3 intermediate two-directional beam

Figure 47 — Use of two-direction intermediate beams in the slab-on-girder floor system

7.4.1.3 Joist systems

7.4.1.3.1 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 axes, spanning the distance between columns. A thin, solid slab spans the space between joists; see Figure 48. This system should comply with the provisions for structural integrity specified in 7.4.3. The thin slab cannot be cantilevered beyond the edge joist. In this system, the joists area is usually of the same depth as the girders, but it can have a shallower depth. The separation between parallel joists, measured centre-to-centre 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 50. The thin slab should comply with the minimum thickness provisions specified in 7.4.5.2.1.

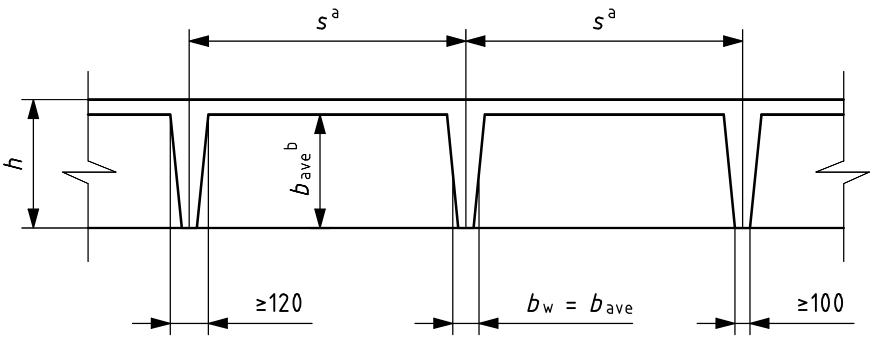


Key

1 joist
2 thin top slab
3 edge joist
4 girder or beam

Figure 48 — Joist floor system

Dimensions in millimetres



Key

a Restrictions: $s \leqslant 2{,}5\ h$; $s \leqslant 1{,}2$ m.

b $b_{ave} \leqslant 5$.

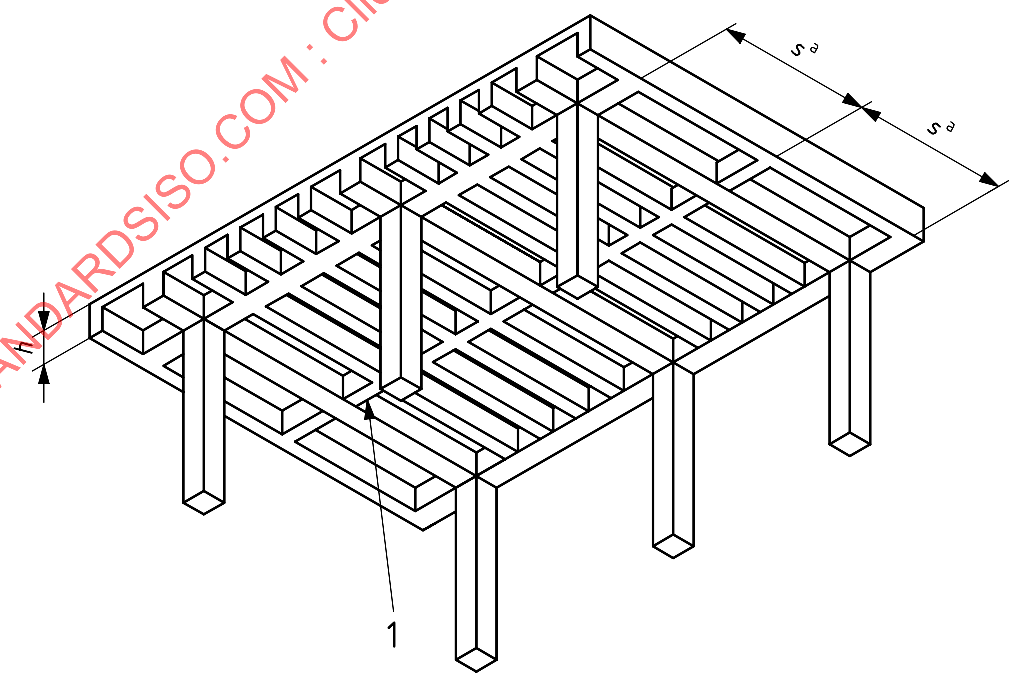
Figure 49 — Joist section dimensions

7.4.1.3.2 Type of formwork

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

7.4.1.3.3 Distribution ribs

In joist systems that span in only one direction, in order to avoid a concentrated load being carried by just one joist, transverse distribution ribs should be employed with separations of no more than 10 times the total depth, h, of the joist without exceeding 4 m; see Figure 50.



Key

1 distribution rib

a Restrictions: $s \leqslant 10\ h$; $s \leqslant 4$ m.

Figure 50 — Distribution ribs

7.4.1.3.4 Two-way joist systems

For approximately equal spans in both directions, it could be advantageous to use joists in both directions. In this case, in order for the system to be classified as a joist system, the joists must be supported on girders. This system is referred as a waffle-on-beams slab; see Figure 51. If the beams are omitted, the system should be classified as the waffle-slab system as described in 7.4.1.3.4.

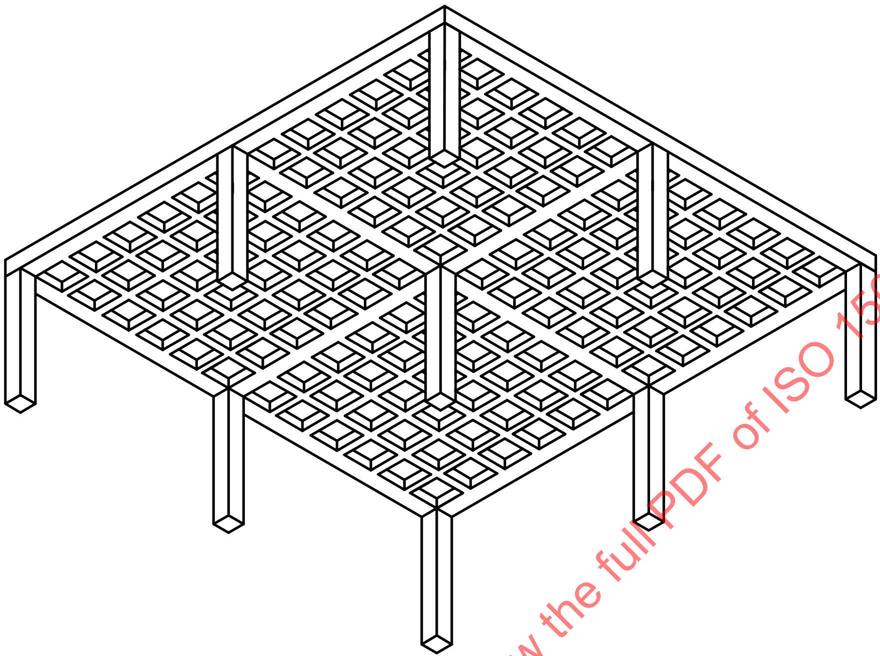


Figure 51 — Two-way joist system or waffle-on-beams system

7.4.1.3.5 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 provisions can easily be accommodated because of the relatively large depth of the system. The clear spacing between joists is a trade-off: the thinner the top slab, the larger the number of joists required. This allows the designer great freedom in the choice of appropriate dimensions.

7.4.2 Criteria for the selection of the floor system

The structural designer should select a floor system from the systems specified in this International Standard in 7.4.1. Several alternatives should be studied and the final selection should be made taking into account the merits of each of them in terms of the following:

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

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

c) presence of cantilevers and their maximum span and direction;

d) type of occupancy of the building;

e) available material strengths, both for concrete and reinforcing steel;

f) expected behaviour of the slab system and the adequacy of compliance with the serviceability and deflection criteria,

g) amount of materials, such as concrete, steel and formwork, required to build the floor system, taking into account that the floor system is probably responsible for the majority of the materials employed to build the structure;

h) local tradition in floor-system construction, which plays an important role in the selection and which, if followed, might simplify construction coordination;

i) workmanship, training and proficiency should affect the selection, by the avoidance of systems that require more training and proficiency than the local workers have;

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

7.4.3 Structural integrity

7.4.3.1 General

The following should constitute minimum provisions 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.

7.4.3.2 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 in accordance with 7.6.5.5. This reinforcement should always be lap-spliced using the minimum lap spliced length specified in 7.3.8.2.

7.4.3.3 Other beams and girders

All beams and girders, except the perimeter girders in accordance with 7.4.3.2, should have closed stirrups and a minimum area of continuous bottom longitudinal reinforcement, as specified in 7.6.4.5 and 7.6.5.5. This reinforcement should always be lap-spliced, at or close to the supports, using the minimum lap splice length specified in 7.3.8.2.

7.4.3.4 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 specified in 7.3.8.2, and discontinuous supports should be terminated with a standard hook; see 7.6.4.5.

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

7.4.4.1 General

The way the load is transferred 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 International Standard, the way the loads are carried to the support should be classified into one-way and two-way action.

7.4.4.2 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 or a structural concrete wall 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, beams or structural concrete walls that provide vertical support at all edges, and the long-slab span is greater than twice that of the short-slab span; or

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

7.4.4.3 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, has girders, beams or structural concrete walls along the full length of the edges that provide vertical support at all edges, and the long slab span is less than or equal to twice the short-slab span.

7.4.4.4 Floor 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 assign a 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 loads on the supporting elements should be verified and corrected as needed during the design and dimensioning stage of each of the structural elements.

7.4.5 Minimum allowable depth of the elements of the floor system

7.4.5.1 General

The following minimum allowable depth for elements of the floor 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.

7.4.5.2 Solid one-way slabs supported by girders, beams, joists or structural walls

7.4.5.2.1 Top thin, solid slab that spans the space between joists

The top thin slab should have a minimum thickness of $l/20$ 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.

7.4.5.2.2 Non-structural elements not likely to be damaged by large deflections

When a slab does not support partitions or other non-structural elements or when they are built of materials that are not likely to be damaged by large deflections, the minimum thickness, h, should not be less than the values given in Table 8, where the span length, l, should be taken as the centre-to-centre distance between supports, except when the span is less than 3 m, in which case it should be permitted to take l as the clear span.

Table 8 — Minimum thickness, h, for one-way solid slabs supporting non-structural elements that can accommodate large deflections

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	$\frac{l}{10}$

7.4.5.2.3 Non-structural elements likely be damaged by large deflections

When a slab supports either the top or the bottom edge of partitions or other non-structural elements that are likely to be damaged by large deflections, the minimum thickness, h, should not be less than the values specified in Table 9, where the span length, l, should be taken as the centre-to-centre distance between supports, except when the span is less than 3 m, in which case it should be permitted to take l as the clear span.

Table 9 — Minimum thickness, h, for one-way solid slabs supporting non-structural elements likely to be damaged by large deflections

Continuity across the supports	Minimum thickness h
Simply supported	$\frac{l}{14}$
One-end continuous	$\frac{l}{16}$
Both-ends continuous	$\frac{l}{19}$
Cantilever	$\frac{l}{7}$

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

7.4.5.3.1 Non-structural elements not likely to be damaged by large deflections

When a girder, beam or one-way joist does not support partitions or other non-structural elements or they are built of materials that are not likely to be damaged by large deflections, the minimum thickness, h, should not be less than the values specified in Table 10, where the span length, l, should be taken as the centre-to-centre distance between supports, except for joists when the span is less than 3 m, in which case it should be permitted to take l as the clear span.

Table 10 — Minimum thickness, h, for girders, beams and one-way joists supporting non-structural elements that can accommodate large deflections

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	$\frac{l}{8}$

7.4.5.3.2 Non-structural elements likely to be damaged by large deflections

When a girder, beam or one-way joist supports either top or bottom edge of partitions or other non-structural elements that are likely to be damaged by large deflections, the minimum thickness, h, should not be less than the values given in Table 11, where the span length, l, should be taken as the centre-to-centre distance between supports, except for joists when the span is less than 3 m, it should be permitted to take l as the clear span.

Table 11 — Minimum thickness, h, for girders, beams and one-way joists supporting non-structural elements likely to be damaged by large deflections

Continuity across the supports	Minimum thickness h
Simply supported	$\frac{l}{11}$
One-end continuous	$\frac{l}{12}$
Both-ends continuous	$\frac{l}{14}$
Cantilever	$\frac{l}{5}$

7.4.5.4 Two-way slabs supported by girders, beams or structural concrete walls

The minimum allowable depth of two-way slabs, including two-way joist and waffle-on-beams systems, supported by girders, beams or structural concrete walls at all edges of the panel, should be as specified in Equation (66), and for solid slabs should be not 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.

$$h = \frac{l_n}{30 + 3 \cdot \beta} \tag{66}$$

where

l_n is the clear span length in the long direction, measured face-to-face of the supporting beams;

β is the ratio of long clear span to short clear span of the slab panel.

The procedure for the design of two-way slabs-on-girders in accordance with this International Standard is that the supporting girders or beams should have a depth not less than three times the slab thickness; see 7.5.8.1.

7.4.6 Initial trial dimensions for the floor system

Initial trial dimensions should be defined for all the elements of the floor system. These initial trial dimensions should be assigned using the minimum depth or thickness, h, given in 7.4.5. 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; for joists, it should be defined using the minimum width dimensions given in 7.4.1.3.1.

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.

7.5 Solid slabs supported on girders, beams, joists or structural concrete walls

7.5.1 General

The design of one-way and two-way solid slabs supported by girders, beams or structural concrete walls at their edges should be performed in accordance with the provisions of 7.5. Provisions for the top thin solid slab that span between joists are also included.

7.5.2 Design-load definition

7.5.2.1 Loads to be included

The design load for solid slabs supported on girders, beams, joists or structural concrete walls should be established in accordance with the requirements of 7.2. The gravity loads that should be included in the design are the following:

a) dead loads: selfweight of the structural element, flat non-structural elements, standing non-structural elements and fixed equipment loads, if any;

b) live loads;

c) appropriate values of roof live load, rain load and snow load, if the slab is part of the roof system.

7.5.2.2 Dead load and live load

The values of q_d for dead load and q_l for live load should be expressed in newtons per square metre. The variable q_d should include the selfweight of the solid slab, at 24 N/m^2 per millimetre of thickness, and the weight of the flat and standing non-structural elements should be expressed in newtons per square metre. Live load should be determined in accordance with 7.2.6. If the slab is part of the roof system, the specified snow loads in 7.2.7 should be included, if appropriate.

7.5.2.3 Factored design load

The value of the factored design load, q_u, expressed in newtons per square metre, should be the greater of the values calculated by combining q_d and q_l in accordance with Equations (3) and (4). If the slab is part of a roof system, Equations (5) and (6) should also be investigated, choosing the greatest value from among the results of all four equations.

7.5.3 Details of reinforcement

7.5.3.1 General

For the purposes of this International Standard, the reinforcement of solid slabs-on-girders should be of the types described and should be in accordance with the provisions of 7.5.3.2 to 7.5.3.8.

7.5.3.2 Shrinkage and temperature reinforcement

7.5.3.2.1 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 only one direction.

7.5.3.2.2 Location

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

7.5.3.2.3 Minimum reinforcement area

Shrinkage and temperature reinforcement should be in accordance with the minimum reinforcement steel ratio, ρ_t, of 7.3.9.2.1.

7.5.3.2.4 Maximum and minimum reinforcement spacing

Shrinkage and temperature reinforcement should not be spaced further apart than specified by 7.3.7.7, nor should it be placed closer than specified by 7.3.7.2.

7.5.3.2.5 Reinforcement splicing

It should be permitted to lap-splice shrinkage and temperature reinforcement at any location. The splice length should be in accordance with 7.3.8.2.

7.5.3.2.6 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.

7.5.3.3 Positive flexural reinforcement

7.5.3.3.1 Description

Positive flexural reinforcement should be provided in the lower part of the slab section, as specified in 7.5 and should comply with the general provisions of 7.5.3.3 and the particular provisions for each slab type in accordance with 7.5.4 to 7.5.8.

7.5.3.3.2 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 to the bottom surface of the slab as permitted by the provisions (see 7.3.4.1) for concrete cover. 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.

7.5.3.3.3 Minimum reinforcement area

Positive flexural reinforcement should have an area at least equal to the area specified by 7.3.9.2.2.

7.5.3.3.4 Maximum reinforcement area

Positive flexural reinforcement area should not exceed the values set forth in 7.3.9.2.3.

7.5.3.3.5 Maximum and minimum reinforcement spacing

Positive flexural reinforcement should not be spaced further apart than specified by 7.3.7.6 nor should it be placed closer than permitted by 7.3.7.2.

7.5.3.3.6 Cut off points

It should be permitted to suspend, at the locations indicated in 7.5.6 to 7.5.8 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.

7.5.3.3.7 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.

7.5.3.3.8 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.

7.5.3.3.9 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.

7.5.3.4 Negative flexural reinforcement

7.5.3.4.1 Description

Negative flexural reinforcement should be provided in the upper part of the slab section, at edges and supports, in the amounts and lengths as specified in 7.5, and should be in accordance with the general provisions of 7.5.3.4 and the particular provisions for each slab type as specified in 7.5.4 to 7.5.8.

7.5.3.4.2 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 to the upper surface of the slab as permitted by the provisions (see 7.3.4.1) for concrete cover. 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.

7.5.3.4.3 Minimum reinforcement area

Negative flexural reinforcement should have an area at least equal to the area specified by 7.3.9.2.2.

7.5.3.4.4 Maximum reinforcement area

Negative flexural reinforcement area should not exceed the values specified in 7.3.9.2.3.

7.5.3.4.5 Maximum and minimum reinforcement spacing

Negative flexural reinforcement should not be spaced further apart than specified by 7.3.7.6, nor should it be placed closer than permitted by 7.3.7.2.

7.5.3.4.6 Cut off points

It should be permitted to suspend all the negative flexural reinforcement, except for cantilevers, at the locations indicated in 7.5.6 to 7.5.8 for each slab type. Where adjacent spans are unequal, negative flexural reinforcement cut-off points should be based on the provisions for the longer span.

7.5.3.4.7 Reinforcement splicing

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

7.5.3.4.8 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, in accordance with the anchorage distances specified in 7.3.8.3. 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.

7.5.3.5 Shear reinforcement

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

7.5.3.6 Corner reinforcement

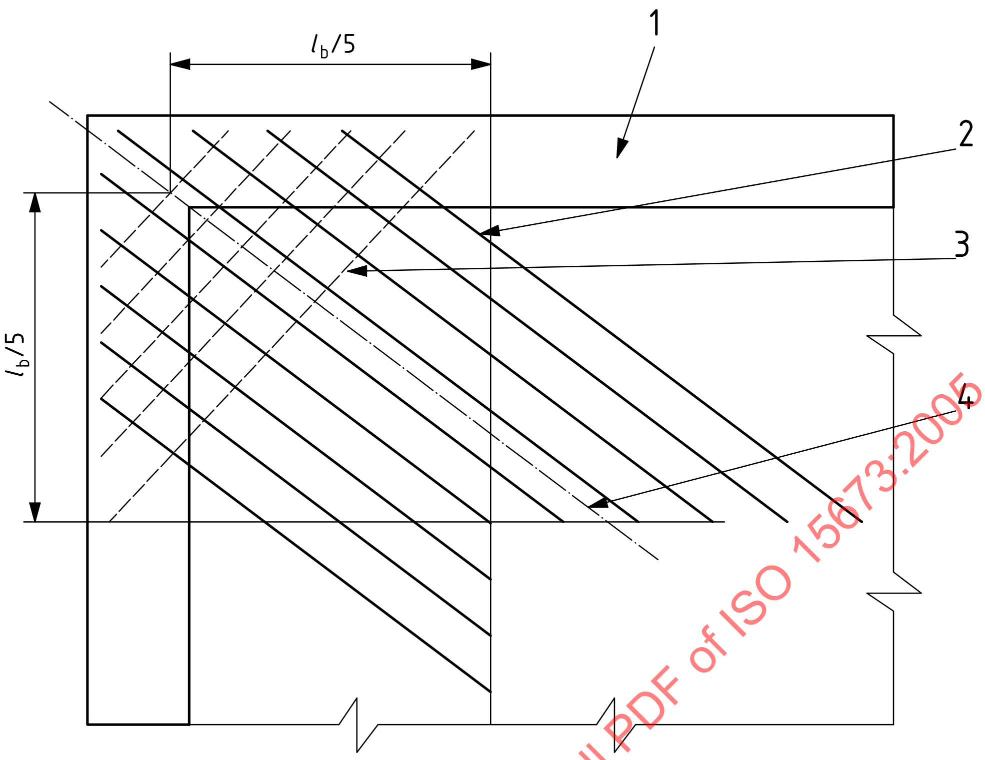
Special top and bottom slab reinforcement, in addition to other reinforcement, should be provided at exterior corners of the slab, different from cantilevers, for a distance equal to one-fifth of the longer clear span of the slab panel (see Figure 52). The amount of reinforcement, top and bottom, should 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 7.5.3.6.1 and 7.5.3.6.2.

7.5.3.6.1 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 or structural concrete walls.

7.5.3.6.2 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 or structural concrete walls.



Key

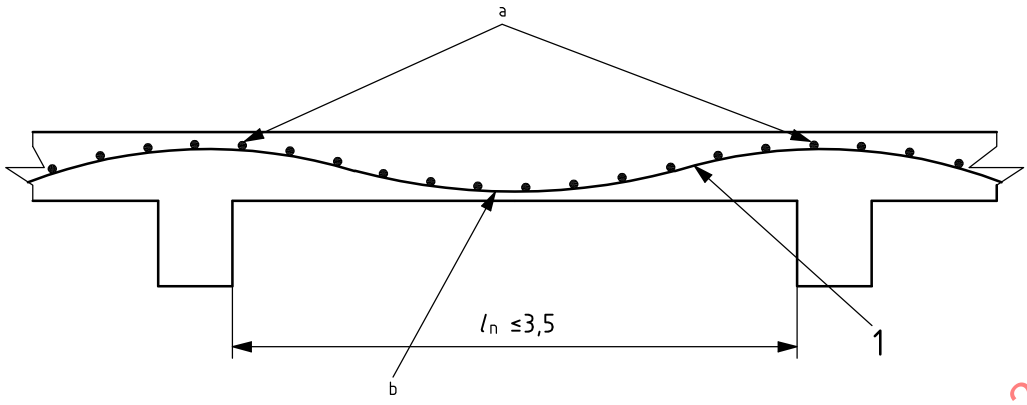
1 girder, beam or wall
2 corner top reinforcement
3 corner bottom reinforcement
4 slab panel diagonal

Figure 52 — Special slab corner reinforcement

7.5.3.7 Welded-wire fabric used in short span slabs

Welded-wire fabric used in slabs not exceeding 3,5 m in clear span should be permitted to simultaneously act as negative and positive flexural reinforcement by curving it from a point near the top of slab over the support to a point near the bottom of slab at mid-span, provided such reinforcement is either continuous over or anchored at the supports. The area of longitudinal reinforcement should comply with the maximum required factored moment, either positive or the negative; see Figure 53. In two-way slabs, the wires of the welded-wire fabric in the perpendicular direction should comply with the required area of flexural reinforcement in that direction, and in one-way slabs with the required shrinkage and temperature reinforcement.

Dimension in metres



Key

1 welded-wire fabric as positive and negative reinforcement

a Placed close to the top of the slab over supports.

b Placed close to the bottom of the slab near mid-span.

Figure 53 — Welded-wire fabric in short spans

7.5.3.8 Practical considerations for the value of d_c and d as applied in solid slabs

A determination of the distance, [d_c], from the extreme tension fibre to the centroid of tension reinforcement should include the appropriate concrete cover as specified in 7.3.4, the bar diameters employed and the existence of reinforcement in a perpendicular direction placed between the reinforcement under study and the concrete surface. It should be permitted to use the values of d_c specified in this subclause to compute d in accordance with the relationship $d = h - d_c$. 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.

7.5.4 Top thin solid slab that spans between joists

7.5.4.1 Dimensions

A thin solid slab that spans between joists should comply with the minimum thickness as specified in 7.4.5.2.1. The top thin slab should not be permitted to be cantilevered beyond the edge joist; see 7.4.1.3.1.

7.5.4.2 Factored flexural moment

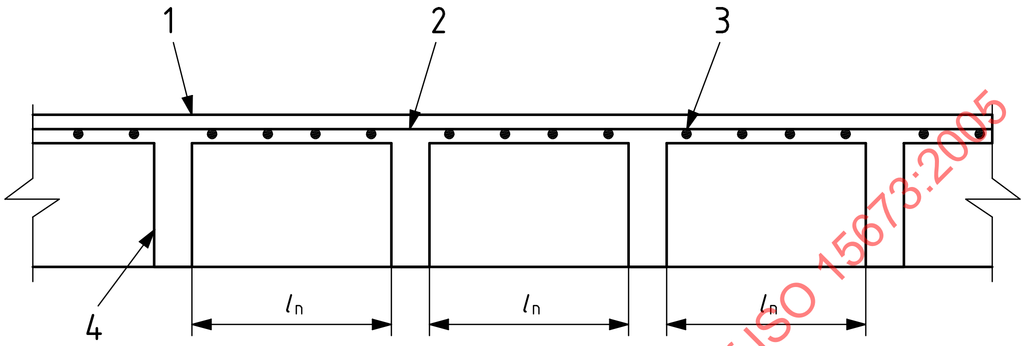
The factored flexural moment, M_u, expressed in newton·metres per metre, for negative and positive flexural moment in a thin slab that spans between joists in joist construction, should be calculated in accordance with Equation (67)

$$M_u^+ = M_u^- = \frac{q_u \cdot l_n^2}{12} \tag{67}$$

where l_n is the clear spacing, expressed in metres, between joists and q_u is expressed in newtons per square metre; see Figure 54.

7.5.4.3 Reinforcement

The flexural reinforcement ratio, ρ, perpendicular to the joist direction, should be calculated in accordance with Equation (27) or (29), with the value of M_u derived from Equation (67), converted to newton·millimetres (1 · m/m = 10^3 N · mm/m), defining d, expressed in millimetres, as one-half of the thickness of the thin slab, and $b = 1\,000$ mm. The value of ρ should be defined as equal to or greater than the shrinkage and temperature ratio specified in 7.3.9.2.1; see Figure 54. The flexural reinforcing bar separations should meet the provisions of 7.3.7.6. The reinforcement parallel to the joist direction should meet the provisions of 7.5.3.2.



Key

1 top thin slab

2 negative and positive flexural reinforcement

3 shrinkage and temperature reinforcement

4 joist

Figure 54 — Reinforcement of the thin solid slab that spans between joists

7.5.4.4 Shear strength verification

The factored shear, V_u, expressed in newtons per metre, for the thin slab that spans between joists in joist construction should be calculated in accordance with Equation (68):

$$V_u = \frac{q_u \cdot l_n}{2} \tag{68}$$

where l_n is the clear spacing, expressed in metres, between joists and q_u is expressed in newtons per square metre; see Figure 54. The design shear strength, $\phi \cdot V_n$, expressed in newtons per metre, should be calculated in accordance with Equation (52), defining d, expressed in millimetres, as one-half the thickness of the thin slab and $b_w = b = 1\,000$ mm. This should be in accordance with Equation (50).

7.5.4.5 Calculation of the reactions on the joists

The factored uniformly distributed reaction, r_u, expressed in newtons per metre, on the supporting joists should be calculated in accordance with Equation (69):

$$r_u = \frac{2 \cdot V_u \cdot l}{l_n} \tag{69}$$

where

V_u is the factored shear, expressed in newtons per metre, in accordance with 7.5.4.4,

l is the centre-to-centre spacing, expressed in metres, of the joist;

l_n is the clear spacing, expressed in metres, between joists.

7.5.5 Cantilevers of slabs supported on girders, beams or walls

7.5.5.1 Dimensions

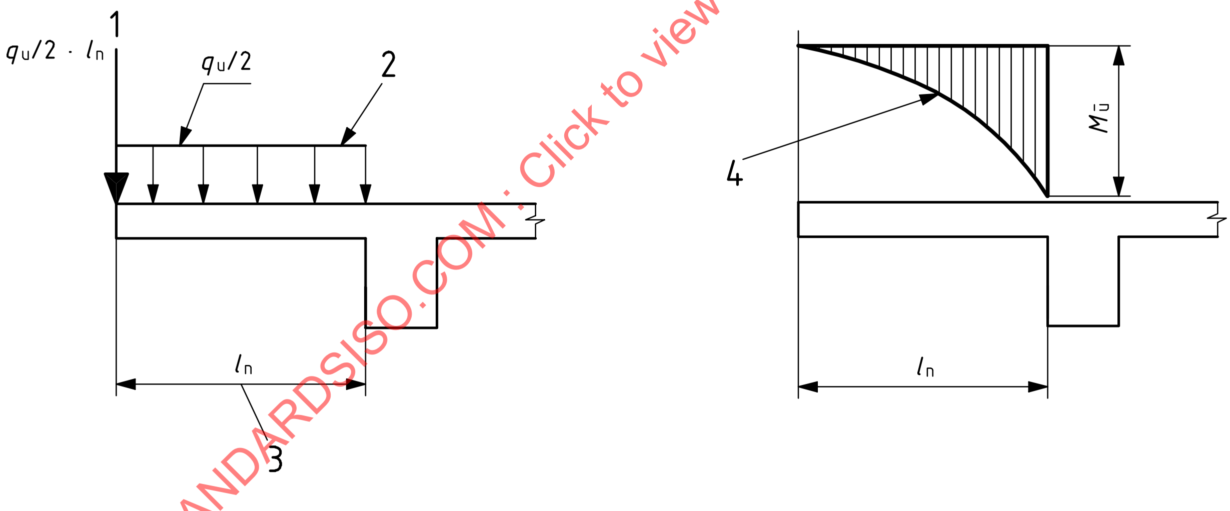
Solid slab cantilevers, spanning beyond the edge girder, beam or structural concrete wall, should be in accordance with the minimum thickness provisions specified in 7.4.5.2. The cantilever span should not exceed the limits specified in 6.1. No openings for ducts or shafts should be permitted in the internal one-half span of the cantilever. It should be permitted for the slab to be cantilevered in two directions at corners, with the same limitations for single cantilevers. The thin top slab that spans between joists should not be cantilevered beyond the edge joist.

7.5.5.2 Factored negative flexural moment

The factored negative flexural moment, M_u, for slab cantilevers that span beyond the edge-supporting girders, beams or structural concrete walls should be calculated in accordance with Equation (70) on the assumption that half of the distributed factored load, q_u, acts as a concentrated load at the tip of the cantilever and the other half acts as a uniformly distributed load over the full span. However, 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 55.

$$M_u^- = \frac{3 \cdot q_u \cdot l_n^2}{4} \tag{70}$$

where l_n is the clear span, expressed in metres, of the cantilever, q_u is expressed in newtons per square metre, and M_u^- is expressed in newton·metres per metre.



Key

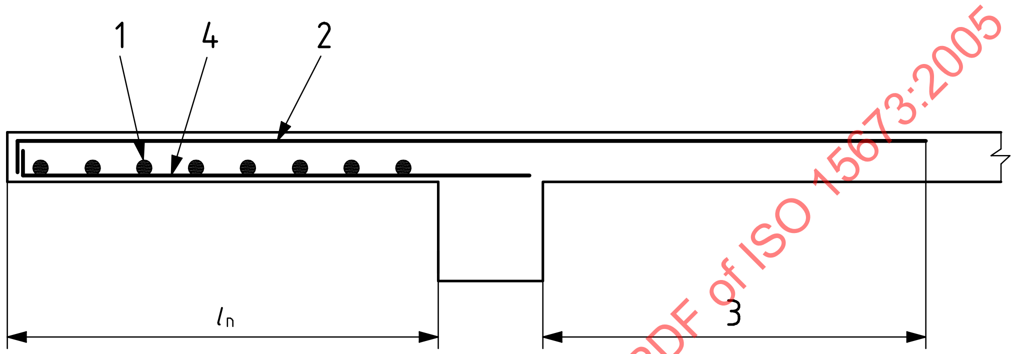
1 concentrated load
2 uniform load
3 cantilever clear span
4 moment diagram

Figure 55 — Calculation of the negative moment for slab cantilevers

7.5.5.3 Reinforcement

7.5.5.3.1 Negative flexural reinforcement

The negative flexural reinforcement ratio, ρ, in the direction of the cantilever should be calculated in accordance with Equation (27) or (29), using the value of M_u derived from Equation (70), converted to newton millimetres per metre (1 · m/m = 10^3 N · mm/m), with the appropriate value of d, expressed in millimetres, and $b = 1\,000$ mm. The negative flexural reinforcement should be in accordance with 7.5.3.4. This reinforcement should be anchored in the first interior span not less than l_d for the reinforcing bar (see 7.3.8.1), nor the distance required for the negative reinforcement of the interior slab panel at the edge support; see Figure 56.



Key

1 shrinkage and temperature reinforcement

2 negative cantilever reinforcement

3 distance required for the negative reinforcement of the first interior span, but not less than l_n for the bar

4 minimum positive reinforcement

Figure 56 — Reinforcement for slab cantilevers

7.5.5.3.2 Positive flexural reinforcement

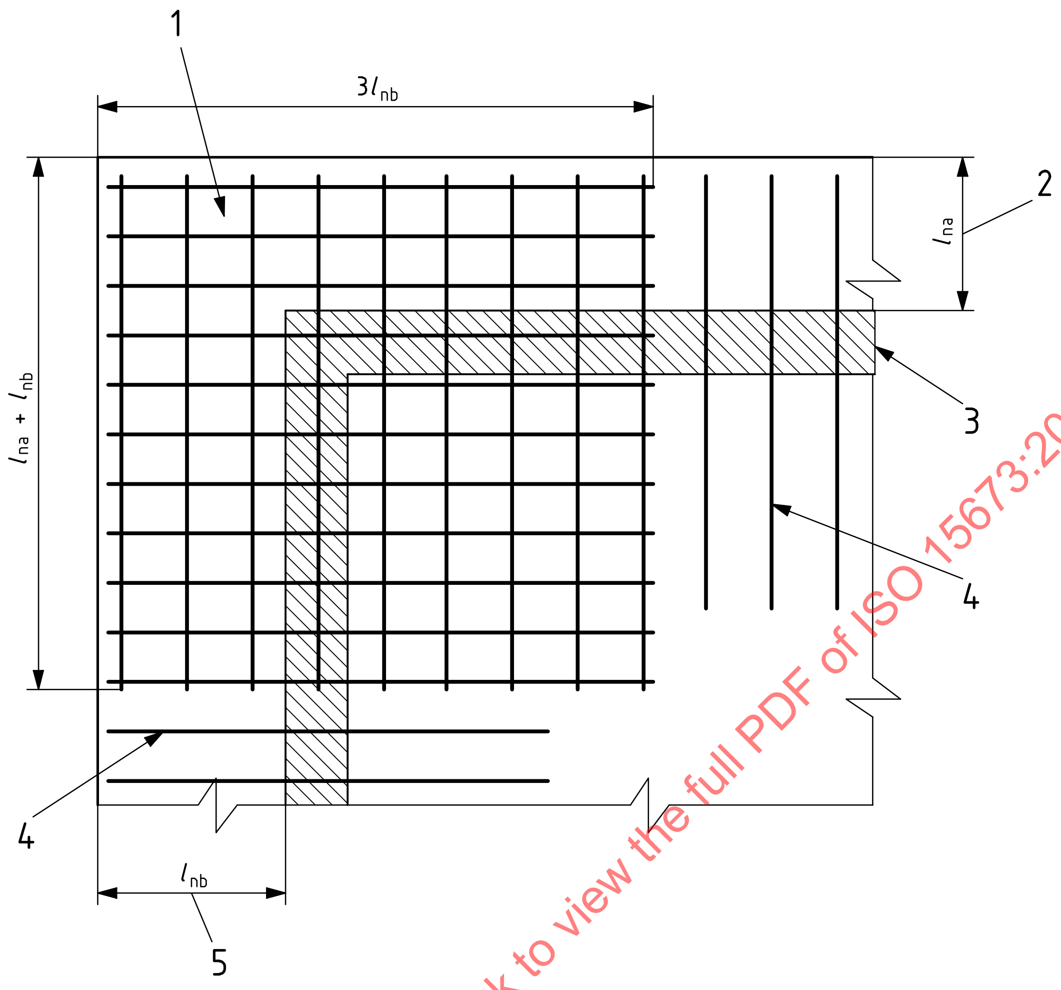
A minimum amount of positive flexural reinforcement with an area greater or equal to the shrinkage and temperature reinforcement, in accordance with the provisions of 7.5.3.2, should be provided in the direction of the cantilever; see Figure 56.

7.5.5.3.3 Shrinkage and temperature reinforcement

Reinforcement parallel to the edge of the cantilever, in accordance with the provisions of 7.5.3.1, should be provided; see Figure 56.

7.5.5.3.4 Reinforcement of two-way cantilevers

At corners where the slab is cantilevered in two directions, the negative flexural reinforcement should be calculated for the larger span cantilever, in accordance with the provisions of 7.5.5.3.1. This reinforcement should be placed in both directions (see Figure 57) 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 in accordance with the provisions of 7.5.5.3.2 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

Figure 57 — Negative flexural reinforcement in two-way slab cantilevers

7.5.5.4 Shear verification

The factored shear, V_u, expressed in newtons per metre, for the support of cantilever slabs should be calculated in accordance with Equation (71):

$$V_u = q_u \cdot l_n \quad (71)$$

where l_n should be the clear span, expressed in metres, of the cantilever and q_u should be expressed in newtons per square metre.

For two-way cantilevers, the value of V_u should be taken as twice the value derived from Equation (71) using the larger cantilever span.

The design shear strength $\phi \cdot V_n$, expressed in newtons per metre, should be calculated in accordance with Equation (52) with the appropriate value of d, expressed in millimetres and $b = 1\,000$ mm. This should be in accordance with Equation (50).

7.5.5.5 Calculation of the reactions on the supports

The uniformly distributed factored reaction, r_u, on the support of the cantilever, expressed in newtons per metre, should be the value derived from Equation (72):

$$r_u = \frac{V_u \cdot l}{l_n} \tag{72}$$

where

V_u is the factored shear, expressed in newtons per metre, as specified in 7.5.5.4;

l is the span, expressed in metres, of the cantilever measured from the centreline of the supporting element;

l_n is the clear span, expressed in metres, of the cantilever.

Where two-way cantilevers exist, it should be permitted to calculate of the value of R_u in accordance with Equation (72), employing the value of V_u for the larger cantilever span derived from Equation (71) without doubling it.

7.5.6 One-way one-span solid slabs spanning between girders, beams or structural concrete walls

7.5.6.1 Dimensions

One-way one-span solid slabs should comply with the minimum thickness provisions as specified in 7.4.5.2. In addition to the appropriate provisions specified in 7.5.6, these slabs should comply with the general dimensional provisions as specified in 6.1 and the particular provisions specified in 7.4.1.2 for slab-on-girder systems.

7.5.6.2 Factored flexural moment

The factored positive and negative flexural moment, M_u, expressed in newton·metres per metre, for one-span one-way slabs should be calculated in accordance with Equations (73) and (74) as specified in Table 12.

Table 12 — Factored flexural moment for one-way, one-span slabs

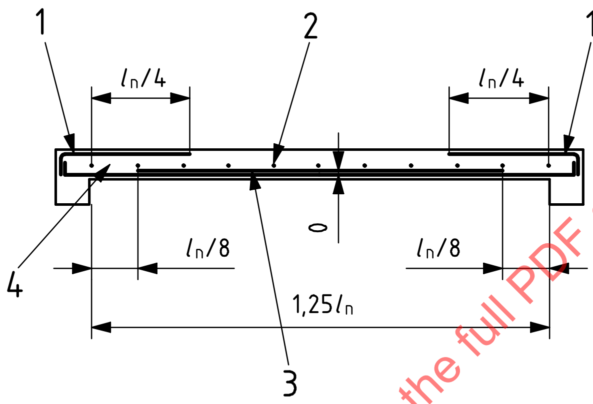
Positive moment: $M_u^+ = \frac{q_u \cdot l_n^2}{8}$	(73)
Negative moment at supports: $M_u^- = \frac{q_u \cdot l_n^2}{24}$	(74)

7.5.6.3 Longitudinal flexural reinforcement

7.5.6.3.1 Positive flexural reinforcement

The positive reinforcement ratio, ρ, in the direction of the span, l_n, should be determined in accordance with Equation (27) or (29), with the value of M_u^+ derived from Equation (73) converted to newton·millilmetres (1 N·m/m = 10^3 N · mm/m), using d expressed in millimetres, and b = 1 000 mm. This reinforcement should be in accordance with the provisions specified in 7.5.3.3. In those cases in which the slab is cast monolithically with a supporting girder, beam or structural concrete wall 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_n/8, measured from the internal face of the supports into the span; see Figure 58.

Dimension in millimetres



Key

1 negative flexural reinforcement

2 shrinkage and temperature reinforcement

3 positive flexural reinforcement

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

Figure 58 — Reinforcement for one-span one-way slabs

7.5.6.3.2 Negative flexural reinforcement

The negative flexural reinforcement ratio, ρ, in the direction of the span, l_n, should be determined in accordance with Equation (27) or (29), with the value of M_u^- derived from Equation (74) converted to newton·millimetres (1 N·m/m = 10^3 N · mm/m), using d expressed in millimetres, and b = 1 000 mm. This reinforcement should be in accordance with the provisions specified in 7.5.3.4. At a distance equal to l_n/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 58.

7.5.6.3.3 Shrinkage and temperature reinforcement

The reinforcement perpendicular to the span should meet the provisions for shrinkage and temperature reinforcement specified in 7.5.3.2; see Figure 58.

7.5.6.4 Shear verification

The factored shear, V_u, expressed in newtons per metre, for the one-span one-way slab should be calculated at the face of the supports in accordance with Equation (75):

$$V_u = \frac{q_u \cdot l_n}{2} \tag{75}$$

where l_n is the clear span, expressed in metres and q_u is expressed in newtons per square metre; see Figure 58.

The design shear strength, $\phi \cdot V_n$, in N/m, should be calculated in accordance with Equation (52), with d expressed in millimetres and $b_w = b =$ 1 000 mm. This should be in accordance with Equation (50).

7.5.6.5 Calculation of the reactions on the supports

Uniformly distributed factored reaction, r_u, on the supports of one-way one-span slabs expressed in newtons per metre, should be the value derived from Equation (76) plus the uniformly distributed reaction from any cantilever spanning from that support:

$$r_u = \frac{V_u \cdot l}{l_n} \tag{76}$$

where

V_u is the factored shear, expressed in newtons per metre, as specified in 7.5.6.4;

l is the centre-to-centre span, expressed in metres, of the slab;

l_n is the clear span, expressed in metres, of the slab.

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

7.5.7.1 Dimensions

One-way solid slabs with two or more spans should comply with the minimum thickness provisions as specified in 7.4.5.2. In addition to the appropriate provisions specified in 7.5, slabs should comply with the general dimensions as specified in 6.1 and the particular provisions specified in 7.4.1.2 for slab-on-girder systems.

The following restrictions should be in effect for slabs designed in accordance with 7.5.7.

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 %; see 6.1.

c) The loads are uniformly distributed.

d) The unit live load, q_l, does not exceed three times the unit dead load, q_d.

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

7.5.7.2 Factored flexural moment

The factored positive and negative flexural moment, M_u, in N·m/m, for one-way slabs should be calculated using Equations (77) to (82) and those given in Table 13 for slabs with two or more spans.

For positive moment, use Equations (77) and (78):

a) at end spans, in accordance with Equation (77):

$$M_u^+ = \frac{q_u \cdot l_n^2}{11} \tag{77}$$

b) at interior spans, in accordance with Equation (78):

$$M_u^+ = \frac{q_u \cdot l_n^2}{16} \tag{78}$$

For negative moment at supports, use Equations (79) to (82).

a) at the interior face of an external support, in accordance with Equation (79):

$$M_u^- = \frac{q_u \cdot l_n^2}{24} \tag{79}$$

b) at the exterior face of the first internal support for only two spans, in accordance with Equation (80):

$$M_u^- = \frac{q_u \cdot l_n^2}{9} \tag{80}$$

c) at the faces of internal supports for more than two spans, in accordance with Equation (81):

$$M_u^- = \frac{q_u \cdot l_n^2}{10} \tag{81}$$

d) at the faces of all supports for slabs with spans not exceeding 3 m, in accordance with Equation (82):

$$M_u^- = \frac{q_u \cdot l_n^2}{12} \tag{82}$$

7.5.7.3 Longitudinal flexural reinforcement

7.5.7.3.1 Positive flexural reinforcement

The positive reinforcement ratio, ρ, in the direction of the span, l_n, should be determined in accordance with Equation (27) or (29), with the appropriate value of M_u^+ derived from Equations (77) or (78), converted to newton·millimetres per metre (1 N·m/m = 10^3 N·mm/m), using d, expressed in millimetres, and b = 1 000 mm. This reinforcement should be in accordance with the provisions specified in 7.5.3.3. At internal supports, at a distance equal to $l_n/8$ 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 Figures 59 and 60.

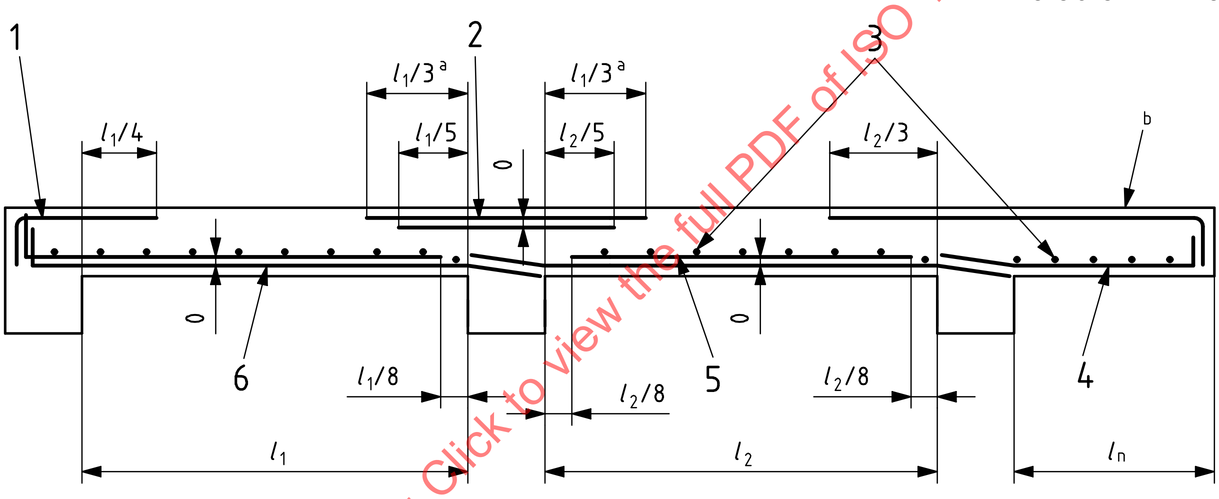
7.5.7.3.2 Negative flexural reinforcement

The negative flexural reinforcement ratio, ρ, in the direction of the span, l_n, should be determined in accordance with Equation (27) or (29), with the appropriate value of M_u^- derived from Equations (79) to (82), converted to newton·millimetres per metre (1 N·m/m = 10^3 N·mm/m), using d, expressed in millimetres, and b = 1 000 mm. This reinforcement should be in accordance with the provisions specified in 7.5.3.4. At internal supports, at a distance equal to $l_n/3$ where l_n should correspond to the largest of the neighbouring 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 $l_n/4$ measured from the internal face of the support into the span, all the negative flexural reinforcement should be permitted to be suspended; see Figures 59 and 60.

7.5.7.3.3 Shrinkage and temperature reinforcement

The reinforcement perpendicular to the span should meet the provisions for shrinkage and temperature reinforcement specified in 7.5.3.2; see Figures 59 and 60.

Dimensions in millimetres



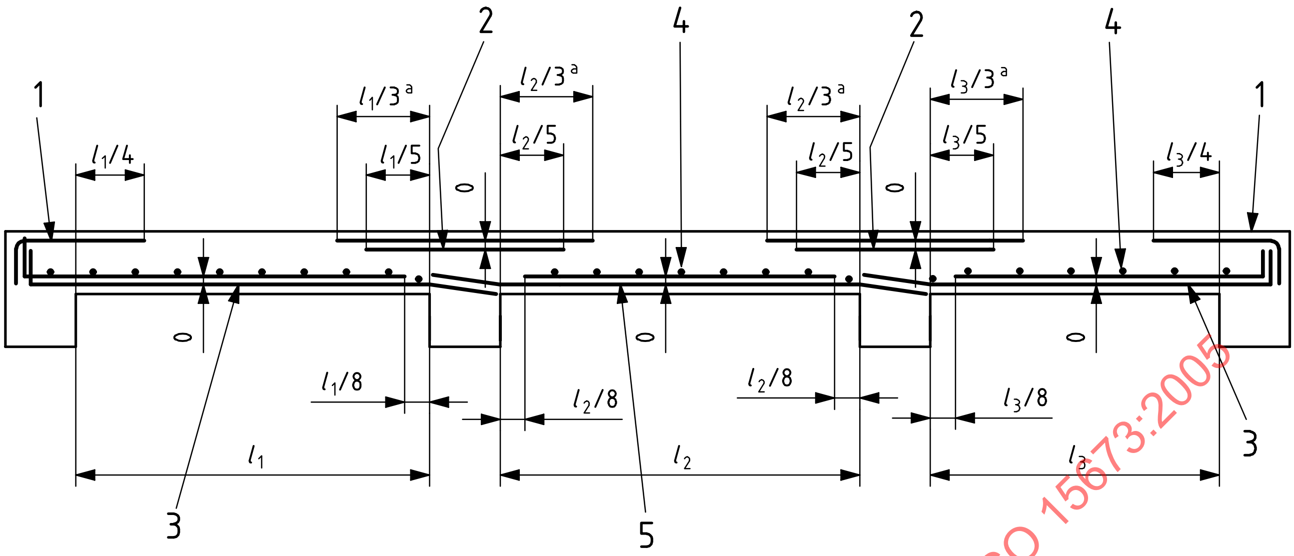
Key

1 negative reinforcement at the interior face of an external support
2 negative reinforcement at the interior support for two spans
3 shrinkage and temperature reinforcement
4 minimum cantilever positive reinforcement
5 positive reinforcement interior span
6 positive reinforcement end span

a Negative reinforcement cut-off points should be based upon the greater of the two neighbouring spans.
b Greater negative reinforcement from that required for the external support or for the cantilever.

Figure 59 — Reinforcement for two-span one-way slabs supported by girders, beams or structural concrete walls

Dimensions in millimetres



Key

1 negative reinforcement at the interior face of an external support

2 negative reinforcement at the faces of interior support for more than two spans

3 positive reinforcement end span

4 shrinkage and temperature reinforcement

5 positive reinforcement end span

[a] Negative reinforcement cut-off points should be based upon the greater of the two neighbouring spans.

Figure 60 — Reinforcement for one-way slabs supported by girders, beams or structural concrete walls, with three or more spans

7.5.7.4 Shear verification

The factored shear, V_u, in newtons per metre, for the slab should be calculated at the faces of all supports in accordance with Equation (83) at the exterior face of the first interior support and Equation (84) at faces of all other supports:

$$V_u = 1{,}15 \cdot \frac{q_u \cdot l_n}{2} \tag{83}$$

where l_n is the clear span, expressed in metres, and q_u is expressed in newtons per square metre; see Figures 59 and 60.

$$V_u = \frac{q_u \cdot l_n}{2} \tag{84}$$

The design shear strength, $\phi \cdot V_n$, expressed in newtons per metre, should be calculated in accordance with Equation (52), with d expressed in millimetres and $b_w = b = 1\,000$ mm. There should be compliance with Equation (50) at all faces of the supports.

7.5.7.5 Calculation of the reactions on the supports

Uniformly distributed factored reaction, r_u, expressed in newtons per metre, on the support contributed by any span of one-way slabs should be derived in accordance with Equation (85)

$$r_u = \frac{V_u \cdot l}{l_n} \tag{85}$$

where

V_u is the factored shear, expressed in newtons per metre, from 7.5.7.4;

l is the centre-to-centre span, expressed in metres;

l_n is the clear span, expressed in metres.

Total factored uniformly distributed reaction on the external supports should be equal to the value of the factored uniformly distributed reaction, r_u, from the span derived in accordance with Equation (85) 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, derived in accordance with Equation (85) for both neighbouring spans at that support.

7.5.8 Two-way solid slabs spanning between girders, beams or structural concrete walls

7.5.8.1 Dimensions

Two-way solid slabs having girders, beams or structural concrete walls at all edges should be in accordance with the minimum thickness as specified in 7.4.5.4. In addition to the appropriate provisions of 7.5, two-way slabs should be in accordance with the general dimensional provisions specified in 6.1 and the particular provisions of 7.4.1.2 for slab-on-girder systems.

The following restrictions should be in effect for the use of the procedure specified in 7.5.8.

a) There are two or more spans.

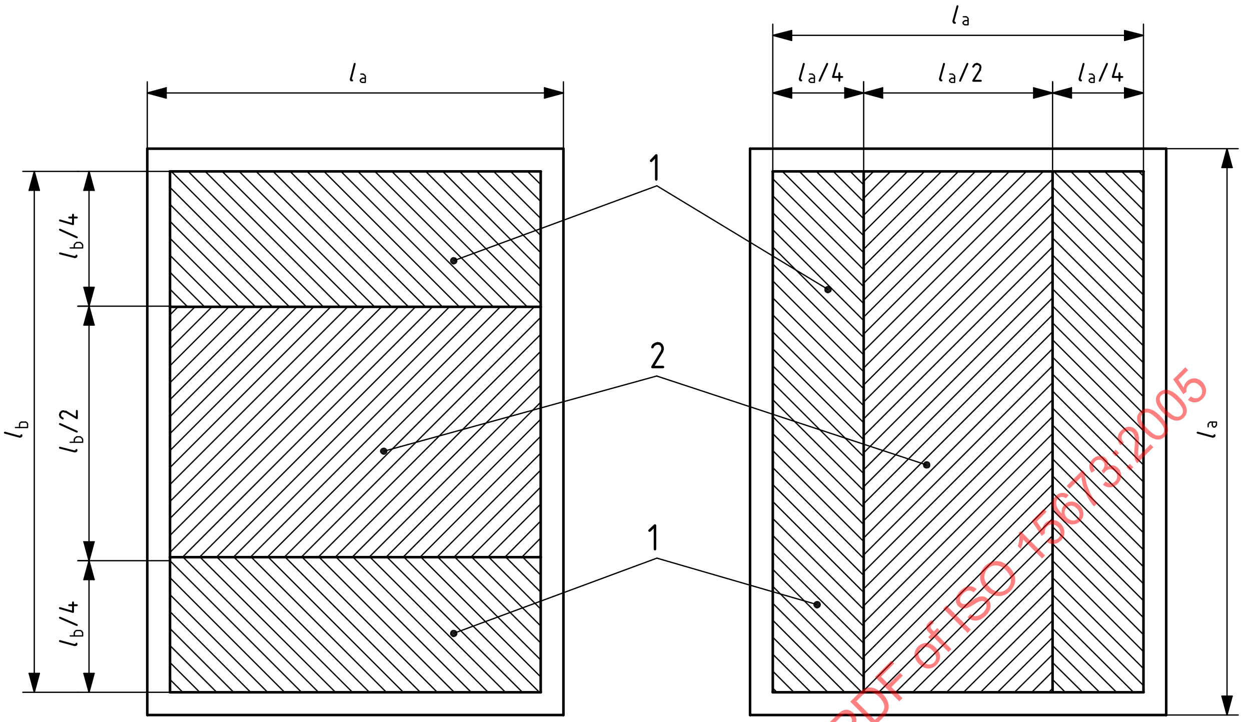
b) The spans are approximately equal, with the larger of two adjacent spans not greater than the shorter by more than 20 % of the larger; see 6.1.

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.

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 61.



Key

1 border region

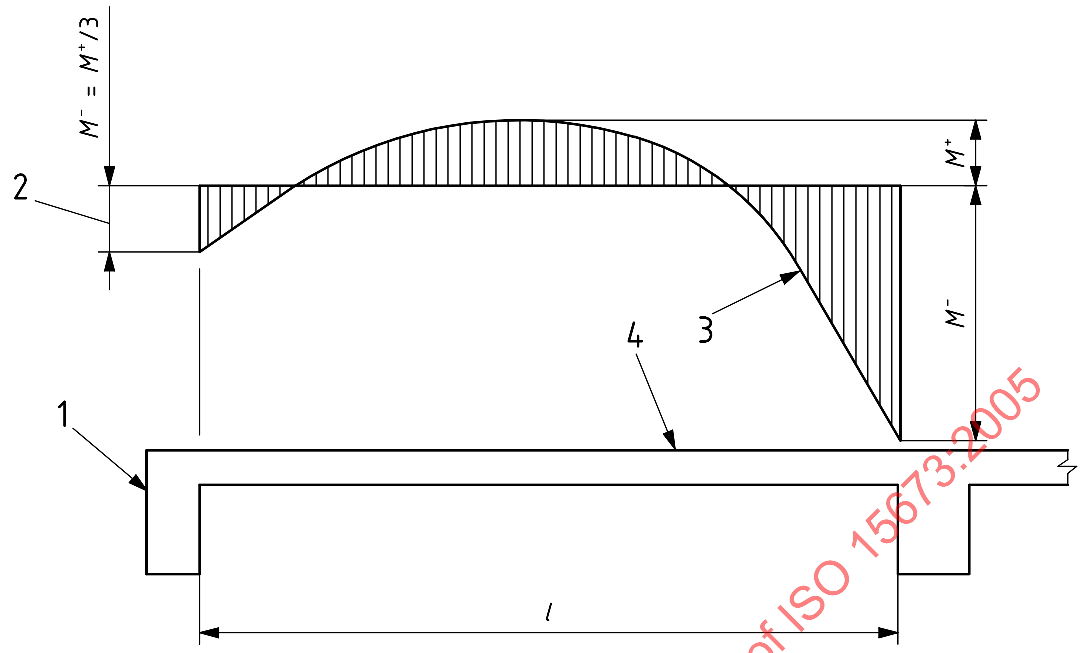
2 central region

Figure 61 — Central and border regions for two-way slabs supported on girders, beams or structural concrete walls

7.5.8.2 Factored flexural moment

The factored positive and negative moment, M_u, for two-way solid slabs should be calculated using the procedure specified in 7.5.8.2. The negative and positive factored flexural moment for the central region of the panel, in each direction, should be calculated in accordance with the Equations specified in Table 13 for central panels, in Table 14 for edge panels with the short span at the edge, in Table 15 for edge panels with the long span at the edge and in Table 16 for corner panels. In each table, the values of the factored flexural moments should be derived 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 62.

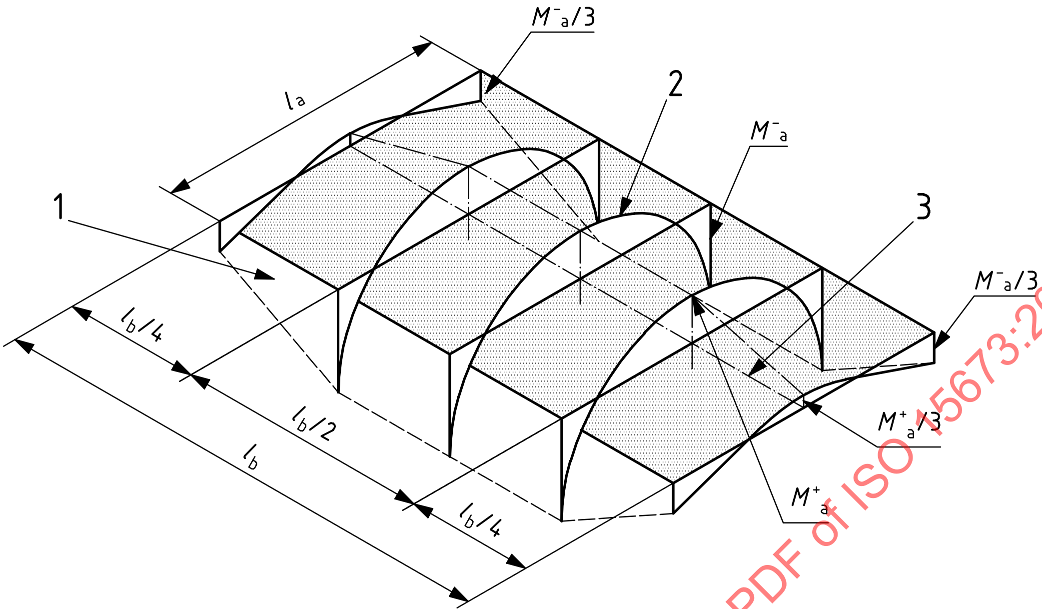


Key

1 discontinuous edge
2 value of the negative moment at the discontinuous edge
3 moment diagram
4 end slab span

Figure 62 — 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 63 for moments in the short directions and in Figure 64 for moments in the long direction.



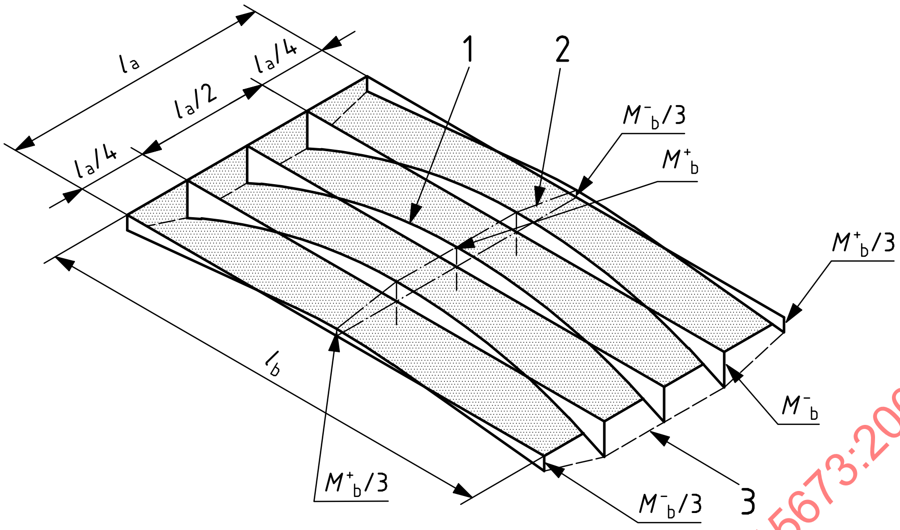
Key

1 variation of M_a^- along l_b

2 variation of M_a along l_a

3 variation of M_a^+ along l_b

Figure 63 — Variation of moment, M_a, across the width of critical design sections, for two-way slabs supported on girders, beams or structural concrete walls



Key

1 variation of M_b along l_b

2 variation of M_b^+ along l_a

3 variation of M_b^- along l_a

Figure 64 — Variation of moment, M_b, across the width of critical design sections, for two-way slabs supported on girders, beams or structural concrete walls

Table 13 — Central panel of two-way slabs supported on girders, beams or structural concrete walls

Panel span ratio $\beta = l_b/l_a$	Short direction l_a			Long direction l_b		
	Negative moment	Positive moment	Load fraction	Negative moment	Positive moment	Load fraction
1,0	$M_a^- = \frac{q_u \cdot l_a^2}{22}$	$M_a^+ = \frac{q_u \cdot l_a^2}{42}$	$\alpha_a = 0{,}50$	$M_b^- = \frac{q_u \cdot l_b^2}{22}$	$M_b^+ = \frac{q_u \cdot l_b^2}{42}$	$\alpha_b = 0{,}50$
1,2	$M_a^- = \frac{q_u \cdot l_a^2}{16}$	$M_a^+ = \frac{q_u \cdot l_a^2}{30}$	$\alpha_a = 0{,}67$	$M_b^- = \frac{q_u \cdot l_b^2}{35}$	$M_b^+ = \frac{q_u \cdot l_b^2}{60}$	$\alpha_b = 0{,}33$
1,4	$M_a^- = \frac{q_u \cdot l_a^2}{14}$	$M_a^+ = \frac{q_u \cdot l_a^2}{25}$	$\alpha_a = 0{,}80$	$M_b^- = \frac{q_u \cdot l_b^2}{50}$	$M_b^+ = \frac{q_u \cdot l_b^2}{100}$	$\alpha_b = 0{,}20$
1,6	$M_a^- = \frac{q_u \cdot l_a^2}{13}$	$M_a^+ = \frac{q_u \cdot l_a^2}{22}$	$\alpha_a = 0{,}87$	$M_b^- = \frac{q_u \cdot l_b^2}{85}$	$M_b^+ = \frac{q_u \cdot l_b^2}{145}$	$\alpha_b = 0{,}13$
1,8	$M_a^- = \frac{q_u \cdot l_a^2}{12}$	$M_a^+ = \frac{q_u \cdot l_a^2}{20}$	$\alpha_a = 0{,}92$	$M_b^- = \frac{q_u \cdot l_b^2}{135}$	$M_b^+ = \frac{q_u \cdot l_b^2}{225}$	$\alpha_b = 0{,}08$
2,0	$M_a^- = \frac{q_u \cdot l_a^2}{11}$	$M_a^+ = \frac{q_u \cdot l_a^2}{18}$	$\alpha_a = 0{,}94$	$M_b^- = \frac{q_u \cdot l_b^2}{170}$	$M_b^+ = \frac{q_u \cdot l_b^2}{340}$	$\alpha_b = 0{,}06$
> 2,0	$M_a^- = \frac{q_u \cdot l_a^2}{10}$	$M_a^+ = \frac{q_u \cdot l_a^2}{16}$	$\alpha_a = 1{,}00$	Temperature and shrinkage reinforcement		$\alpha_b = 0{,}00$

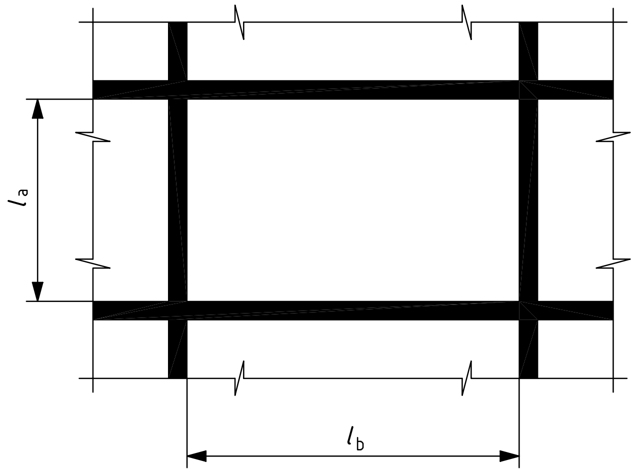


Figure 65 — Central panel of two-way slabs supported on girders, beams or structural concrete walls

Table 14 — Edge panel with l_a parallel to edge of two-way slabs supported on girders, beams or walls

Panel span ratio $\beta = l_b/l_a$	Short direction l_a			Long direction l_b		
	Negative moment	Positive moment	Load fraction	Negative moment	Positive moment	Load fraction
1,0	$M_a^- = \frac{q_u \cdot l_a^2}{16}$	$M_a^+ = \frac{q_u \cdot l_a^2}{35}$	$\alpha_a = 0{,}67$	$M_b^- = \frac{q_u \cdot l_b^2}{33}$	$M_b^+ = \frac{q_u \cdot l_b^2}{40}$	$\alpha_b = 0{,}33$
1,2	$M_a^- = \frac{q_u \cdot l_a^2}{14}$	$M_a^+ = \frac{q_u \cdot l_a^2}{28}$	$\alpha_a = 0{,}80$	$M_b^- = \frac{q_u \cdot l_b^2}{50}$	$M_b^+ = \frac{q_u \cdot l_b^2}{65}$	$\alpha_b = 0{,}20$
1,4	$M_a^- = \frac{q_u \cdot l_a^2}{13}$	$M_a^+ = \frac{q_u \cdot l_a^2}{23}$	$\alpha_a = 0{,}88$	$M_b^- = \frac{q_u \cdot l_b^2}{90}$	$M_b^+ = \frac{q_u \cdot l_b^2}{110}$	$\alpha_b = 0{,}12$
1,6	$M_a^- = \frac{q_u \cdot l_a^2}{12}$	$M_a^+ = \frac{q_u \cdot l_a^2}{21}$	$\alpha_a = 0{,}93$	$M_b^- = \frac{q_u \cdot l_b^2}{135}$	$M_b^+ = \frac{q_u \cdot l_b^2}{160}$	$\alpha_b = 0{,}07$
1,8	$M_a^- = \frac{q_u \cdot l_a^2}{12}$	$M_a^+ = \frac{q_u \cdot l_a^2}{20}$	$\alpha_a = 0{,}95$	$M_b^- = \frac{q_u \cdot l_b^2}{200}$	$M_b^+ = \frac{q_u \cdot l_b^2}{220}$	$\alpha_b = 0{,}05$
2,0	$M_a^- = \frac{q_u \cdot l_a^2}{11}$	$M_a^+ = \frac{q_u \cdot l_a^2}{18}$	$\alpha_a = 0{,}97$	$M_b^- = \frac{q_u \cdot l_b^2}{330}$	$M_b^+ = \frac{q_u \cdot l_b^2}{340}$	$\alpha_b = 0{,}03$
> 2,0	$M_a^- = \frac{q_u \cdot l_a^2}{10}$	$M_a^+ = \frac{q_u \cdot l_a^2}{16}$	$\alpha_a = 1{,}00$	Temperature and shrinkage reinforcement		$\alpha_b = 0{,}00$

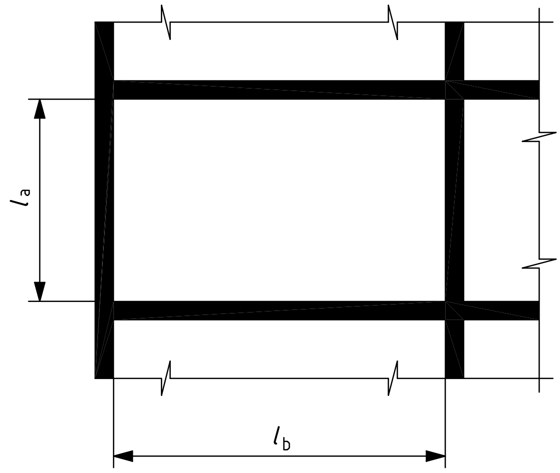


Figure 66 — Edge panel with l_a parallel to the edge of two-way slabs supported on girders, beams or walls

Table 15 — Edge panel with l_b parallel to edge, of two-way slabs supported on girders, beams or walls

Panel span ratio $\beta = l_b/l_a$	**Short direction** l_a			**Long direction** l_b		
	Negative moment	**Positive moment**	**Load fraction**	**Negative moment**	**Positive moment**	**Load fraction**
1,0	$M_a^- = \frac{q_u \cdot l_a^2}{30}$	$M_a^+ = \frac{q_u \cdot l_a^2}{39}$	$\alpha_a = 0{,}33$	$M_b^- = \frac{q_u \cdot l_b^2}{16}$	$M_b^+ = \frac{q_u \cdot l_b^2}{35}$	$\alpha_b = 0{,}67$
1,2	$M_a^- = \frac{q_u \cdot l_a^2}{19}$	$M_a^+ = \frac{q_u \cdot l_a^2}{26}$	$\alpha_a = 0{,}51$	$M_b^- = \frac{q_u \cdot l_b^2}{22}$	$M_b^+ = \frac{q_u \cdot l_b^2}{50}$	$\alpha_b = 0{,}49$
1,4	$M_a^- = \frac{q_u \cdot l_a^2}{15}$	$M_a^+ = \frac{q_u \cdot l_a^2}{20}$	$\alpha_a = 0{,}66$	$M_b^- = \frac{q_u \cdot l_b^2}{32}$	$M_b^+ = \frac{q_u \cdot l_b^2}{70}$	$\alpha_b = 0{,}34$
1,6	$M_a^- = \frac{q_u \cdot l_a^2}{12}$	$M_a^+ = \frac{q_u \cdot l_a^2}{17}$	$\alpha_a = 0{,}77$	$M_b^- = \frac{q_u \cdot l_b^2}{50}$	$M_b^+ = \frac{q_u \cdot l_b^2}{100}$	$\alpha_b = 0{,}23$
1,8	$M_a^- = \frac{q_u \cdot l_a^2}{11}$	$M_a^+ = \frac{q_u \cdot l_a^2}{15}$	$\alpha_a = 0{,}85$	$M_b^- = \frac{q_u \cdot l_b^2}{70}$	$M_b^+ = \frac{q_u \cdot l_b^2}{150}$	$\alpha_b = 0{,}15$
2,0	$M_a^- = \frac{q_u \cdot l_a^2}{10}$	$M_a^+ = \frac{q_u \cdot l_a^2}{14}$	$\alpha_a = 0{,}92$	$M_b^- = \frac{q_u \cdot l_b^2}{100}$	$M_b^+ = \frac{q_u \cdot l_b^2}{200}$	$\alpha_b = 0{,}08$
> 2,0	$M_a^- = \frac{q_u \cdot l_a^2}{9}$	$M_a^+ = \frac{q_u \cdot l_a^2}{11}$	$\alpha_a = 1{,}00$	Temperature and shrinkage reinforcement		$\alpha_b = 0{,}00$

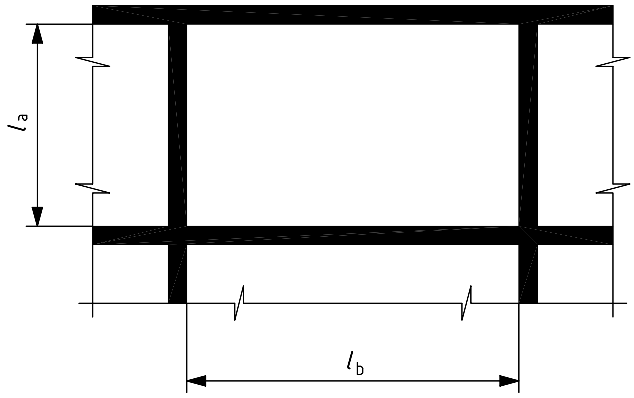


Figure 67 — Edge panel with l_b parallel to edge of two-way slabs supported on girders, beams or walls

Table 16 — Corner panel of two-way slabs supported on girders, beams or structural concrete walls

Panel span ratio $\beta = l_b/l_a$	Short direction l_a			Long direction l_b		
	Negative moment	Positive moment	Load fraction	Negative moment	Positive moment	Load fraction
1,0	$M_a^- = \frac{q_u \cdot l_a^2}{20}$	$M_a^+ = \frac{q_u \cdot l_a^2}{31}$	$\alpha_a = 0,50$	$M_b^- = \frac{q_u \cdot l_b^2}{20}$	$M_b^+ = \frac{q_u \cdot l_b^2}{31}$	$\alpha_b = 0,50$
1,2	$M_a^- = \frac{q_u \cdot l_a^2}{15}$	$M_a^+ = \frac{q_u \cdot l_a^2}{23}$	$\alpha_a = 0,67$	$M_b^- = \frac{q_u \cdot l_b^2}{30}$	$M_b^+ = \frac{q_u \cdot l_b^2}{45}$	$\alpha_b = 0,33$
1,4	$M_a^- = \frac{q_u \cdot l_a^2}{13}$	$M_a^+ = \frac{q_u \cdot l_a^2}{19}$	$\alpha_a = 0,80$	$M_b^- = \frac{q_u \cdot l_b^2}{50}$	$M_b^+ = \frac{q_u \cdot l_b^2}{70}$	$\alpha_b = 0,20$
1,6	$M_a^- = \frac{q_u \cdot l_a^2}{11}$	$M_a^+ = \frac{q_u \cdot l_a^2}{16}$	$\alpha_a = 0,87$	$M_b^- = \frac{q_u \cdot l_b^2}{75}$	$M_b^+ = \frac{q_u \cdot l_b^2}{100}$	$\alpha_b = 0,13$
1,8	$M_a^- = \frac{q_u \cdot l_a^2}{11}$	$M_a^+ = \frac{q_u \cdot l_a^2}{15}$	$\alpha_a = 0,92$	$M_b^- = \frac{q_u \cdot l_b^2}{120}$	$M_b^+ = \frac{q_u \cdot l_b^2}{150}$	$\alpha_b = 0,08$
2,0	$M_a^- = \frac{q_u \cdot l_a^2}{10}$	$M_a^+ = \frac{q_u \cdot l_a^2}{14}$	$\alpha_a = 0,96$	$M_b^- = \frac{q_u \cdot l_b^2}{165}$	$M_b^+ = \frac{q_u \cdot l_b^2}{200}$	$\alpha_b = 0,04$
$> 2,0$	$M_a^- = \frac{q_u \cdot l_a^2}{9}$	$M_a^+ = \frac{q_u \cdot l_a^2}{11}$	$\alpha_a = 1,00$	Temperature and shrinkage reinforcement		$\alpha_b = 0,00$

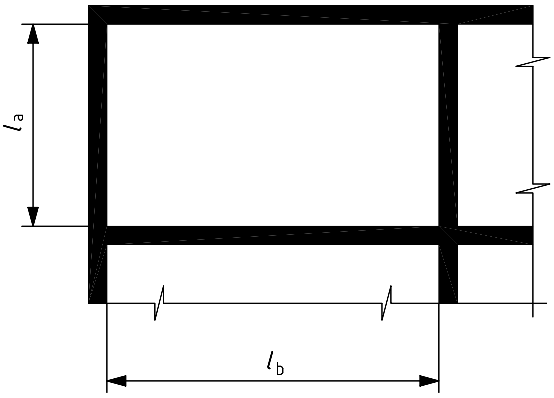


Figure 68 — Corner panel of two-way slabs supported on girders, beams or structural concrete walls

7.5.8.3 Longitudinal flexural reinforcement

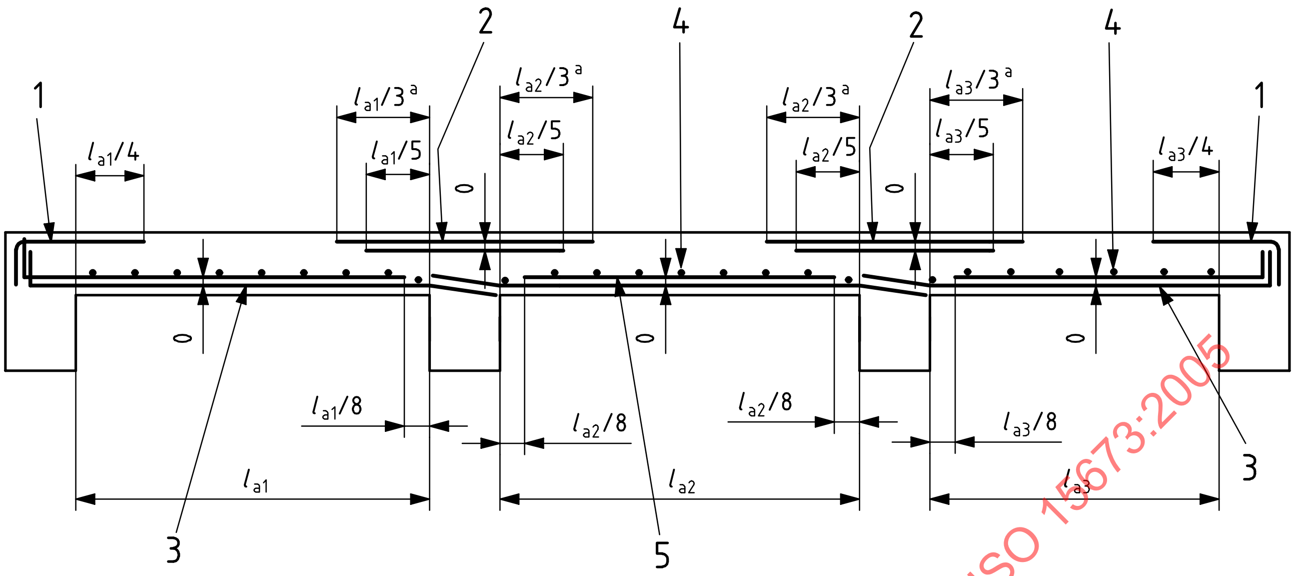
7.5.8.3.1 Positive flexural reinforcement

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 derived from the appropriate equations specified in Tables 13 to 16. 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 Equations (27) or (29) for the corresponding value of M_a^+ or M_b^+ in newton·metres per metre. M_a^+ and M_b^+ should be converted to newton·millimetres per metre (1 N·m/m = 10^3 N·mm/m), using d expressed in millimetres and b = 1 000 mm. All the provisions for positive flexural reinforcement specified in 7.5.3.3 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 centre 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 69.

7.5.8.3.2 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 derived from the appropriate equations specified in Tables 13 to 16. 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 Equations (27) or (29) for the corresponding value of M_a^- or M_b^-, expressed in newton·millimetres. M_a^- and M_b^- should be converted to newton·millimetres per metre (1 N·m/m = 10^3 N·mm/m), using d expressed in millimetres and b = 1 000 mm. All the provisions for negative flexural reinforcement specified in 7.5.3.4 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. At 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 69.

Dimensions in millimetres



Key

1 negative reinforcement at the interior face of an external support without continuity

2 negative reinforcement at the faces of interior support for more than two spans

3 positive reinforcement end span

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

5 positive reinforcement of the interior span

a Negative reinforcement cut-off points should be based upon the greater of the two neighbouring spans.

Figure 69 — Reinforcement for two-way slabs supported by girders, beams, or structural concrete walls

7.5.8.4 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, transmitted in the short and the long direction, respectively, as specified in Tables 13 to 16 for the corresponding panel edge conditions and panel span ratio, β; see Figure 70. The factored shear should not be less than the factored shear caused by factored design load, q_u, expressed in newtons per square metre, acting on a tributary area bounded by 45° lines drawn from the corner of the panel and the centreline of the panel parallel to the long span; see Figure 71.

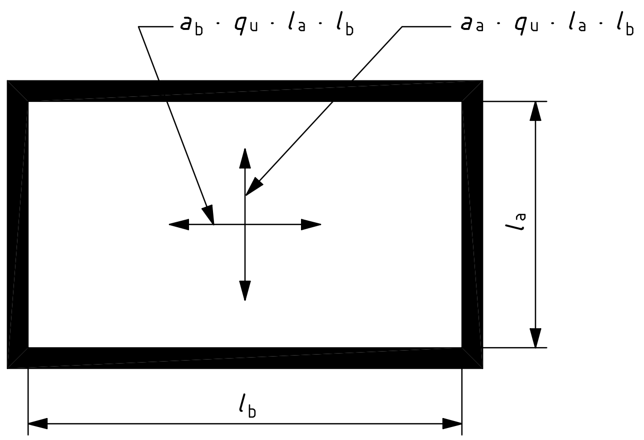


Figure 70 — Fraction of the total load in the panel transmitted in each direction in two-way slabs supported on girders, beams or structural concrete walls

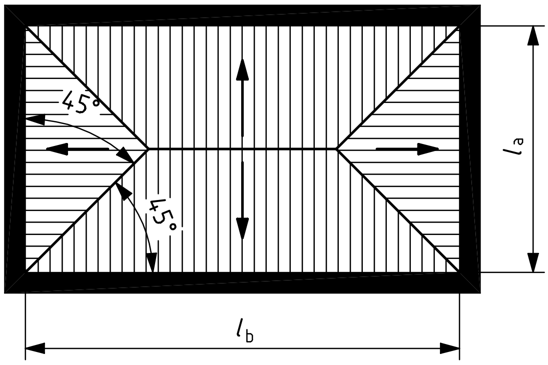


Figure 71 — Tributary areas for minimum shear at the supports of two-way slabs supported on girders, beams or structural concrete walls

The factored shear, V_u, expressed in newtons per metre, should not be less than the value derived from Equation (86) at the short span supporting element:

$$V_u = \frac{\alpha_a \cdot q_u \cdot l_b}{2} \geqslant \frac{q_u \cdot l_a}{4} \tag{86}$$

and from Equation (87) at the long span supporting element:

$$V_u = \frac{\alpha_b \cdot q_u \cdot l_a}{2} \geqslant q_u \cdot \left(\frac{l_a}{2} - \frac{l_a^2}{4 \cdot l_b} \right) \tag{87}$$

The design shear strength, $\phi \cdot V_n$, should be calculated in accordance with Equation (52) with $b_w = b =$ 1 000 mm. There should be compliance with Equation (50). This should be accomplished by using an effective depth, d, expressed in millimetres, for the slab equal to or greater than the largest value derived from Equations (88), (89) and (90):

$$d \geqslant \frac{3 \cdot q_u \cdot \alpha_a \cdot l_a}{\phi \cdot \sqrt{f'_c}} \tag{88}$$

$$d \geqslant \frac{3 \cdot q_u \cdot \alpha_b \cdot l_b}{\phi \cdot \sqrt{f'_c}} \tag{89}$$

$$d \geqslant \frac{3 \cdot q_u \cdot l_a}{2 \cdot \phi \cdot \sqrt{f'_c}} \tag{90}$$

In Equations (88) to (90), q_u should be expressed in newtons per square metre, l_a and l_b should be expressed in metres, f'_c should be expressed in megapascals, and $\phi =$ [0,85]; (see 6.3.3 d).

7.5.8.5 Calculation of the reactions on the supports

The uniformly distributed factored reaction, r_u, expressed in newtons per metre, on the support contributed by any panel of the two-way slabs in the short direction should be the value derived in accordance with Equation (91), and in the long direction, with the value derived from Equation (92):

$$r_u = \frac{V_u \cdot l}{l_a} \tag{91}$$

where

V_u is the corresponding factored shear, expressed in newtons per metre, from Equation (86) or (87);

l is the centre-to-centre span, expressed in metres, in that direction;

l_a and l_b are the corresponding clear spans, expressed in metres.

$$r_u = \frac{V_u \cdot l}{l_b} \qquad (92)$$

where the variables are the same as for Equation (91).

Total factored uniformly distributed reaction on the external supports of edge panels should be equal to the value of the factored uniformly distributed reaction, r_u, from the panel at the edge support, derived from Equation (91) or (92), 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, derived using either Equation (91) or (92), as appropriate, for both neighbouring spans at that support.

7.6 Girders, beams and joists

7.6.1 General

The design of girders, beams and joists should be performed in accordance with the requirements of 7.6. The provisions apply to isolated beams, to girders, beams and joists that are part of a floor system and to girders that are part of a moment-resisting frame supported on columns or concrete structural walls.

7.6.2 Design load definition

7.6.2.1 Loads to be included

The design load for girders, beams and joists should be established in accordance with the provisions of 7.2. 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 in accordance with the provisions of 7.8.

7.6.2.1.1 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.

7.6.2.1.2 Loads carried directly by the beam, girder or joist

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

a) dead loads, including the 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.

7.6.2.2 Factored design load

7.6.2.2.1 Factored design load for loads carried directly by the element

Factored design load for loads carried directly by the element should be in accordance with the following.

a) For uniformly distributed loads carried directly by the girder, beam or joist, the value of the uniformly distributed factored design load, w_u, expressed in newtons per metre, should be the greater of the values derived by combining w_d and w_l in accordance with Equations (3) and (4). If the girder, beam or joist, is part of a roof system, Equations (5) and (6) should also be investigated, choosing the greatest of the values from all four equations.

b) For all concentrated loads carried directly by the girder, beam or joist, the value of any concentrated factored design load, p_u, expressed in newtons, should be the greater of the values derived by combining p_d and p_l using Equation (3) and (4), for each concentrated load locations in the girder, beam or joist span.

7.6.2.2.2 Factored reactions from supported structural elements

Factored reactions from supported structural elements should be in accordance with the following.

a) The largest factored uniformly distributed reaction, r_u, expressed in newtons per metre, from all tributary structural elements should be obtained.

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

7.6.2.2.3 Total factored design load

Total factored design load should be in accordance with the following.

a) The total factored uniformly distributed load W_u, expressed in newtons per metre, should be the sum of the values derived for factored uniformly distributed loads, w_u, in accordance with 7.6.2.2.1 and reactions, r_u, in accordance with 7.6.2.2.2.

b) For all concentrated load locations in the girder, beam or joist span, the total factored concentrated load, P_u, expressed in newtons, should be the sum of the values obtained for factored concentrated loads, p_u, in accordance with 7.6.2.2.1 and reactions, R_u, in accordance with 7.6.2.2.2.

7.6.3 Details of reinforcement

7.6.3.1 General

For the purposes of this International Standard, the reinforcement of girders, beams and joists should be of the types described and should be in accordance with the provisions specified in 7.6.3.2 to 7.6.3.9.

7.6.3.2 Transverse reinforcement

7.6.3.2.1 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 7.3.6) surrounding the longitudinal bar at the end of the stirrup; see Figure 72. Under the present provisions, all stirrups in girders and beams should be closed stirrups with 135° hooks, as shown in Figure 72 (a). In joists, it should be permitted to employ all the stirrup types shown in Figure 72.

7.6.3.2.2 Minimum transverse reinforcement area

The minimum area, A_v, of shear reinforcement within a distance, s, should comply with the provisions of 7.3.13.4.4. The value of A_v corresponds to the product of the area, A_b, of the bar of the stirrup multiplied by the number of vertical legs of the stirrup.

7.6.3.2.3 Maximum and minimum spacing of stirrups

Stirrups should not be spaced further apart than permitted in accordance with the provisions of 7.3.13.4.4, nor should they be placed closer than permitted in accordance with the provisions of 7.3.7.2.

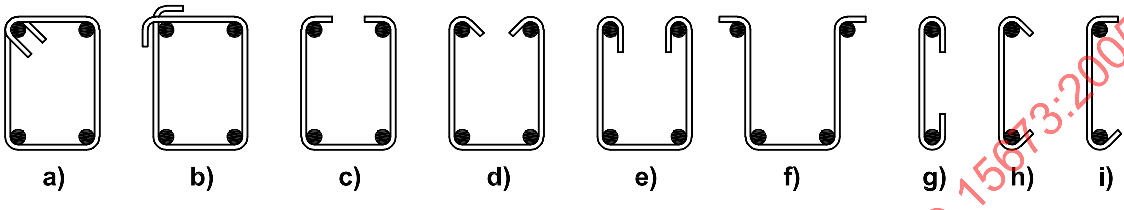


Figure 72 — Typical stirrup shapes

7.6.3.2.4 Stirrup leg splicing

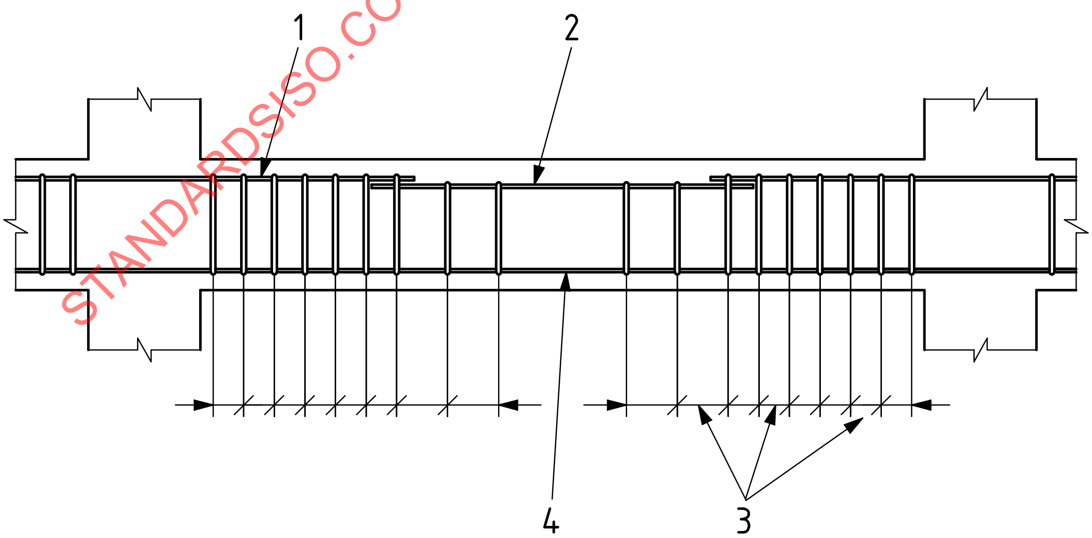
It should not be permitted to lap-splice bars that are part of stirrups.

7.6.3.2.5 Hanger reinforcement

Where beams are supported by other girders or beams of similar height, special hanger reinforcement stirrups should be provided as permitted in accordance with the provisions of 7.6.4.5.4.

7.6.3.2.6 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 the stirrups falling during the casting of the concrete; see 7.6.3.4.10.



Key

1 negative flexural reinforcement

2 stirrup supporting bars

3 stirrup separation, s

4 positive flexural reinforcement

Figure 73 — Typical stirrups spacing along the girder, beam or joist

7.6.3.3 Positive flexural reinforcement

7.6.3.3.1 Description

Positive flexural reinforcement should be provided in the lower part of the girder, beam or joist section, in accordance with the provisions of 7.6, and should comply with the general provisions specified in 7.6.3.3, and the particular provisions for each element type as specified in 7.6.4 or 7.6.5.

7.6.3.3.2 Location

Positive flexural reinforcement should be provided longitudinally in the girder, beam or joist. Positive flexural reinforcement should be located as close as permitted by the concrete cover provisions (see 7.3.4.1) 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.

7.6.3.3.3 Minimum reinforcement area

Positive flexural reinforcement should have an area at least equal to the area specified by 7.3.9.3.1. There should be compliance with minimum number of bars specified by 7.6.3.6.

7.6.3.3.4 Maximum reinforcement area

Positive flexural reinforcement area should not exceed the values specified in 7.3.9.3.2.

7.6.3.3.5 Minimum and maximum reinforcement separation

Positive flexural reinforcement should not be spaced closer than permitted by the provisions of 7.3.7.2 and 7.6.3.5. The maximum reinforcement separation should be in accordance with 7.6.3.6. When two or more layers of positive reinforcement are employed, the layers should not be placed closer than permitted by the provisions of 7.3.7.3.

7.6.3.3.6 Cut-off points

It should be permitted to suspend, at the locations specified in 7.6.4.5 or 7.6.5.5, no more than one-half of the positive flexural reinforcement required to resist the corresponding factored design positive moment at mid-span.

7.6.3.3.7 Reinforcement splicing

It should be permitted to lap-splice the remaining positive flexural reinforcement as specified in 7.6.3.3.6 between the cut-off point and the opposite face of the support.

7.6.3.3.8 Embedment at interior supports

Positive flexural reinforcement suspended at an interior support should be embedded by its continuation to the opposite face of the support, plus the distance required to be in accordance with the lap splice provisions as specified in 7.3.8.2.

7.6.3.3.9 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.

7.6.3.3.10 Positive flexural reinforcement acting in compression

Positive flexural reinforcement acting in compression should be surrounded with stirrups or ties in accordance with the provisions of 7.3.10.3.2.

7.6.3.3.11 Minimum diameter of longitudinal reinforcement

Longitudinal bars of beams and girders should have a nominal diameter, d_b, of at least 12 mm.

7.6.3.4 Negative flexural reinforcement

7.6.3.4.1 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 in accordance with the provisions of 7.6, and should be in accordance with the general provisions of 7.6.3.4 and the particular provisions of 7.6.4 or 7.6.5.

7.6.3.4.2 Location

Negative flexural reinforcement should be provided at edge and intermediate supports. Negative flexural reinforcement should be located as close as permitted by the concrete cover provisions (see 7.3.4.1) 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.

7.6.3.4.3 Minimum reinforcement area

Negative flexural reinforcement should have an area at least equal to the area specified by the provisions of 7.3.9.3.1. There should be compliance with the minimum number of bars as specified by 7.6.3.6.

7.6.3.4.4 Maximum reinforcement area

Negative flexural reinforcement area should not exceed the values specified in 7.3.9.3.2.

7.6.3.4.5 Minimum and maximum reinforcement separation

Negative flexural reinforcement should not be spaced closer than as specified by 7.3.7.2 and 7.6.3.5. The maximum reinforcement separation should be in accordance with 7.6.3.6. When two or more layers of negative reinforcement are employed, the layers should not be placed closer than permitted by the provisions of 7.3.7.3. Negative reinforcement of T-beam construction should be in accordance with 7.6.3.8.1.

7.6.3.4.6 Cut-off points

It should be permitted to suspend the negative flexural reinforcement, except for cantilevers, at the locations specified in 7.6.4.5 or 7.6.5.5. Where adjacent spans are unequal, the cut-off points of the negative flexural reinforcement should be based on the provisions for the longer span.

7.6.3.4.7 Reinforcement splicing

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

7.6.3.4.8 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, column or structural concrete wall that provides support at the edge, in accordance with the anchorage distance specified by 7.3.8.3. At the external edge of cantilevers, negative flexural reinforcement perpendicular to the edge should end in a standard hook.

7.6.3.4.9 Negative flexural reinforcement acting in compression

Negative flexural reinforcement acting in compression should be surrounded with stirrups or ties in accordance with 7.3.10.3.2.

7.6.3.4.10 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 equal to or greater than the bar diameter of the stirrups. It should be permitted to lap-splice these bars a length equal to or greater than 150 mm.

7.6.3.5 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 7.3.4), the maximum nominal coarse aggregate size and the minimum clear spacing between bars (see 7.3.7). When these computations are not performed, it should be permitted to employ the provisions of 7.6.3.5.1 to 7.6.3.5.3.

7.6.3.5.1 Girders and beams with $b_w \geqslant 300$ 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 in accordance with Equation (93)

$$N_b \leqslant \frac{b_w}{50} - 3 \tag{93}$$

where

b_w is the girder or beam width, expressed in millimetres; see Table 17;

N_b is the number of bars in a layer.

7.6.3.5.2 Girders and beams with $b_w < 300$ mm

Three longitudinal bars should be permitted for girders and beams whose width, b_w, is less than 300 mm and equal to or greater than 250 mm. Two longitudinal bars should be employed for girders and beams whose width, b_w, is less than 250 mm; see Table 17.

Table 17 — Maximum number of longitudinal bars in a layer for girders and beams

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

7.6.3.5.3 Joists

The maximum number of longitudinal bars in joists should be one for web widths, b_w, equal to or less than 150 mm (see Figure 49), but it should be permitted to bundle in contact up to two bars one located on 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 equal to or greater than 200 mm, the maximum number of bars in a single layer should be one more than those allowed for girders and beams as specified in 7.6.3.5.1 and 7.6.3.5.2.

7.6.3.6 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 rather than a small number of large-diameter bars. For joists, the minimum number of longitudinal bars should be one. There should be compliance with the provisions specified in 7.6.3.6.1 and 7.6.3.6.2 at sections of maximum positive and negative moment for girders and beams whose width, b_w, is equal to or greater than 300 mm. For girders and beams with b_w less than 300 mm, the minimum number of longitudinal bars should be two.

7.6.3.6.1 Exterior exposure

The minimum number of longitudinal bars in a layer that is employed for girders and beams that are exposed to earth or weather should be equal to or greater than the value derived from Equation (94):

$$N_b \geqslant \frac{b_w}{100} \tag{94}$$

where b_w is the girder or beam width, expressed in millimetres.

7.6.3.6.2 Interior exposure

The minimum number of longitudinal bars in a layer that is employed for girders and beams that are not exposed to earth or weather should be equal to or greater than the value derived from Equation (95):

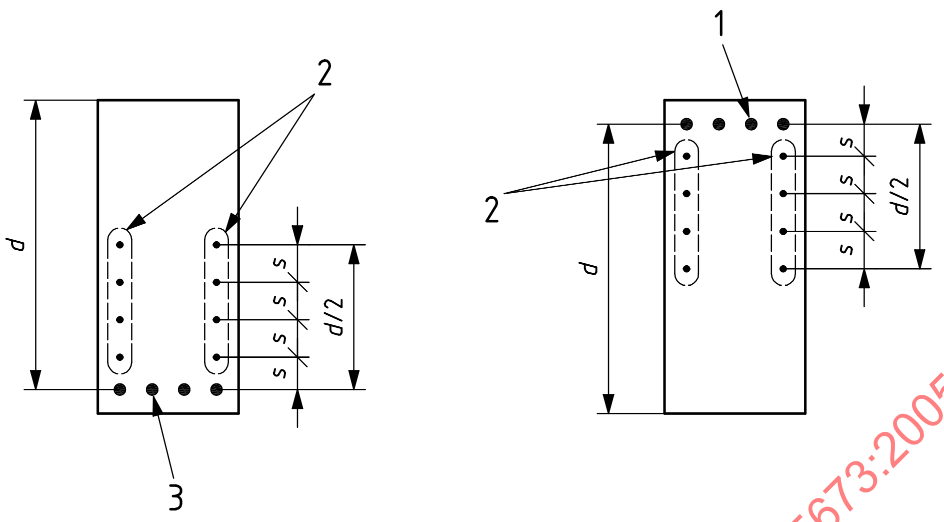
$$N_b \geqslant \frac{b_w}{200} \tag{95}$$

where b_w is the girder or beam width, expressed in millimetres.

7.6.3.7 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, expressed in millimetres, between the bars should be derived in accordance with Equation (96), but it should not exceed $d/6$ or 300 mm; see Figure 74.

$$s = \frac{1\,000 \cdot A_b}{d - 750} \tag{96}$$



Key

1 negative reinforcement in tension
2 skin reinforcement
3 positive reinforcement in tension

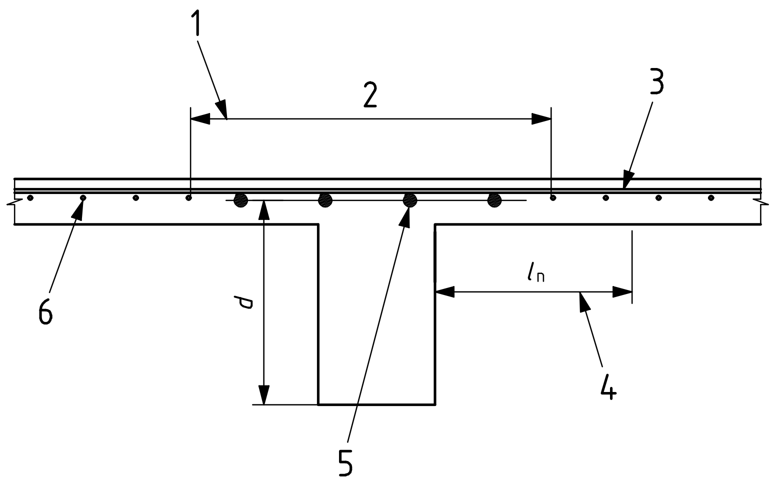
Figure 74 — Skin reinforcement for girders, beams and joists with $d > 800$ mm

7.6.3.8 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 provisions specified in 7.6.3.8.1 and 7.6.3.8.2 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 that required for the slab.

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

Where the flanges of T-beam construction are in tension, negative flexural reinforcement in the direction of the beam should be distributed over a width equal to the smaller of the effective flange width defined in 7.3.11.6 or one-tenth of the span of the beam. If the effective flange width specified in 7.3.11.6 exceeds one-tenth of the span, the rest of the effective flange width should have reinforcement in the direction of the beam equal to or greater than the shrinkage and temperature reinforcement for the slabs specified in 7.3.9.2.1; see Figure 75. In this case, the provisions of 7.6.3.5 should not apply.



Key

1 width for distribution of negative flexural reinforcement in tension

2 minimum of the effective flange width or one-tenth of the beam span

3 transverse reinforcement calculated on the assumption that the flange acts as a cantilever

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

5 beam negative flexural reinforcement in tension

6 shrinkage and temperature reinforcement outside of the width for the distribution of negative reinforcement

Figure 75 — Reinforcement in flanges of T-beams

7.6.3.8.2 Transverse flange reinforcement

In the top of the flange, reinforcement perpendicular to the beam should be provided to resist the factored negative moment derived from Equation (70), using a value of l_n equal to the overhanging portion of the flange width specified in 7.3.11.6, and for isolated T-beams the full width of overhanging flange or the flange width specified in 7.3.11.6. This reinforcement should comply with the provisions for negative flexural reinforcement in slabs in accordance with 7.5.3.4; see Figure 75.

7.6.3.9 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 7.8. Joists and beams that are not part of a frame are exempt from the additional seismic provisions.

7.6.4 Joists and beams supported on girders

7.6.4.1 General

The provisions of 7.6.4 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 specified in 7.4.1.3.4, are covered and should be in accordance with the general provisions of 7.4.

7.6.4.2 Dimensions

The following provisions for dimensions should be observed.

7.6.4.2.1 Joists

In addition to the appropriate provisions of 7.6, joists should be in accordance with the general dimensional provisions as specified in 6.1 and the particular provisions specified in 7.4.1.3.1. The minimum allowable depth should be in accordance with 7.4.5.3 for one-way joists and with 7.4.5.4 for two-way joists.

7.6.4.2.2 Beams

In addition to the appropriate provisions of 7.6, beams supported on girders should be in accordance with the general dimensional provisions of 6.1 and the particular provisions of 7.4.1.2. The minimum allowable depth should comply with 7.4.5.3. The width, b_w, of the web of beams 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 the compression flange or face.

7.6.4.2.3 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.

7.6.4.3 Factored flexural moment

7.6.4.3.1 Cantilevers of joists and beams supported on beams, girders or walls

The factored negative flexural moment, M_u^-, for beam and joist cantilevers that span beyond the edge supporting girders, beams or structural concrete walls, should be calculated on the assumption 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, ΣP_u, and the other one-half acts as a uniformly distributed load over the full span, in accordance with Equation (97), 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 moment, in the same direction, of the first interior span.

$$M_u^- = \frac{3 \cdot W_u \cdot l_n^2}{4} + l_n \cdot \sum P_u \tag{97}$$

where l_n is the clear span, expressed in metres, of the cantilever, W_u is expressed in newtons per metre, ΣP_u is expressed in newtons, and M_u^- is expressed in newton·metres.

7.6.4.3.2 One-span joists and beams supported on beams, girders or walls

The factored positive and negative flexural moment, M_u, expressed in newton·metres, for one-span beams and one-span one-way joists should be calculated using Equation (98) for positive movement at the supports and Equation (99) for negative movement at the supports:

$$M_u^+ = \frac{W_u \cdot l_n^2}{8} + \frac{l_n}{4} \cdot \sum P_u \tag{98}$$

where l_n is the clear span, expressed in metres, of the beam or joist, W_u is expressed in newtons per metre, and ΣP_u is expressed in newtons.

$$M_u^- = \frac{W_u \cdot l_n^2}{24} + \frac{l_n}{16} \cdot \sum P_u \tag{99}$$

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

The factored positive and negative flexural moment, M_u, expressed in newton·metres, for beams and one-way joists supported on beams, girders or structural walls should be calculated in accordance with Equations (100) to (105):

a) For positive moment at end spans:

$$M_u^+ = \frac{W_u \cdot l_n^2}{11} + \frac{l_n}{9} \cdot \sum P_u \tag{100}$$

where l_n is the clear span, expressed in metres, of the beam or joist, W_u is expressed in newtons per metre, and ΣP_u is expressed in newtons.

b) For positive moment at interior spans:

$$M_u^+ = \frac{W_u \cdot l_n^2}{16} + \frac{l_n}{5} \cdot \sum P_u \tag{101}$$

c) For negative moment at supports at the interior face of external supports:

$$M_u^- = \frac{W_u \cdot l_n^2}{24} + \frac{l_n}{16} \cdot \sum P_u \tag{102}$$

d) For negative moment at the exterior face of the first internal support for only two spans:

$$M_u^- = \frac{W_u \cdot l_n^2}{9} + \frac{l_n}{6} \cdot \sum P_u \tag{103}$$

e) For negative moment at the faces of internal supports for more than two spans:

$$M_u^- = \frac{W_u \cdot l_n^2}{10} + \frac{l_n}{7} \cdot \sum P_u \tag{104}$$

f) For negative moment at the faces of all supports for joists with spans not exceeding 3 m:

$$M_u^- = \frac{W_u \cdot l_n^2}{12} + \frac{l_n}{8} \cdot \sum P_u \tag{105}$$

7.6.4.3.4 Use of frame analysis for joists and beams supported on beams, girders or walls

It should be permitted to use a frame analysis for obtaining the factored moment and shear as a substitute for the values derived in accordance with 7.6.4.3.1 to 7.6.4.3.3, and 7.6.4.4.1 to 7.6.4.4.3, if the following provisions are met.

a) The analytical 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 analytical 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 $E_c = 4\,500\sqrt{f_c'}$, expressed in megapascals.

e) The 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) The span length should be taken as the distance centre-to-centre of the supports, but it should be permitted to obtain the factored moment and shear at the faces of the 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.

7.6.4.3.5 Two-way joists supported on beams, girders or walls

It should be permitted to obtain the factored moment for two-way joists supported on beams, girders or structural walls in accordance with 7.5.8.1 and 7.5.8.2, except for the minimum depth of the supporting beams or girders in accordance with the provisions of 7.5.8.1 c); see 7.4.1.3.4.

7.6.4.4 Factored shear

7.6.4.4.1 Cantilevers of joists and beams supported on beams, girders or walls

The factored shear, V_u, at the support of the cantilevers should be calculated in accordance with Equation (106):

$$V_u = W_u \cdot l_n + \sum P_u \tag{106}$$

where l_n is the clear span, expressed in metres, of the cantilever, W_u is expressed in newtons per metre and ΣP_u is expressed in newtons.

7.6.4.4.2 One-span joists and beams supported on beams, girders or walls

The factored shear, V_u, expressed in newtons, for one-span beams and one-span one-way joists should be calculated in accordance with Equation (107):

$$V_u = \frac{W_u \cdot l_n}{2} + 0{,}8 \cdot \sum P_u \tag{107}$$

where l_n is the clear span, expressed in metres, of the beam or joist, W_u is expressed in newtons per metre and ΣP_u is expressed in newtons.

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

The factored positive and negative flexural moment, M_u, expressed in newton·metres, for beams and one-way joists supported on beams, girders or structural walls should be calculated in accordance with Equation (108) for the exterior face of first interior support and Equation (109) for the faces of all other supports:

$$V_u = 1{,}15 \cdot \frac{W_u \cdot l_n}{2} + 0{,}80 \cdot \sum P_u \tag{108}$$

where l_n is the clear span, expressed in metres, of the beam or joist, W_u is expressed in newtons per metre and ΣP_u is expressed in kilonewtons.

$$V_u = \frac{W_u \cdot l_n}{2} + 0{,}75 \cdot \sum P_u \tag{109}$$

where the variables are the same as for Equation (108).

7.6.4.4.4 Use of frame analysis

It should be permitted to use a frame analysis for obtaining the factored shears as a substitute for the values derived in accordance with 7.6.4.4.1 to 7.6.4.4.3 if the provisions of 7.6.4.3.4 are met.

7.6.4.4.5 Two-way joists supported on beams, girders or walls

It should be permitted to obtain the factored shear for two-way joists supported on beams, girders or structural walls in accordance with the provisions of 7.5.8.1 and 7.5.8.4, except for the minimum depth of the supporting beams or girders in accordance with 7.5.8.1 c); see 7.4.1.3.4.

7.6.4.5 Reinforcement

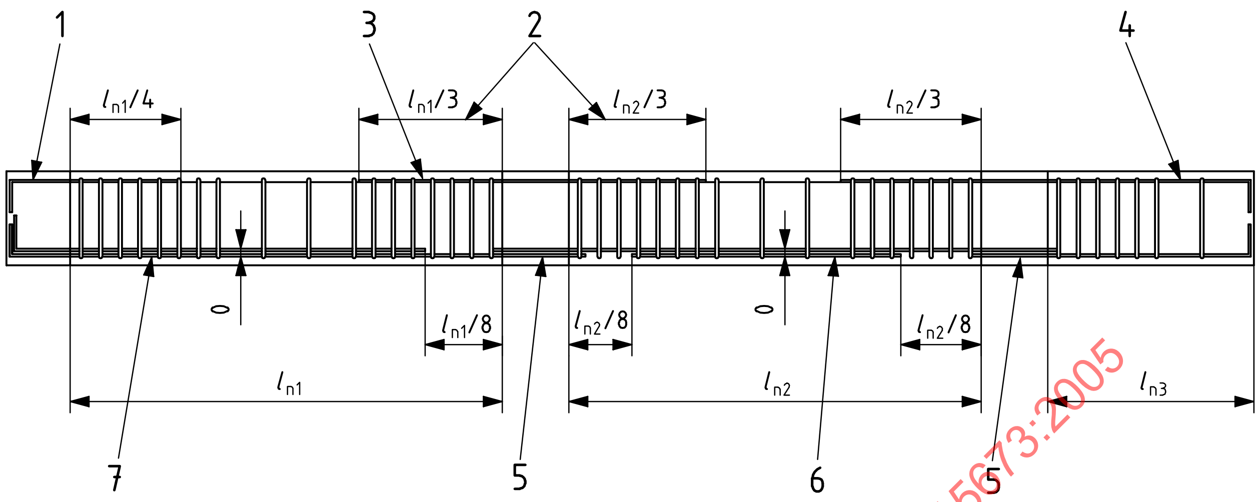
7.6.4.5.1 Positive flexural reinforcement

The positive reinforcement area should be determined in accordance with Equations (27) or (29), with the appropriate value of M_u^+ derived in accordance with 7.6.4.3 converted to newton·millimetres (1 N·m = 10^3 N·mm), using d and b expressed in millimetres. 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 specified in 7.3.11.6. The positive flexural reinforcement should comply with the provisions of 7.6.3.3. At internal supports, at a distance equal to $l_n/8$ 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 76.

7.6.4.5.2 Negative flexural reinforcement

The negative flexural reinforcement area should be determined in accordance with Equations (27) or (29), for the greater of the values of M_u^- derived from 7.6.4.3 for both sides of the support, converted to newton·millimetres (1 N·m = 10^3 N·mm), using d and b expressed in millimetres. This reinforcement should comply with the provisions of 7.6.3.4. 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 7.6.3.8. At a distance equal to $l_n/4$ for external supports and $l_n/3$ 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 76. No suspension of negative reinforcement should be permitted in cantilevers.

Dimensions in millimetres



Key

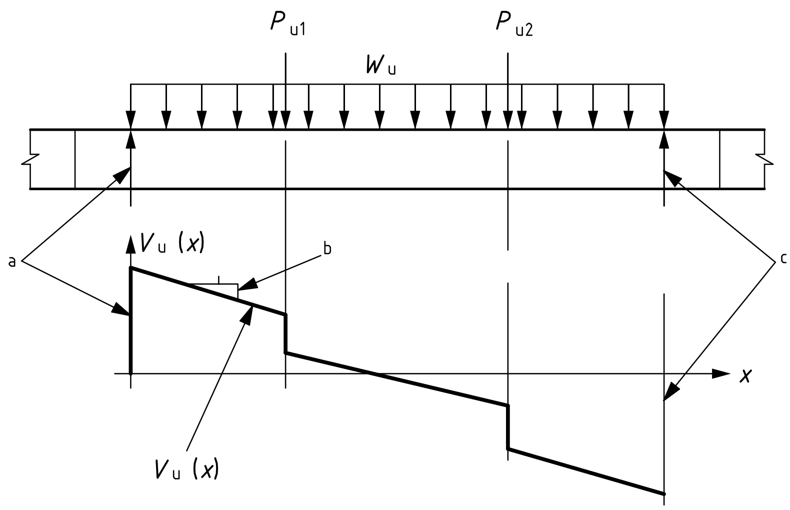
1 negative reinforcement at the interior face of the external support
2 negative reinforcement at cut-off points based on the greater of the neighbouring spans
3 negative reinforcement of the interior support
4 greater of the cantilever negative reinforcement or that required for internal support
5 splice in accordance with 7.3.8.2
6 positive reinforcement of the interior span
7 positive reinforcement of the end span

Figure 76 — Reinforcement for beams and joists supported on beams or girders

7.6.4.5.3 Transverse reinforcement

The values of V_u at the faces of the right and left supports should be derived using the appropriate equation in accordance with 7.6.4.4. A diagram showing the shear variation within the span should be constructed, starting with the value of V_u, expressed in newtons, at the face of the left support taken as positive. The shear proceeding to the right from this point should be decreasing at a rate equal to $[(V_u)_{\text{left supp.}} + (V_u)_{\text{right supp.}} - \Sigma P_u]/l_n$, expressed in kilonewtons. At any place where a concentrated load is applied, the value of P_u, expressed in newtons, 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, expressed in newtons, should be reached in the diagram; see Figure 77. At any place within the span, the value of $(l \cdot V_n)$ as calculated in accordance with the provisions of 7.3.13.4 should be equal to or greater than the absolute value of $V_u(x)$ as shown in the graph of the calculation.

The shear reinforcement should comply with the provisions of 7.6.3.2 and 7.3.13.4. The limits for $(\phi \cdot V_n)$ as defined in 7.3.13.4.4 should be marked in the shear diagram, and a minimum amount of shear reinforcement in accordance with Equation (55) should be established. Appropriate values for the spacing, s, of the stirrups should be defined for the different region within the shear diagram. A minimum practicable spacing of stirrups in accordance with the provisions of 7.3.7.2 should be observed. The first stirrup should not be placed further than $s/2$ from the face of the supporting element, s being the required spacing of stirrups at the support.



Key

a $(V_u)_{\text{left supp.}}$

b $[(V_u)_{\text{left supp.}} + (V_u)_{\text{right supp.}} - \Sigma P_u]/l_n$.

c $(V_u)_{\text{right supp.}}$

Figure 77 — Calculation of the shear diagram of the beam or joist supported on beams or girders

7.6.4.5.4 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 78. The determination of the hanger reinforcement should be made in accordance with the following.

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]; see 6.3.3 d).

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, determined in accordance with Equation (110):

$$A_i \geqslant \frac{\left(1 - \frac{h_b}{h_s}\right) V_u}{\phi \cdot f_y} \tag{110}$$

where

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;

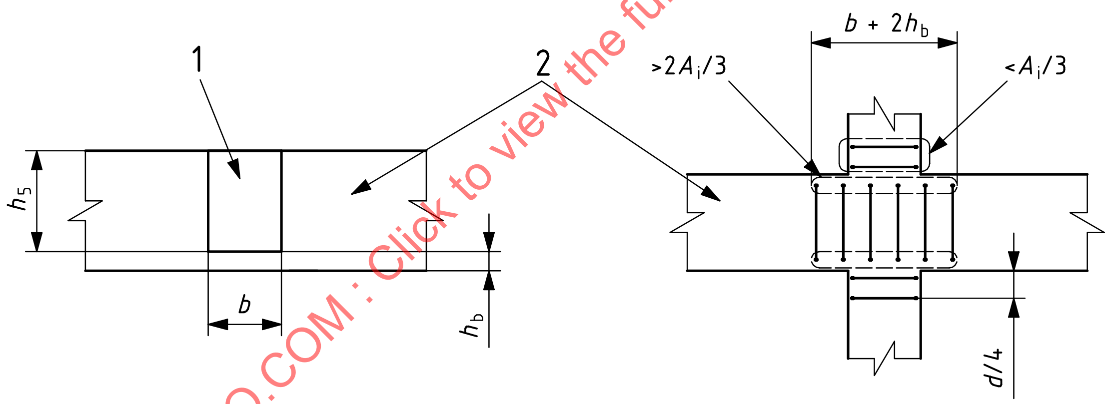
f_y is the specified yield strength of the steel of the stirrups;

ϕ is [0,85]; see 6.3.3 d).

d) Additional stirrups with an area equal to or greater than 2/3 of A_i should be placed in the supporting girder for a distance along the longitudinal axis of the supporting girder equal to or less than the width, b_w, of the supported beam 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.

e) 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 (where d is the effective depth of the supported beam) along the longitudinal axis of the supported beam from the face of the supporting girder.

f) The bottom longitudinal reinforcement of the supported beam should be placed over the bottom longitudinal reinforcement of the supporting girder.



Key

1 supported beam

2 supporting girder

Figure 78 — Hanger reinforcement

7.6.4.6 Calculation of the reactions on beams and girders

7.6.4.6.1 One-way joists

The factored reaction on the supports of joist should be permitted to be considered uniformly distributed. The factored reaction, r_u, expressed in newtons per metre, on the supports should be the value derived in accordance with Equation (111) plus the uniformly distributed reaction from any cantilever spanning that support. In Equation (111), V_u is the factored shear, expressed in newtons, in accordance with 7.6.4.4; l is the centre-to-centre span, expressed in metres, of the joist; l_n is the clear span, expressed in metres, of the joist; and s is the centre-to-centre spacing, expressed in metres, between joists; see Figure 49.

$$r_u = \frac{V_u \cdot l}{s \cdot l_n} \tag{111}$$

7.6.4.6.2 Two-way joists supported on beams, girders or walls

It should be permitted to obtain the required factored reactions for two-way joists supported on beams, girders or structural walls in accordance with the provisions of 7.5.8.1 and 7.5.8.5, except for the minimum depth of the supporting beams or girders as specified in 7.5.8.1 c); see 7.4.1.3.4.

7.6.4.6.3 Beams

The factored reaction, R_u, expressed in newtons, on the supports should be the value derived in accordance with Equation (112) plus the factored reaction from any cantilever spanning from that support:

$$R_u = \frac{V_u \cdot l}{l_n} \tag{112}$$

where

V_u is the factored shear, expressed in newtons, from 7.6.4.4;

l is the centre-to-centre span of the beam, expressed in metres;

l_n is the clear span, expressed in metres, of the beam.

7.6.5 Girders that are part of a frame

7.6.5.1 General

The provisions of 7.6.5 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.

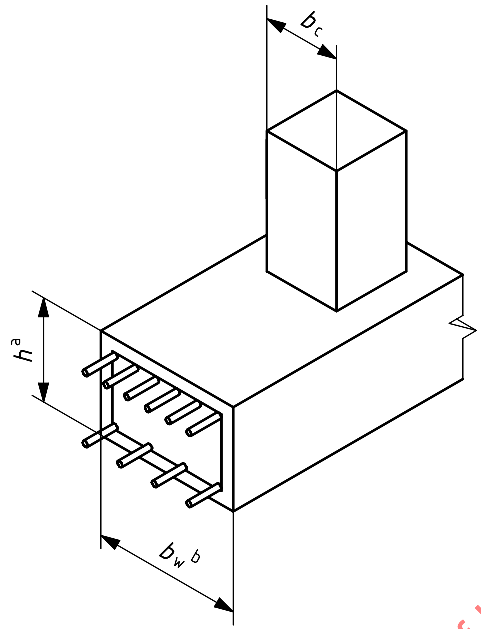
7.6.5.2 Dimensions

7.6.5.2.1 General

In addition to the appropriate provisions of 7.6, girders that are part of a frame should comply with the general dimensional provisions as specified in 6.1 and the particular provisions for beams spanning between columns as specified in 7.4.1.

7.6.5.2.2 Girder depth and width

The girder should be prismatic without haunches, brackets or corbels. The height, h, should comply with the minimum thickness as specified in 7.4.5.3. The clear span of the member should not be less than four times its height, h. The width-to-height ratio, b_w/h, should not be less than 0,3. The width, b, 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 79.



Key

a Restrictions on h: equal to or greater than the minimum of the values calculated in accordance with 7.4.5.3;
$< l_n/4$

b Restrictions on b_w: $\geqslant$ 200 mm
$\geqslant 0{,}3 \cdot h$
$< b_c + 1{,}5 \cdot h$

Figure 79 — Limits on girder depth and width

7.6.5.2.3 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 either the thickness of the wall or 200 mm. Vertical reinforcement of the wall should pass through the girder or beam as specified in 7.9.

7.6.5.2.4 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.

7.6.5.2.5 Special provisions

The following restrictions should be in effect for girders of frames designed in accordance with 7.6.5:

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 % of the larger span; see 6.1.

c) Loads are uniformly distributed and adjustments for concentrated loads are performed.

d) The unit live load, w_l, does not exceed three times unit dead load, w_d.

e) Sloping girders should not have a slope exceeding 15°.

7.6.5.3 Factored flexural moment

7.6.5.3.1 Factored positive and negative moment

The factored positive and negative moment, M_u, expressed in newton·metres, for girders and beams that are part of a frame where the vertical elements are columns and concrete structural walls, should be calculated in accordance with Equations (113) to (119):

a) For a positive moment at end spans:

$$M_u^+ = \frac{W_u \cdot l_n^2}{14} + \frac{l_n}{6} \cdot \sum P_u \tag{113}$$

where

l_n is the clear span, expressed in metres;

W_u is expressed in newtons per metre;

$\sum P_u$ is the sum of all total factored concentrated loads, expressed in newtons, that act on the span.

b) For a positive moment at interior spans:

$$M_u^+ = \frac{W_u \cdot l_n^2}{16} + \frac{l_n}{7} \cdot \sum P_u \tag{114}$$

c) For a negative moment at supports for the interior face of an external column or perpendicular structural wall:

$$M_u^- = \frac{W_u \cdot l_n^2}{16} + \frac{l_n}{10} \cdot \sum P_u \tag{115}$$

d) For a negative moment at the exterior face of the first internal column or perpendicular structural wall for only two spans:

$$M_u^- = \frac{W_u \cdot l_n^2}{9} + \frac{l_n}{6} \cdot \sum P_u \tag{116}$$

e) For a negative moment at the faces of internal columns or perpendicular structural walls for more than two spans:

$$M_u^- = \frac{W_u \cdot l_n^2}{10} + \frac{l_n}{6,5} \cdot \sum P_u \tag{117}$$

f) For a negative moment at the faces of structural walls parallel to the plane of the frame:

$$M_u^- = \frac{W_u \cdot l_n^2}{12} + \frac{l_n}{7} \cdot \sum P_u \tag{118}$$

g) For a negative moment at the support of girder cantilevers:

$$M_u^- = \frac{3 \cdot W_u \cdot l_n^2}{4} + l_n \cdot \sum P_u \tag{119}$$

7.6.5.3.2 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 7.4.1.3.3) 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 factor should also be taken into consideration in deriving the factored shear as specified in 7.6.5.4.1.

7.6.5.3.3 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 derived in accordance with 7.6.5.3.1 and 7.6.5.4.1 if the following provisions 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 $E_c = 4\,500\sqrt{f'_c}$, expressed in megapascals.

d) The 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) The span length should be taken as the distance centre-to-centre of the supports, but it should be permitted to obtain the factored moment and shear at the faces of the supports.

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

7.6.5.4 Factored shear

7.6.5.4.1 Calculation of factored shear

The value of V_u, expressed in newtons, for the slab should be calculated at the faces of all supports using the Equations (120) to (122):

a) at exterior face of first interior column:

$$V_u = 1{,}15 \cdot \frac{W_u \cdot l_n}{2} + 0{,}80 \cdot \sum P_u \qquad (120)$$

where

l_n is the clear span, expressed in metres;

W_u is expressed in newtons per metre;

$\sum P_u$ is the sum of all total factored concentrated loads, expressed in newtons, that act on the span.

b) at faces of all other columns:

$$V_u = \frac{W_u \cdot l_n}{2} + 0{,}75 \cdot \sum P_u \qquad (121)$$

c) at supports of girder cantilevers:

$$V_u = W_u \cdot l_n + \sum P_u \qquad (122)$$

7.6.5.4.2 Use of frame analysis

It should be permitted to use a frame analysis for obtaining the factored shears as a substitute for the values derived in accordance with 7.6.5.4.1 if the provisions of 7.6.5.3.3 are met.

7.6.5.5 Reinforcement

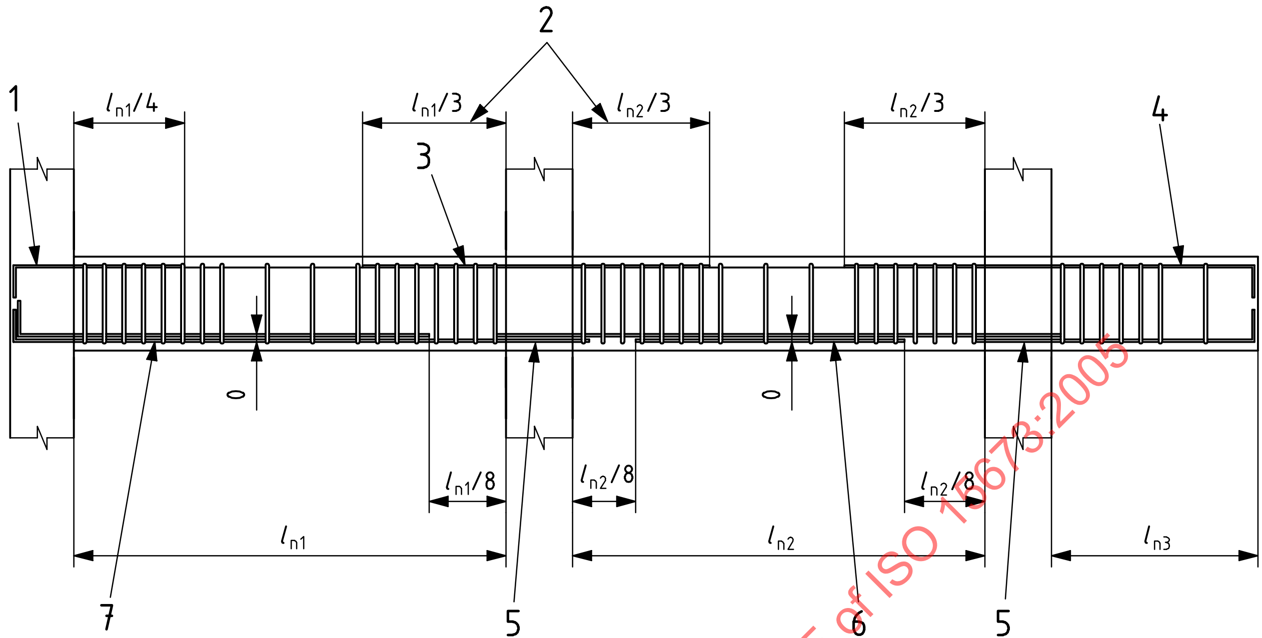
7.6.5.5.1 Positive flexural reinforcement

The positive flexural reinforcement area should be determined in accordance with the provisions of 7.3.11.4, with the appropriate value of M_u^+ derived from Equation (113) or (114), converted to newton·millimetres (1 N·m = 10^3 N mm), with d and b expressed in millimetres. If a slab exists in the upper part of the girder, it should be permitted to employ the provisions taking into account the T-beam effect specified in 7.3.11.6. Positive flexural reinforcement should be in accordance with the provisions of 7.6.3.3. At internal supports, at a distance equal to $l_n/8$ 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 Figures 80 and 81.

7.6.5.5.2 Negative flexural reinforcement

The negative flexural reinforcement area should be determined in accordance with the provisions of 7.3.11.4, for the greater of the values of M_u^- for both sides of the support, derived from Equations (115) to (119), converted to newton·millimetres (1 N·m = 10^3 N·mm), with d and b expressed in millimetres. This reinforcement should be in accordance with the provisions of 7.6.3.4. 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 be in accordance with 7.6.3.8. At internal supports at a distance equal to $l_n/3$ and at external supports at a distance equal to $l_n/4$ measured from the face of the support into the span, all the negative flexural reinforcement should be permitted to be suspended (see Figure 80), except in the beams belonging to the perimeter ring of beams as specified by 7.4.3.2, where one-quarter of the negative reinforcement should be continuous through the span, or spliced at mid-span; see Figure 81. No suspension of negative reinforcement should be permitted in cantilevers.

Dimensions in millimetres



Key

1 negative reinforcement at the interior face of an external

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

3 negative reinforcement interior support

4 greater of the cantilever negative reinforcement or required for internal support

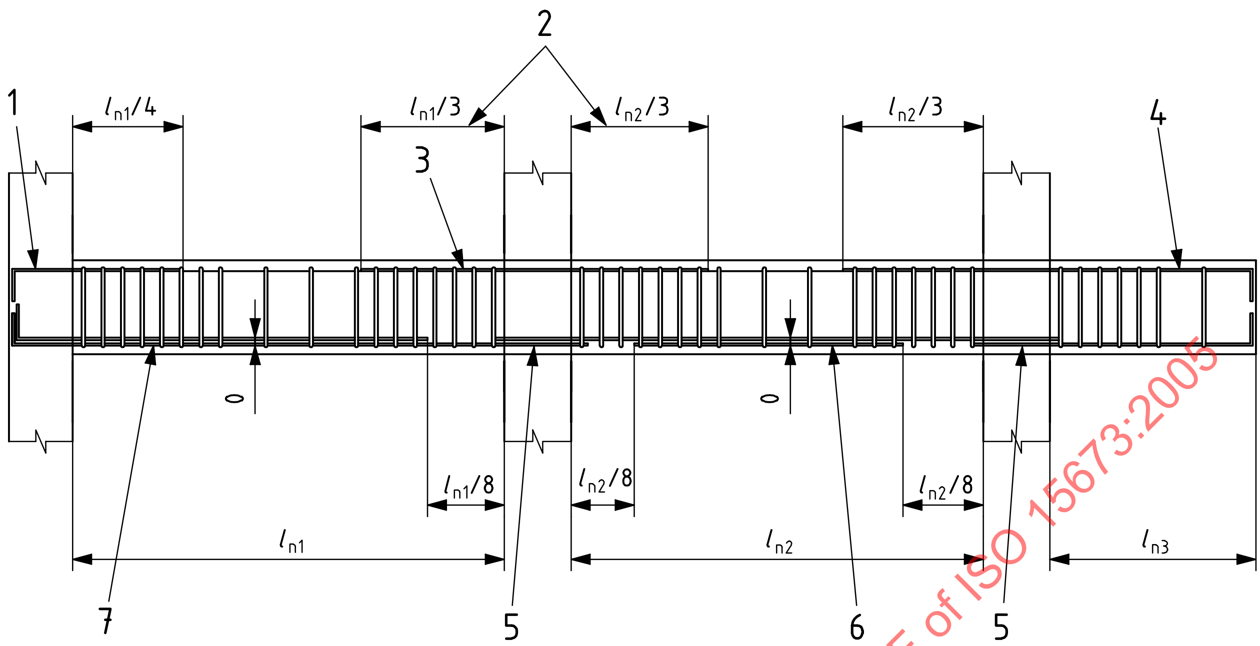
5 splice according to 7.3.8.2

6 positive reinforcement interior span

7 positive reinforcement end span

Figure 80 — Reinforcement in girders that are part of a moment resisting frame by columns or structural concrete walls

Dimensions in millimetres



Key

1 negative reinforcement at the interior face of an external

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

3 negative reinforcement interior support

4 greater of the cantilever negative reinforcement or required for internal support

5 splice according to 7.3.8.2

6 positive reinforcement interior span

7 positive reinforcement exterior span

Figure 81 — Reinforcement in girders that are part of the perimeter frame

7.6.5.5.3 Transverse reinforcement

The values of V_u at the faces of the right and left supports should be derived using the appropriate value from Equation (120) to (122). A diagram of shear variation along the span should be calculated in accordance with the provisions of 7.6.4.5.3 (see Figure 77). At any place within the span, the value of $(\phi \cdot V_n)$ as calculated in accordance with the provisions of 7.3.13.4 should be greater than or equal to the absolute value of $V_u(x)$ as shown in the calculated diagram.

The shear reinforcement should be in accordance with the provisions of 7.6.3.2 and 7.3.13.4. The limits for $(\phi \cdot V_n)$ as defined in 7.3.13.4.4 should be marked on the shear diagram, and a minimum amount of shear reinforcement as defined by Equation (55) should be established. Appropriate values of the spacing, s, of the stirrups should be defined for the different regions within the shear diagram. A minimum practicable spacing of stirrups as specified by 7.3.7.2 should be observed. The first stirrup should not be placed further than $s/2$ from the face of the supporting element, s being the required spacing of stirrups at the support.

7.6.5.5.4 Hanger reinforcement

When the girder supports a beam of essentially the same depth, hanger reinforcement as specified by 7.6.4.5.4 should be employed.

7.6.5.6 Calculation of the reactions on columns and structural concrete walls

7.6.5.6.1 Vertical reaction at column and walls

The factored reaction, R_u, on the supports expressed in newtons, should be the value derived in accordance with Equation (123) plus the factored reaction from any cantilever spanning from that support:

$$R_u = \frac{V_u \cdot l}{l_n} \tag{123}$$

where

V_u is the factored shear, expressed in newtons, from Equations (120) to (122)

l is the centre-to-centre span, expressed in metres, of the beam;

l_n is the clear span of the beam, expressed in metres.

7.6.5.6.2 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 in the plane of the frame that spans 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, ΔM_u, 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 82) 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 82) the whole girder should be loaded with the factored dead load and the other alternate spans should be loaded with the factored live load.

7.6.5.6.3 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 83), the column factored moment should correspond to ΔM_u.

b) For joints of columns, or walls, of floors different from the roof (Types A, C and E of Figure 83), the unbalanced moment should be distributed to the column or wall above in accordance with Equation (124):

$$\left(M_u\right)_{up} = \Delta M_u \cdot \frac{\left(I_c/h_{pi}\right)_{up}}{\left(I_c/h_{pi}\right)_{up} + \left(I_c/h_{pi}\right)_{down}} \tag{124}$$

c) For joints of columns or walls of floors different from the roof (Types A, C and E of Figure 83), the unbalanced moment should be distributed to the column or wall in accordance with Equation (125):

$$\left(M_u\right)_{down} = \Delta M_u \cdot \frac{\left(I_c/h_{pi}\right)_{down}}{\left(I_c/h_{pi}\right)_{up} + \left(I_c/h_{pi}\right)_{down}} \tag{125}$$

d) In Equations (124) and (125), I_c should be evaluated in accordance with Equation (126):

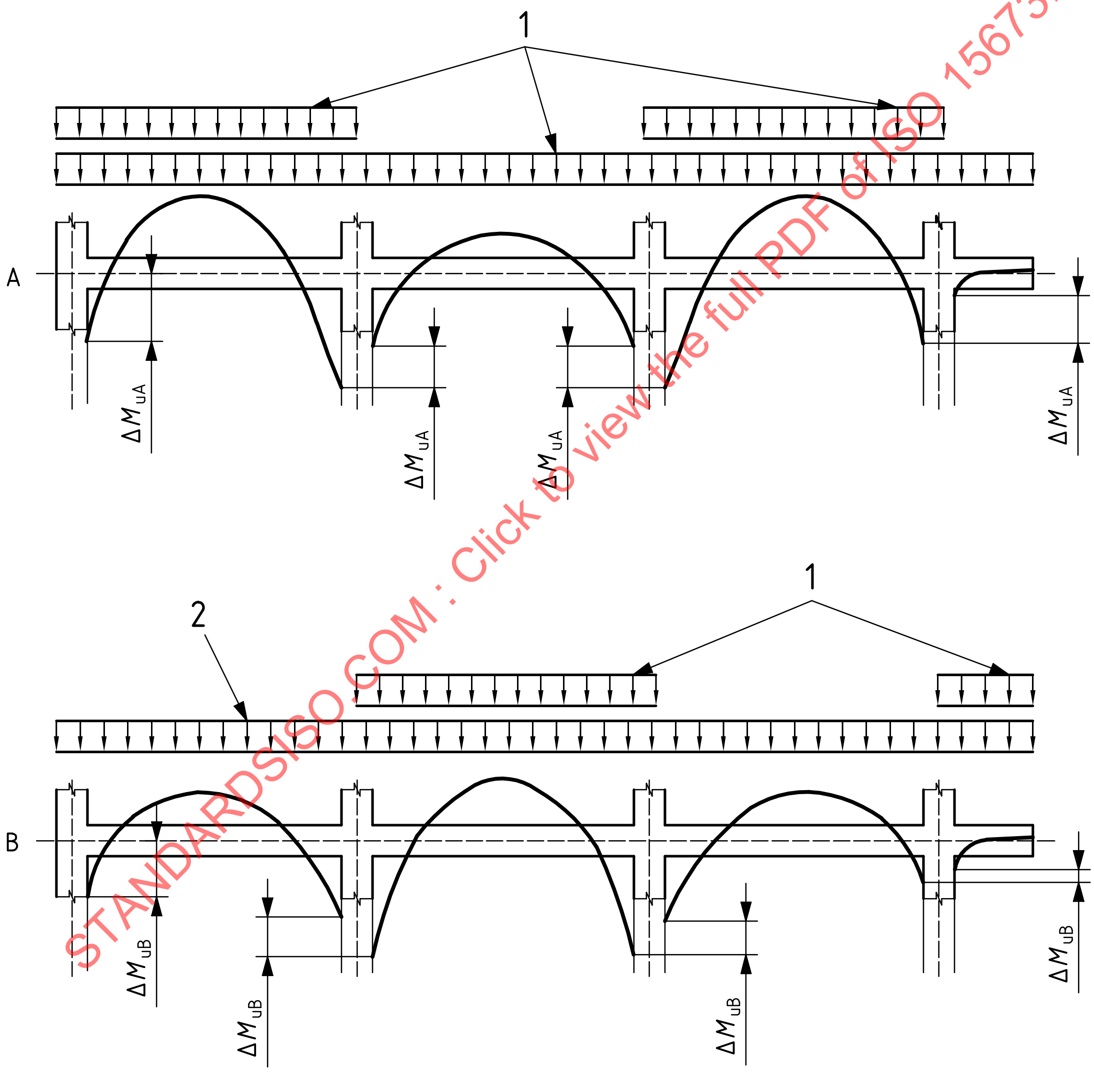
$$I_c = \frac{b_c \cdot h_c^3}{12} \tag{126}$$

e) where

b_c is the dimension, expressed in metres, of the column or wall section in the direction perpendicular to the girder span;

h_c is the dimension, expressed in metres, of the column, or wall, section in the direction perpendicular to the girder span;

h_{pi} is the story height corresponding to the column or wall (see Figure 82).



Key

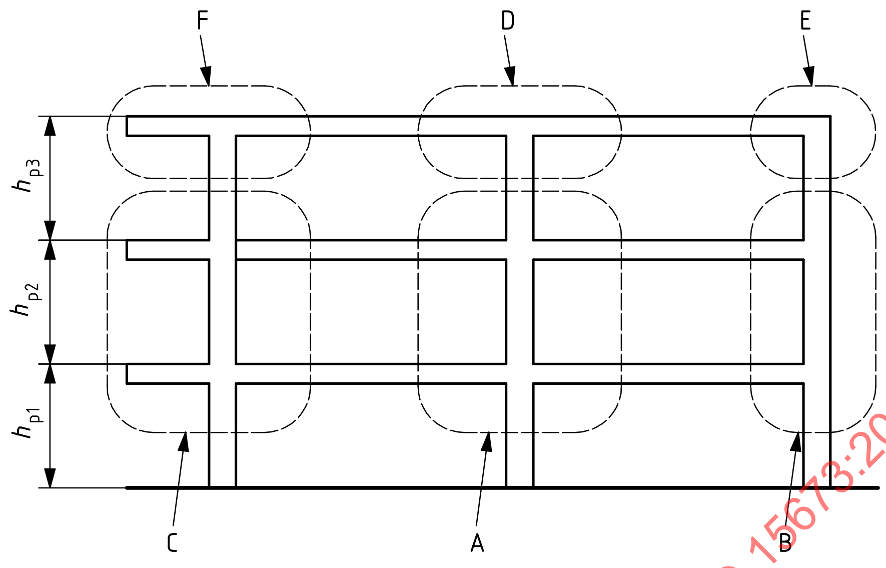
1 factored live load

2 factored dead load

A Case A

B Case B

Figure 82 — Girder unbalanced moment to be transferred to columns



Key

A to E types of supports

Figure 83 — Types of joints for determination of column moments

7.7 Columns

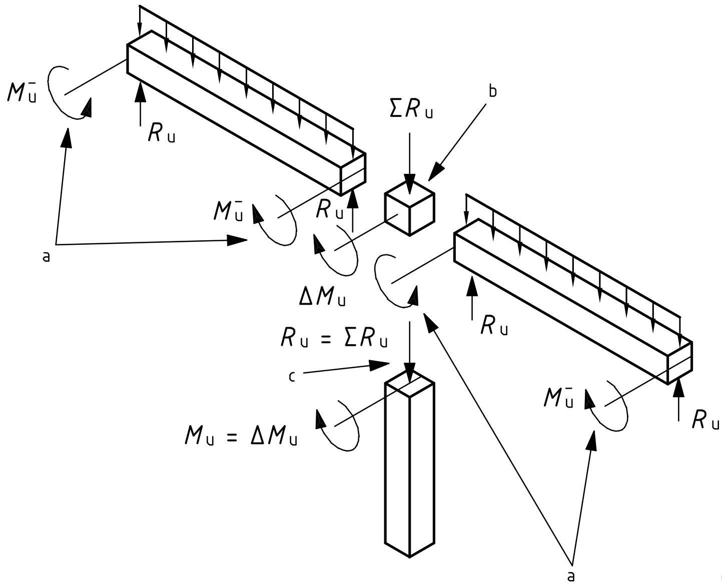
7.7.1 General

The design of columns should be performed in accordance with the provisions of 7.7. The members covered by this subclause are members reinforced with longitudinal bars and lateral ties, and members reinforced with longitudinal bars and continuous spirals. Both rectangular and circular sections are covered.

7.7.2 Design load definition

7.7.2.1 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 floor located above the column, plus the selfweight of the column. Tributary loads should be established in accordance with the provisions of 7.2.1 and the particular provisions for each tributary element type; see Figures 84 and 85.



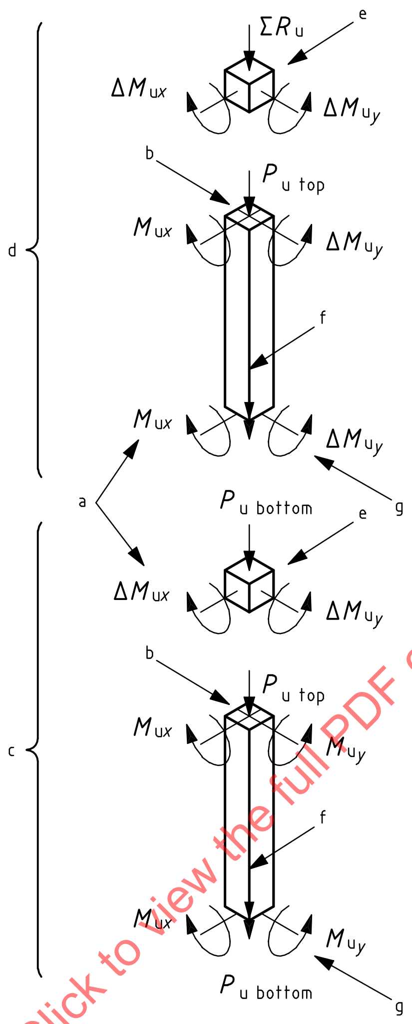
Key

a Reactions at the ends of the element.

b Actions at the joint.

c Loads applied to the top of the column.

Figure 84 — Column factored design forces from one story and in one direction



Key

a Column moment obtained from the distribution of unbalanced moment at the joint at the same level.

b Loads applied to the top of the column.

c Story n–1.

d Story n.

e Actions at the joint of story n from the tributary elements.

f Selfweight of the column.

g $P_{\text{u bottom}}$ equal to $P_{\text{u top}}$ plus the column selfweight.

h Actions at the joint of story n–1 from the tributary elements.

i $P_{\text{u top}}$ of story n–1 equal to $P_{\text{u bottom}}$ of story n plus ΣR_{u} from story n–1.

Figure 85 — Column forces in different floors

7.7.2.2 Dead load and live load

The values of P_{d} for dead load and P_{l} for live load should be expressed in newtons. P_{d} should include the selfweight of the column, assuming concrete unit weight of 24×10^3 N/m^3. The selfweight should be factored employing the load factors for dead load of the corresponding combination in accordance with 7.2.2.1. It should be permitted to apply the selfweight of the column corresponding to each floor to the lower part of the column in that floor; see Figure 85.

7.7.2.3 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 the lower part of the column in each story. A distinction should be made regarding the direction of the axes in the plan along which the moments M_{ux} and M_{uy} act; see Figure 85.

7.7.3 Dimensions

7.7.3.1 General

In addition to the appropriate provisions specified in 7.7.3, the columns should be in accordance with the general provisions for dimension specified 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. The column section shape should be either rectangular or circular. All other cross-section shapes are beyond the scope of this International Standard.

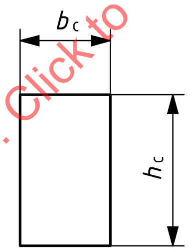
7.7.3.2 Limiting section dimensions

7.7.3.2.1 Minimum section dimensions for rectangular columns

For the purposes of this International Standard, the dimension for the cross-section of rectangular columns should be in accordance with the following limits (see Figure 86):

a) The shortest cross-sectional dimension should not be less than 300 mm. For columns in buildings located in seismic risk zones, see 7.8.5.3.1.

b) The ratio of the largest cross-sectional dimension to the perpendicular shortest dimension should not exceed 3.



Key

Restrictions: $b_c \leqslant 300$ mm; $h_c/b_c \leqslant 3$.

Figure 86 — Minimum cross-section dimensions for rectangular columns

7.7.3.2.2 Minimum section dimensions for circular columns

Columns with circular cross-section should have a diameter of at least 300 mm.

Dimension in millimetres

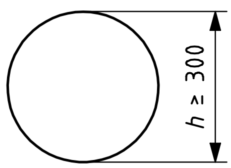
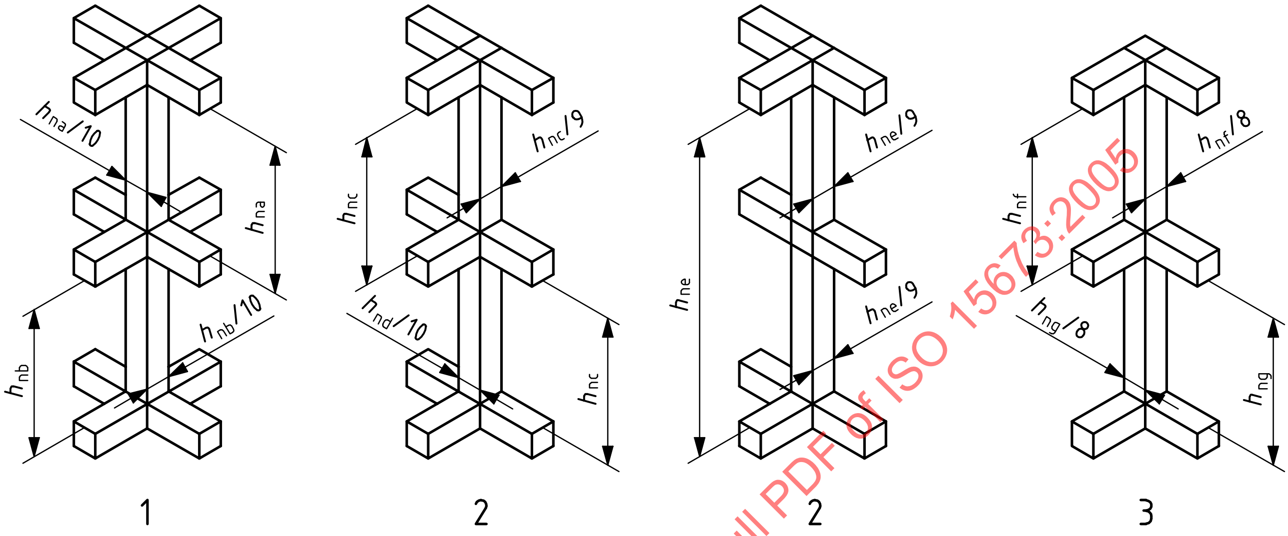


Figure 87 — Minimum cross-section dimension for circular columns

7.7.3.3 Distance between lateral supports

7.7.3.3.1 General

It should be considered that lateral restraint is provided by the floor system in the two horizontal directions at all levels that are supported by the column; see Figure 88.



Key

1 central
2 edge
3 corner

Figure 88 — Lateral restraint for columns

7.7.3.3.2 Central columns

The clear distance, h_n, between lateral supports for central columns should not exceed 10 times the dimension of the column cross-section parallel to the direction of the support; see Figure 88.

7.7.3.3.3 Edge columns

The clear distance, h_n, between lateral supports perpendicular to an edge for edge column should not exceed 9 times the dimension of the column cross-section perpendicular to the edge; see Figure 88.

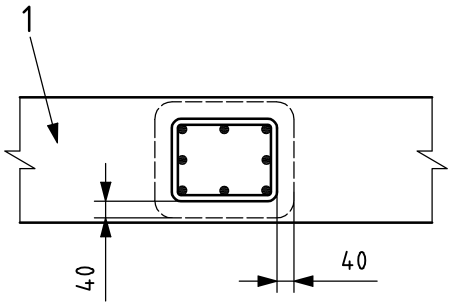
7.7.3.3.4 Corner columns

The clear distance, h_n, between lateral supports for corner columns should not exceed 8 times the minimum dimension of the column cross-section; see Figure 88.

7.7.3.4 Column built monolithically with a wall

The outer limits of the effective cross-section of a tied or spirally reinforced column built monolithically with a concrete wall should extend more than 40 mm outside the tie or spiral reinforcement or the lateral wall faces; see Figure 89.

Dimensions in millimetres



Key

1 wall

Figure 89 — Effective cross-section of columns built monolithically with a wall

7.7.4 Details of reinforcement

7.7.4.1 General

For the purposes of this International Standard, the reinforcement of columns should be of the types in accordance with 7.7.4 and should be in accordance with the provisions of 7.7.4.2 to 7.7.4.4.

7.7.4.2 Longitudinal reinforcement

7.7.4.2.1 Description and location

Longitudinal reinforcement should be provided in the periphery of the column section, as specified in 7.3.9.4.4. Longitudinal reinforcement should be located as close to the lateral surfaces of the column as the concrete cover provisions permit (see 7.3.4.1 and 7.7.4.2.9). The amount of longitudinal reinforcement should be such as to resist the simultaneous action of a combination of factored axial load and factored moments at the section acting about the two main axes of the section of the column; see Figure 91.

7.7.4.2.2 Minimum and maximum longitudinal reinforcement area

The maximum and minimum longitudinal reinforcement area should be in accordance with the provisions of 7.3.9.4.1 ($0{,}01 \leqslant \rho_t \leqslant 0{,}06$). The maximum longitudinal reinforcement area is also limited by the beam reinforcement in the beam–column joint.

7.7.4.2.3 Minimum diameter of longitudinal bars

Longitudinal bars of columns should comply with the provision of a minimum nominal diameter, d_b, of 16 mm as specified in 7.3.9.4.2.

7.7.4.2.4 Minimum number of longitudinal bars

The minimum number of longitudinal bars in rectangular and round columns should be 4 bars in rectangular columns or 6 bars in circular columns as specified in 7.3.9.4.3.

7.7.4.2.5 Minimum and maximum reinforcement separation

Longitudinal reinforcement should not be spaced closer than 1,5 d_b or 40 mm as specified in 7.3.7.4.

7.7.4.2.6 Reinforcement splicing

It should be permitted to lap-splice up to one-half of the longitudinal reinforcement at any given section, as long as only alternate bars are lap-spliced; see Figure 91. All lap splices of longitudinal reinforcement should be in accordance with 7.3.8.2.1. [Alternative methods, like gas pressure welding or mechanical connectors, could be used taking account of the practical situation of each country.]

7.7.4.2.7 End anchorage of reinforcement

Longitudinal reinforcement at the upper end of the columns and at the foundation elements that transmit the loads to the underlying soil should extend to the extreme and end with a standard hook.

7.7.4.2.8 Longitudinal bar offset

Offset bent longitudinal bars should conform to the following provisions.

a) The slope of the inclined portion of an offset bar with the axis of column should not exceed 1 in 6.

b) The portions of the bar above and below an offset should be parallel to axis of column.

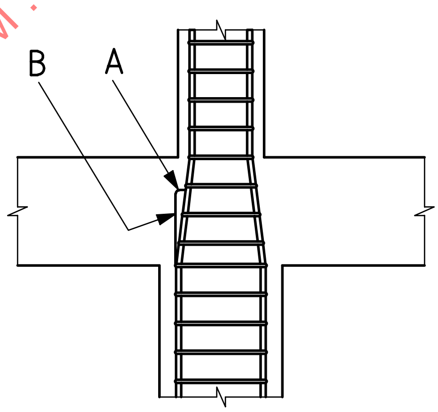
c) The horizontal support at offset bends should be provided by lateral ties or spirals.

d) The 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 the bend points.

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 by 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 be in accordance with 7.3.8.2.1.



The ratio of A/B should not exceed 1/6.

Figure 90 — Longitudinal bar offset

7.7.4.2.9 Maximum number of longitudinal bars per face of rectangular column

The maximum number of longitudinal bars in a layer should be determined taking into consideration the longitudinal and transverse reinforcement bar diameters, the appropriate concrete cover (see 7.3.4), the maximum nominal coarse aggregate size and the minimum clear spacing between bars (see 7.3.7). When these computations are not performed, it should be permitted to apply the following provisions.

a) For a column section with a considered dimension, b_c, equal to or greater than 400 mm, it should be permitted to determine the maximum number of bars in a layer in accordance with Equation (127)

$$N_b \leqslant \frac{b_c}{68} - 1 \tag{127}$$

where b_c is the column dimension, expressed in millimetres; see Table 18.

b) Three longitudinal bars should be permitted in the face of columns whose considered dimension, b_c, is less than 400 mm and equal to or greater than 300 mm; see Table 18.

Table 18 — Maximum number of longitudinal bars per face of rectangular column

Column dimension b_c mm	Maximum number of longitudinal bars
$b_c < 300$	section not permitted
$300 \leqslant b_c < 400$	3
$400 \leqslant b_c$	$\leqslant \left(\frac{b_c}{68} - 1\right)$

7.7.4.2.10 Maximum number of longitudinal bars in circular columns

The maximum number of longitudinal bars in circular columns should be determined taking into account the longitudinal and transverse reinforcement bar diameters, the appropriate concrete cover (see 7.3.4), the maximum nominal coarse aggregate size, and the minimum clear spacing between bars (see 7.3.7). When these computations are not performed, it should be permitted to determine the maximum number of bars in accordance with Equation (128):

$$N_b \leqslant \frac{h}{22} - 6 \tag{128}$$

where h is the column diameter, expressed in millimetres; see Table 19.

Table 19 — Maximum number of longitudinal bars in circular columns

Column diameter h mm	Maximum number of longitudinal bars
$h < 300$	section not permitted
$300 \leqslant h$	$\leqslant \left(\frac{h}{22} - 6\right)$