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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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## Foreword

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

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

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

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

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

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

This second edition cancels and replaces the first edition (ISO 15673:2005), which has been technically revised with the following changes.

- recent research available in concrete frame and wall buildings as a result of poor structural behaviour observed during recent earthquakes have changed the design and detailing requirements for these type of buildings in seismic prone areas;
- concrete structural design criteria has been unified in almost all countries in order to use similar, if not identical load combinations with the same load factors, as well as strength reduction factors; this is a substantial change and has been changing in recent years in order to simplify and unify design criteria for different construction materials such as timber, steel, masonry and lastly concrete;
- concrete cover requirement have been updated to most recent international building code standards.

## Introduction

This document is developed for countries that do not have existing national standards. This document should 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. This document is self-contained; therefore, actions (loads) and simplified analysis procedures are included, as well as minimum acceptable construction practice guidelines.

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

The earthquake resistance guidelines are included to account for the fact that numerous underdeveloped regions of the world lay 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.

This document contains guidelines that can be modified by the national standards body due to local design and construction requirements and practices. These guidelines that can be modified are included using [*boxed values*]. The authorities in each member country are expected to review the “boxed values” and may substitute alternative definitive values for these elements for use in the national application of the document.

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

The purpose of these guidelines is to provide a registered civil engineer or architect with sufficient information to perform the design of the structural reinforced concrete that comprises the structural framing of a low-rise building that complies with the limitations established in [6.1](#). The rules of design as set forth in the present document are simplifications of the more elaborate requirements.

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

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

# Guidelines for the simplified design of structural reinforced concrete for buildings

## 1 Scope

This document provides guidelines for the design and construction of low-rise concrete building structures of small area to be built in the less developed areas of the world.

This document is applicable to the planning, design and construction of structural reinforced concrete structures to be used in new low-rise buildings of restricted occupancy, number of stories, and area.

This document can be used as an alternative to the development of a National Concrete Building Code, or equivalent document in countries where no national design codes are available by themselves, or as an alternative to the National Concrete Building Code in countries where specifically considered and accepted by the national standard body or other appropriate regulatory organization.

## 2 Normative references

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

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

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 4354, *Wind actions on structures*

ISO 6274, *Concrete — Sieve analysis of aggregates*

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

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

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

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

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

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 10144, *Certification scheme for steel bars and wires for the reinforcement of concrete — Welded-wire reinforcement*

ISO 29581-1, *Cement — Test methods — Part 1: Analysis by wet chemistry*

### 3 Terms and definitions

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

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

- IEC Electropedia: available at <http://www.electropedia.org/>
- ISO Online browsing platform: available at <http://www.iso.org/obp>

#### 3.1 acceleration of gravity

*g*  
acceleration produced by gravity at the surface of the Earth

Note 1 to entry: For the application of these guidelines, its value can be approximated to *g* approximately [10] m/s<sup>2</sup>.

#### 3.2 admixture

material other than water, *aggregate* (3.3), or hydraulic cement, used as an ingredient of *concrete* (3.20) 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* (3.20) 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 1 to entry: For deformed bars, it is common practice to use 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 1 to entry: 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* (3.51) soil that provides adequate safety against bearing failure of the soil, or *settlement* (3.90) of the foundation of such magnitude as to impair the structure

Note 1 to entry: Its value is defined at the *working stress* (3.122) 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 elements**

portions along wall edges strengthened by longitudinal and *transverse reinforcement* (3.117)

Note 1 to entry: Boundary elements do not necessarily require an increase in thickness of the wall.

**3.11****buildings**

structures, usually enclosed by walls and a roof, constructed to provide support or shelter for an intended *occupancy* (3.76)

**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 which, when mixed with water, has hardening properties, used either in *concrete* (3.20) or by itself

**3.14****center of mass**

geometric place where all the *mass* (3.68) of the floor would be located in plan (assuming the floor diaphragm as an infinite rigid body in its own plane)

**3.15****center of rigidity**

geometric place located in plan and established assuming that the floor diaphragm is an infinite rigid body in its own plane, where applying an horizontal force, in any direction, no diaphragm rotation is presented around a vertical axis

**3.16****column**

vertical member used primarily to support axial compressive *loads* (3.66)

**3.17****collector elements**

elements that serve to transmit the inertia forces within the diaphragm to members of the *lateral-force resisting system* (3.60)

**3.18****combined footing**

*footing* (3.49) that transmits the load carried by several *columns* (3.16) or *structural concrete walls* (3.110) to the supporting soil

**3.19****compression reinforcement**

*reinforcement* (3.85) provided to resist compression stresses induced by *bending moments* (3.9) acting on the member section

**3.20****concrete**

mixture of portland cement or any other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without *admixtures* (3.2)

**3.21****concrete mix design**

choice and proportioning of the ingredients of *concrete* (3.20)

**3.22  
concrete, specified compressive strength of**

$f'_c$

compressive cylinder strength of *concrete* (3.20) used in design

Note 1 to entry: It is expressed in megapascals (MPa).

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

**3.23  
confinement hook**

*hook* (3.56) on a *stirrup* (3.105), *hoop* (3.57), or *cross-tie* (3.28) having a bend not less than 135° with a six-diameter (but not less than 75 mm) extension that engages the *longitudinal reinforcement* (3.67) and projects into the interior of the stirrup or hoop

**3.24  
confinement stirrup or tie**

closed *stirrup* (3.105), *tie* (3.115) or continuously wound spiral

Note 1 to entry: A closed stirrup or tie can be made up of several *reinforcement* (3.85) elements each having *confinement hooks* (3.23) at both ends. A continuously wound spiral should have a confinement hook at both ends.

**3.25  
contraction joint**

formed, sawed, or tooled groove in a concrete structure to create a weakened plane and regulate the location of cracking resulting from the dimensional change of different parts of the structure

**3.26  
corrosion**

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

**3.27  
cover**

<concrete> thickness of *concrete* (3.20) between the surface of any reinforcing bar and the nearest face of the concrete member

**3.28  
cross-tie**

continuous reinforcing bar having a 135° *hook* (3.56) at one end and a hook not less than 90° at least a six-diameter extension at the other end

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

**3.29  
curing**

process of keeping the *concrete* (3.20) damp for a period of time, usually several days, starting from the moment it is cast, in order for the *cement* (3.13) to be provided with enough water to harden and attain the intended strength

Note 1 to entry: Appropriate curing will greatly reduce shrinkage, increase strength of concrete, and should reduce surface cracking. Curing time will depend on temperature and relative humidity of surrounding air, the amount of wind, the direct sunlight exposure, the type of concrete mix employed, and other factors.

**3.30  
curtain wall**

walls that are part of the façade or enclosure of the building

**3.31****deformed reinforcement**

steel *reinforcement* (3.85) that has deformations in its surface to increase its bond to the *concrete* (3.20)

Note 1 to entry: The following steel reinforcement should be considered deformed reinforcement under these guidelines: deformed reinforcing bars, deformed *wire* (3.121), welded plain wire reinforcement, and welded deformed wire reinforcement conforming to the appropriate ISO standards.

**3.32****depth of member**

*h*

vertical size of a cross section of a horizontal structural element

**3.33****design load combinations**

combinations of factored *loads* (3.66) and forces

**3.34****design strength**

product of the *nominal strength* (3.74) multiplied by a *strength reduction factor*,  $\phi$  (3.107)

**3.35****development length**

length of embedded reinforcement required to develop the *design strength* (3.34) of *reinforcement* (3.85) at a critical section

**3.36****development length for a bar with a standard hook**

shortest distance between the critical section (where the strength of the bar is to be developed) and a tangent to the outer edge of the 90° or 180° *hook* (3.56)

**3.37****differential settlement**

when the *foundation* (3.51) of different parts of a structure settle different amounts

**3.38****drift**

difference between the horizontal displacements of two consecutive levels

**3.39****durability**

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

**3.40****effective depth of section**

*d*

distance measured from the extreme compression fibre to the centroid of tension *reinforcement* (3.85)

**3.41****embedment length**

length of embedded *reinforcement* (3.85) provided beyond a critical section

**3.42****essential facilities**

buildings and other structures that are intended to remain operational in the event of extreme environmental loading from wind, snow, or earthquakes

**3.43****factored loads and forces**

specified nominal *loads* (3.66) and forces multiplied by the load factors

**3.44**

**fire protection of reinforcement**

amount of concrete cover necessary for protection of the *reinforcement* (3.85) against the effects of the high temperatures produced by fire

Note 1 to entry: The concrete cover is a function of the number of hours of exposure to the fire.

**3.45**

**flange**

top or bottom part of an I-shaped or T-shaped section separated by the *web* (3.119)

**3.46**

**flexural**

pertaining to the flexure *bending moment* (3.9)

**3.47**

**flexural reinforcement**

*reinforcement* (3.85) provided to resist the tensile stresses induced by flexural moments acting on the member section

**3.48**

**floor system**

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

Note 1 to entry: It includes the *beams* (3.7) and *girders* (3.54), the *joists* (3.58) (if employed), and the *slab* (3.95) that spans between them.

**3.49**

**footing**

portion of the *foundation* (3.51) which transmits *loads* (3.66) directly to the soil

Note 1 to entry: May be the widening part of a *column* (3.16), a *structural concrete wall* (3.110) or several columns, in a *combined footing* (3.18).

**3.50**

**formwork**

temporary construction to contain *concrete* (3.20) in a plastic state while casting and setting take place, and that forms the final shape of the element as the concrete hardens

**3.51**

**foundation**

any part of the structure that serves to transmit *loads* (3.66) to the underlying soil, or to contain it

**3.52**

**foundation beam**

*beam* (3.7) that rests on the *foundation* (3.51) soil and spans between *footings* (3.49), used either to support walls or to limit *differential settlement* (3.37) of the foundation

**3.53**

**foundation mat**

continuous *slab* (3.95) laid over the ground as part of the *foundation* (3.51) and that transmits to the underlying soil the *loads* (3.66) from the structure

**3.54**

**girder**

main horizontal support *beam* (3.7), usually supporting other beams

**3.55**

**gravity loads**

*loads* (3.66) that act downward and are caused by the *acceleration of gravity, g*, (3.1) acting on the *mass* (3.68) of the elements that cause the dead and *live loads* (3.63)

**3.56****hook**

bend at the end of a reinforcing bar

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

**3.57****hoop**

closed *stirrup* (3.105), *tie* (3.115), or continuously wound spiral

Note 1 to entry: A closed stirrup or tie can be made up of several *reinforcement* (3.85) elements, each having seismic *hooks* (3.56) at both ends. A continuously wound spiral shall have a seismic hook at both ends

**3.58****joist**

T-shaped or I-shaped *beam* (3.7) used in parallel series directly supporting floor and ceiling loads (3.66), and supported in turn by larger *girders* (3.54), beams, or bearing *structural concrete walls* (3.110)

**3.59****lap splice**

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

**3.60****lateral-force resisting system**

portion of the structure composed of members proportioned to resist forces related to earthquake effects

**3.61****lightweight aggregate concrete**

*concrete* (3.20) made with coarse granular material that weighs less than the granular material used in normal weight concrete

Note 1 to entry: This type of concrete is not covered by these guidelines.

**3.62****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)

**3.63****live load**

*loads* (3.66) produced by the use and *occupancy* (3.76) of the building and do not include construction or environmental loads such as wind load, snow load, rain load, earthquake load, flood load, or dead load, without *load factors* (3.65)

**3.64****load effects**

forces and internal deformations produced in structural members by the applied *loads* (3.66)

**3.65****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

**3.66****loads**

forces or other actions that result from the *weight* (3.120) of all building materials, occupants and their possessions, environmental effects, differential movement, and restrained dimensional changes

3.67

**longitudinal reinforcement**

*reinforcement* (3.85) that is laid parallel to the longitudinal axis of the element, generally to account for flexural effects

3.68

**mass**

quantity of matter in a body

3.69

**mesh wire**

welded-wire *reinforcement* (3.85)

3.70

**modulus of elasticity**

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

3.71

**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* (3.47) in the upper part of the element section

3.72

**negative reinforcement**

in horizontal or nearly horizontal elements, the *flexural reinforcement* (3.47) required for *negative moment* (3.71) and that is placed in the upper part of the section of the element

3.73

**nominal loads**

magnitudes of the *loads* (3.66) (dead, live, soil, wind, snow, rain, flood, and earthquake), without *load factors* (3.65)

3.74

**nominal strength**

capacity of a structure or member to resist the effects of *loads* (3.66), as determined by theoretical computations using specified material strengths and dimensions, which 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

3.75

**non-structural elements**

correspond to architectural, mechanical, and electrical components and systems permanently attached to the building

3.76

**occupancy**

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

3.77

**partitions**

non-structural walls that are employed to divide spaces

Note 1 to entry: They do not support other parts of the building except themselves. When they are built in the exterior, sometimes they are referred to as *curtain walls* (3.30).

3.78

**pedestal**

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

**3.79****permanent loads**

*loads* (3.66) in which variations over time are rare or of small magnitude

Note 1 to entry: All other loads are variable loads; see also *nominal loads* (3.73).

**3.80****pile**

slender timber, *concrete* (3.20) or structural steel element embedded in the ground to support *loads* (3.66)

**3.81****plain reinforcement**

smooth surfaced steel *reinforcement* (3.85), or reinforcement that does not conform to the definition of *deformed reinforcement* (3.31)

**3.82****positive moment**

flexural moment that produces tension stresses at the lower part of the section of a horizontal, or nearly horizontal element, and that requires placing positive *flexural reinforcement* (3.47) in the lower part of the element section

**3.83****positive reinforcement**

in horizontal or nearly horizontal elements, the *flexural reinforcement* (3.47) required for *positive moment* (3.82) and that is placed in the lower part of the section of the element

**3.84****reaction**

resistance to a force or load, or upward resistance of a *support* (3.112) such as a *structural concrete wall* (3.110) or *column* (3.16) against the downward pressure of a loaded member such as a beam

**3.85****reinforcement**

steel bars, *wire* (3.121), or *mesh wire* (3.69), used for reinforcing the *concrete* (3.20) where tensile stresses are expected, due either to the applied *loads* (3.66), *differential settlement* (3.37) or to environmental effects such as variation of temperature

**3.86****required factored strength**

strength of a member or cross section required to resist factored *loads* (3.66) or related internal moments and forces

**3.87****retaining wall**

wall built to hold back earth

**3.88****selfweight**

*weight* (3.120) of the structural element, caused by the material that composes the element

**3.89****service load**

*load* (3.66), without *load factors* (3.65)

**3.90****settlement**

downward movement of the supporting soil due to construction *loads* (3.66)

**3.91**

**shear**

internal force acting tangential to the plane where it acts or perpendicular to the elements longitudinal axis

Note 1 to entry: Also called diagonal tension.

**3.92**

**shear reinforcement**

*reinforcement* (3.85) designed to resist *shear* (3.91)

**3.93**

**shores**

vertical or inclined *support* (3.112) members designed to carry the *weight* (3.120) of the *formwork* (3.50), *concrete* (3.20), and construction *loads* (3.66) above

**3.94**

**shrinkage and temperature reinforcement**

*reinforcement* (3.85) normal to *flexural reinforcement* (3.47) provided for shrinkage and temperature stresses in structural *solid slabs* (3.97) and *footings* (3.49) where flexural reinforcement extends in one direction only

**3.95**

**slab**

upper flat part of a reinforced *concrete* (3.20) floor carried by supporting *joists* (3.58), *beams* (3.7), walls or *columns* (3.16)

**3.96**

**slab on grade**

*slab* (3.95) set directly on the ground that serves either as a internal traffic surface or as part of the *foundation* (3.51)

**3.97**

**solid slab**

*slab* (3.95) of uniform thickness that does not have voids to make it lighter

**3.98**

**span length**

horizontal distance between *supports* (3.112) of a horizontal structural element, such as for a *slab* (3.95), *joist* (3.58), *beam* (3.7), or *girder* (3.54)

**3.99**

**specifications**

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

**3.100**

**specified lateral earthquake forces**

lateral forces corresponding to the appropriate distribution of the design base *shear* (3.91) force for earthquake-resistant design

**3.101**

**specified wind forces**

nominal pressure of wind to be used in design

**3.102**

**spiral reinforcement**

continuously wound *reinforcement* (3.85) in the form of a cylindrical helix

**3.103****spread footing**

isolated *footing* (3.49) that transmits the load carried by a single *column* (3.16) to the supporting soil

**3.104****stairway**

flight of steps leading from one level to another

**3.105****stirrup**

*reinforcement* (3.85) used to resist *shear* (3.91) and torsion stresses in a structural member

Note 1 to entry: Typically bars, *wires* (3.121), or welded wire reinforcement (plain or deformed) either single leg or bent into L, U, or rectangular shapes and located perpendicular to or at an angle to *longitudinal reinforcement* (3.67). The term “stirrups” is usually applied to lateral reinforcement in *girders* (3.54), *beams* (3.7), and *joists* (3.58). The term “ties” apply to those in *columns* (3.16) and walls. See also *tie* (3.115).

**3.106****story height**

vertical distance between the upper part of the *slab* (3.95) of a story and the upper part of the slab of the floor below

**3.107****strength reduction factor**

$\phi$

coefficient that accounts for deviations of the actual strength from the *nominal strength* (3.74), according to the manner and consequences of failure

Note 1 to entry: Including the probability of understrength members due to variations in material strengths and dimensions, approximations in the design equations, to reflect the degree of ductility and required reliability on the member under the *load effects* (3.64) being considered, and to reflect the importance of the element in the structure.

**3.108****stress**

intensity of force per unit area

**3.109****structural concrete**

all *concrete* (3.20) used for structural purposes including plain and reinforced concrete and prestressed concrete

**3.110****structural concrete walls**

walls proportioned to resist combinations of *shear* (3.91), moments, and axial forces

Note 1 to entry: A “shearwall” is a “structural wall.”

**3.111****structural diaphragms**

structural members, such as floor and roof *slabs* (3.95) which transmit inertial forces induced by earthquake motions

**3.112****support**

structural element that provides support to another structural element

**3.113****tank**

container for the storage of water or other fluids

**3.114**

**temporary facilities**

buildings or other structures that are to be in service for a limited time and have a limited exposure period for environmental loadings

**3.115**

**tie**

loop of reinforcing bar or *wire* (3.121) enclosing *longitudinal reinforcement* (3.67)

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

**3.116**

**tie elements**

elements which serve to transmit inertia forces and prevent separation of such building components as *footings* (3.49) and *walls* (3.118)

**3.117**

**transverse reinforcement**

*reinforcement* (3.85) located perpendicular to the longitudinal axis of the element, comprising *stirrups* (3.105), *ties* (3.115), *spiral reinforcement* (3.102), among others

**3.118**

**wall**

member, usually vertical, used to enclose or separate spaces

**3.119**

**web**

thin vertical portion of an I-shaped or T-shaped section that connects the *flanges* (3.45)

**3.120**

**weight**

vertical downward force exerted by a *mass* (3.68), when subjected to the *acceleration of gravity* (3.1)

Note 1 to entry: The weight is equal to the value of the mass multiplied by the acceleration of gravity,  $g$ .

**3.121**

**wire**

reinforcing bar of small diameter

**3.122**

**working stress**

allowable *stress* (3.108) to be used with unfactored *loads* (3.66)

**3.123**

**yield strength**

$f_y$

specified minimum yield strength or yield point of *reinforcement* (3.85)

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

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

## 4 Symbols and abbreviated terms

### 4.1 Symbols

Symbol	Explanation	Unit
$a$	depth of equivalent uniform compressive stress block	mm
$a_m$	acceleration magnifying factor	
$a_x$	acceleration at floor level	
$A_a$	effective peak horizontal acceleration coefficient	
$A_b$	area of an individual reinforcement bar or wire	mm <sup>2</sup>
$A_c$	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,	mm <sup>2</sup>
$A_g$	gross area of section of element	mm <sup>2</sup>
$A_j$	effective cross-sectional area within a joint for shear evaluation or area of additional hanger reinforcement, where beams are supported by girders or other beams	mm <sup>2</sup>
$A_s$	area of longitudinal tension reinforcement	mm <sup>2</sup>
$A'_s$	area of longitudinal compression reinforcement	mm <sup>2</sup>
$A_{s,min}$	minimum area of longitudinal tension reinforcement	mm <sup>2</sup>
$A_{se}$	total extreme steel area in a column or structural concrete wall for computation of the balanced moment strength	mm <sup>2</sup>
$A_{si}$	total side steel area in a column or structural concrete wall for computation of the balanced moment strength	mm <sup>2</sup>
$A_{st}$	total area of longitudinal reinforcement	mm <sup>2</sup>
$A_{su}$	wind exposed surface area	m <sup>2</sup>
$A_v$	area of shear reinforcement within a distance $s$	mm <sup>2</sup>
$b$	width of compression face of member, or width of the section of the member	mm
$b_c$	width of the column section, or largest plan dimension of capital or drop panel, for punching shear evaluation	mm
$b_{col}$	dimension of column section in the direction perpendicular to the girder span	m
$b_f$	effective width of the compression flange in a T-shaped section	mm
$b_w$	web width in a T-shaped section, or web width of girders, beams or joists, or thickness of the web in a structural concrete wall	mm
$b_0$	perimeter of critical section for punching shear in slabs	mm
$d$	effective depth, should be taken as the distance from extreme compression fibre to centroid of tension reinforcement	mm
$d'$	distance from extreme compression fibre to centroid of compression reinforcement	mm
$d_b$	nominal diameter of reinforcing bar or wire	mm
$d_c$	distance from extreme tension fibre to centroid of tension reinforcement or diameter of the confined core of column with spiral reinforcement	mm
$d_p$	diameter of pile, at footing base	mm
$D$	dead loads, or related internal moments and forces	
$E$	load effects of earthquake, or related internal moments and forces	
$E_c$	modulus of elasticity of concrete	MPa
$f'_c$	specified compressive strength of concrete	MPa
$\sqrt{f'_c}$	positive square root of specified compressive strength of concrete	The result should have units of MPa
$f_{cd}$	compressive strength of concrete reduced by the material factor	MPa

Symbol	Explanation	Unit
$f_{cu}$	extreme fibre factored compressive stress at edges of structural walls	MPa
$f_y$	specified yield strength of reinforcement	MPa
$f_{yd}$	yield strength of reinforcement reduced by the material factor	MPa
$f_{ypr}$	probable specified maximum strength of reinforcement	MPa ( $f_{ypr} = 1,25 \cdot f_y$ )
$f_{ys}$	specified yield strength of transverse or spiral reinforcement	MPa
$f_{ysd}$	yield strength of transverse or spiral reinforcement reduced by the material factor	MPa
$F$	loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces	
$F_a$	site soil coefficient	
$F_i, F_x$	design wind or seismic force applied at level $i$ or $x$ , respectively	kN
$F_{iw}, F_{xu}$	factored design lateral force applied to the wall at level $i$ or $x$ , respectively	N
$F_s$	non-structural walls seismic force	N
$h$	depth or thickness of structural element or overall thickness of member	mm
$h_b$	vertical distance measured from the bottom of the supporting girder to the bottom of the supported beam	mm
$h_{col}$	dimension of column section in the direction parallel to the girder span	m
$h_c$	height of the column section	mm
$h_f$	slab thickness	mm
$h_i, h_x$	height above the base to level $i$ or $x$ , respectively	m
$h_n$	clear vertical distance between lateral supports of columns and walls	mm
$h_{pi}$	story height of floor $i$ , measured from floor finish of the story being evaluated to floor finish of the story immediately below	mm
$h_s$	total height of the supporting girder	mm
$h_w$	height of entire structural concrete wall from base to top	mm
$H$	loads due to the weight and pressure of soil, water in soil, or other materials, or related internal moments and forces	
$I_c$	moment of inertia of the column section	m <sup>4</sup>
$l$	span of structural element or length of span measured centre-to-centre of beams or other supports	
$l_a$	length of clear span in the short direction of two-way slabs, measured face-to-face of beams or other supports	m
$l_b$	length of clear span in the long direction of two-way slabs, measured face-to-face of beams or other supports	m
$l_d$	development length for reinforcing bar	mm
$l_m$	length of clear span in the direction that moments, shears and reinforcement are being determined, measured face-to-face of supports	m
$l_j$	clear spacing between joists	m
$l_n$	length of clear span in the long direction of two-way construction, measured face-to-face of supports in slabs without beams, and face-to-face of beams or other supports in other cases or length of clear span, measured face-to-face of supports in slabs without beams, and face-to-face of beams or other supports in other cases	mm
$l_w$	horizontal length of structural concrete wall	mm
$l_o$	column confinement length	
$L$	live loads, or related internal moments and forces	
$L_r$	sloping roof live load, or related internal moments and forces	
$m_w$	mass of the nonstructural wall	kg

Symbol	Explanation	Unit
$M_{bn}$	nominal flexural moment strength at section at balanced conditions	N mm
$M_{br}$	flexural moment strength at section at balanced conditions	N mm
$M_{iu}, M_{xu}$	factored story moment caused by lateral loads at story $i$ or $x$ , respectively	N mm
$M_n$	nominal flexural moment strength at section	N mm
$M_r$	flexural moment strength at section	N mm
$M_{pr}$	probable flexural moment strength of the element at the joint face computed using $f_{ypr}$ and $\phi = 1$	N · m
$M_u$	factored flexural moment at section	N · m
$M_u^-$	factored negative flexural moment at section	N · m
$M_u^+$	factored positive flexural moment at section	N · m
$\sum M_c$	sum of lowest flexural strengths ( $\phi \cdot M_n$ ) of columns framing into a joint	N · m
$\sum M_g$	sum of flexural strengths ( $\phi \cdot M_n$ ) of girders framing into a joint	N · m
$\Delta M_u$	factored unbalanced moment at a column-girder joint or factored unbalanced moment at a wall-girder joint	N · m
$P_d$	non factored dead load axial force at section or non factored concentrated dead load applied directly to the element	N
$P_{bn}$	nominal compression axial load strength at section at balanced conditions	N
$P_{br}$	compression axial load strength at section at balanced conditions	N
$P_{cu}$	factored compression load on wall boundary element, including earthquake effects	N
$P_l$	non factored live load axial force at section or non factored concentrated live load applied directly to the element	N
$P_n$	nominal axial load strength at section	N
$P_{n(max)}$	maximum compression nominal axial load strength at section	N
$P_{tn}$	axial tension strength at section	N
$P_{tu}$	factored tension force on wall boundary element, including earthquake effects	
$P_u$	factored axial load at section or factored concentrated design load applied directly to the element or factored axial load on column or wall	N
$P_{0n}$	axial compressive strength at section	N
$\sum P_u$	sum of all factored concentrated design loads within the span	N
$q_d$	non-factored dead load per unit area	N/m <sup>2</sup>
$q_l$	non-factored live load per unit area	N/m <sup>2</sup>
$q_u$	factored load per unit area	N/m <sup>2</sup>
$r_u$	factored uniformly distributed reaction from the slab on the supporting girder, beam or structural concrete wall	N/m
$R_a$	rain load, or related internal moments and forces	
$R_m$	nonstructural wall seismic force reduction factor	
$R_u$	total factored concentrated reaction from a supported structural element	N
$\sum R_u$	sum of all factored reactions from supported structural elements at the same story	N
$s$	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	mm
$S$	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	N · mm

Symbol	Explanation	Unit
$U$	required factored strength to resist factored loads or related internal moments and forces	
$V_c$	contribution of the concrete to the nominal shear strength at section	N
$\Delta V_e$	factored design shear force from the development of the probable flexural capacity of the element at the faces of the joints	N
$V_{iu}, V_{xu}$	factored story shear caused by lateral loads at story $i$ or $x$ , respectively	N
$V_n$	nominal shear strength at section	N
$V_s$	contribution of the transverse reinforcement to the nominal shear strength at section	N
$V_u$	factored shear force at section	N
$w_d$	non-factored uniformly distributed dead load per unit element length applied directly to the element	N/m
$w_l$	non-factored uniformly distributed live load per unit element length applied directly to the element	N/m
$w_u$	factored uniformly distributed design load per unit element length applied directly to the element	N/m
$W$	wind loads, or related internal moments and forces	
$W_u$	total factored uniformly distributed design load per unit element length	kN/m
$\alpha_a$	fraction of the load that travels in the short direction in two-way slabs-on-girders	
$\alpha_b$	fraction of the load that travels in the long direction in two-way slabs-on-girders	
$\alpha_s$	constant used to compute nominal punching shear strength in slabs	
$\beta$	ratio of clear spans in long to short direction of two-way slabs	
$\phi$	strength reduction factor	
$\rho$	ratio of longitudinal tension reinforcement	$\frac{A_s}{b \cdot d}$
$\rho'$	ratio of longitudinal compression reinforcement	
$\rho_h$	ratio of horizontal reinforcement in structural concrete walls	
$\rho_{max}$	maximum permissible ratio of longitudinal flexural tension reinforcement	
$\rho_{min}$	minimum permissible ratio of longitudinal flexural tension reinforcement	
$\rho_s$	ratio of spiral reinforcement	
$\rho_t$	ratio of total longitudinal reinforcement area to gross concrete section area	$\frac{A_{st}}{b \cdot d}$
$\rho_v$	ratio of vertical reinforcement in structural concrete walls	

#### 4.2 Abbreviated terms

col.	column
deg.	degrees
max.	maximum
min.	minimum
No.	number

## 5 Design and construction procedure

### 5.1 Procedure

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

#### 5.1.1 Step A

Definition of the layout in plan and height of the structure, following the guidelines of [Clause 7](#). Verification that the limitations of [6.1](#) are met.

#### 5.1.2 Step B

Calculation of all gravity loads that act on the structure using the guidelines of [Clause 8](#), excluding the selfweight of the structural elements.

#### 5.1.3 Step C

Definition of an appropriate floor system, depending on the span lengths and the magnitude of the gravity loads, according to the guidelines of [Clause 10](#).

#### 5.1.4 Step D

Trial dimensions for the slab of the floor system. Calculation of the selfweight of the system, and design of the elements that comprise it, correcting the dimension as required by the strength and serviceability limit states, complying with the guidelines of [Clause 11](#) for slab systems with beams.

#### 5.1.5 Step E

Trial dimensions for the beams and girders. Calculation of their selfweight. Flexural and shear design of the beams and girders, correcting the dimension as required by the strength and serviceability limit states, complying with the guidelines of [Clause 12](#).

#### 5.1.6 Step F

Trial dimensions for the columns. Calculation of their selfweight. Column slenderness verification and design for combination of axial load and moment, and shear; correcting the dimension as required by the strength and serviceability limit states, complying with the guidelines of [Clause 13](#).

#### 5.1.7 Step G

If lateral loads such as earthquake, wind, or lateral earth pressure exist, their magnitude is established using the guidelines of [Clause 8](#); 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, using the guidelines of [Clause 16](#) for earthquake forces, the influence of their selfweight is evaluated, and flexure and shear design of the structural concrete walls is performed, complying with the guidelines of [Clause 14](#).

#### 5.1.9 Step I

The loads at the foundation level are determined and a definition of the foundation system is performed employing the guidelines of [Clause 15](#). The structural elements of the foundation are designed.

5.1.10 Step J

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.

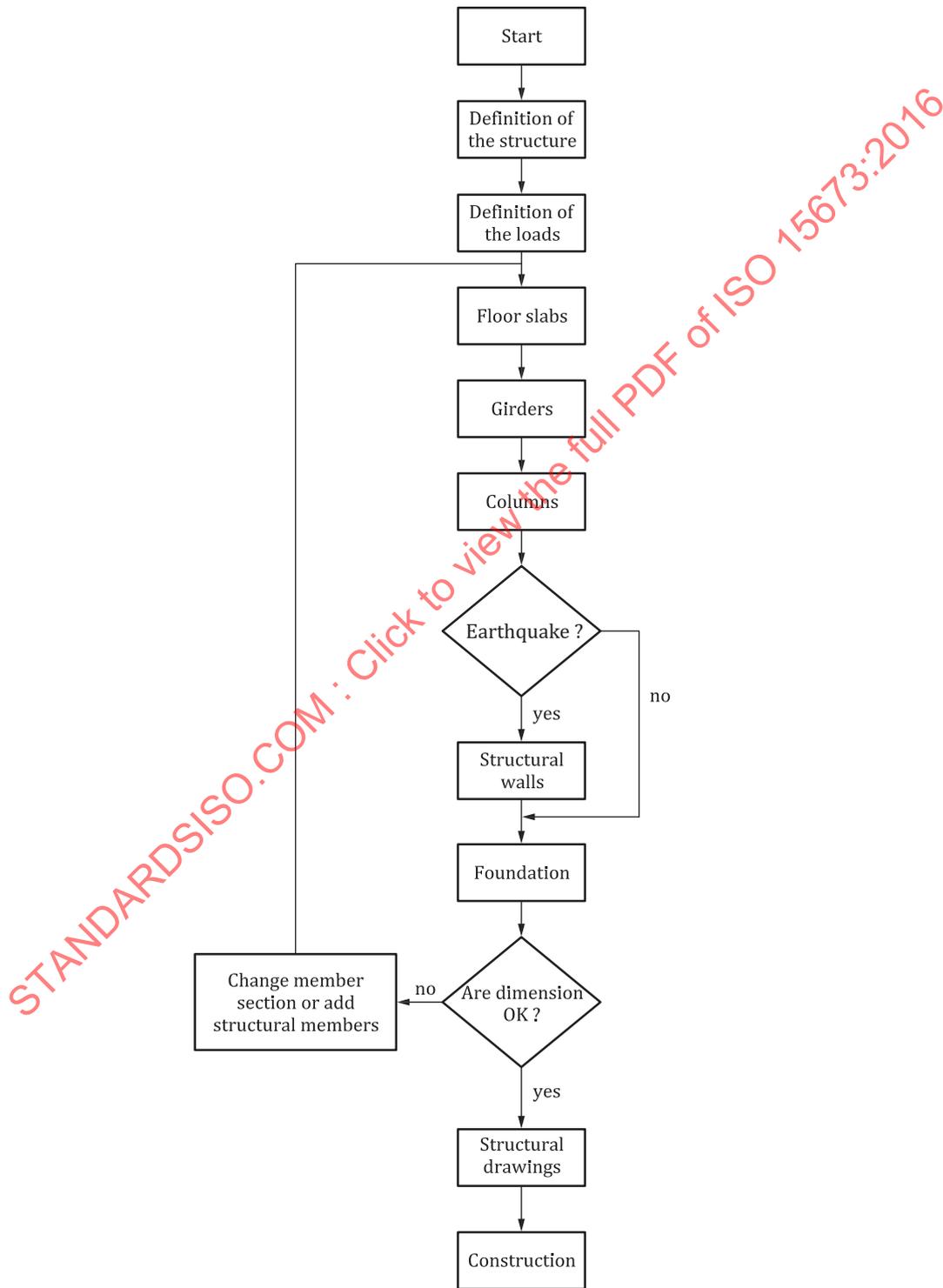


Figure 1 — Design procedure under the standard

## 5.2 Design documentation

### 5.2.1 General

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

### 5.2.2 Calculation memoir

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

- a) the general structural requirements of the project, as required by [Clause 7](#);
- b) a description of the structural system employed;
- c) loads employed;
- d) grade, strength and fabrication standards for all structural materials;
- e) presentation of all design computations;
- f) sketches of the reinforcement layout for all structural elements.

### 5.2.3 Geotechnical report

The geotechnical report should record, as a minimum, the soil investigation performed, the definition of the allowable bearing capacity of the bearing soil, the lateral soil pressures required for design of any soil retaining structure, and all other information required in [Clause 15](#).

### 5.2.4 Structural drawings

All the drawings required for construction of the structure of the building.

### 5.2.5 Specifications

The construction specifications required.

## 6 General guides

### 6.1 Limitations

These guidelines should be employed only when the building being designed complies with all the limitations set forth from [6.1.1](#) to [6.1.10](#).

#### 6.1.1 Occupancy

##### 6.1.1.1 Permitted uses and occupancies

The intended use of the building being designed should be permitted for the occupancy groups and subgroup as presented 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 capacity less than 100 people and not having a stage	[ YES ]
Group B – Business	B	Building for use as offices, professional services, containing eating and drinking establishments with less 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 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 centers 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 guidelines 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 using these guidelines should be [5]. 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<sup>2</sup>.

### 6.1.4 Maximum story height

The maximum story height, measured from finished floor to finished floor 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 should be approximately equal, with the larger of two adjacent spans not 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 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 these guidelines.

### 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 make 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 centre of mass and centre of rigidity shall be kept small to reduce the risk of global torsion of the structure.

## 6.2 Limit states

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

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

- structural integrity limit state;
- lateral load story drift limit state;
- durability limit state;
- fire limit state;
- 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 stipulated in these guidelines. Alternatively to the use of strength reduction factors, material factors may be used as described in [Annex A](#).

The basic requirement for ultimate limit state should be as given in [Formula \(1\)](#):

$$\text{Strength} \geq \text{Load effects} \tag{1}$$

To allow for the possibility that the actual strength may be less than computed and the load effects may be larger than computed, strength reduction factors,  $\phi$ , less than 1,0, and load factors,  $\gamma$ , generally greater than 1,0, should be employed, as given in [Formula \(2\)](#):

$$\phi \cdot R_n \geq \gamma_1 \cdot S_1 + \gamma_2 \cdot S_2 + \dots \tag{2}$$

where

$R_n$  is the nominal strength;

$S$  is the load effects based on the nominal loads prescribed by these guidelines.

Therefore, the ultimate limit state design format requires that:

$$\text{Design strength} \geq \text{Required factored strength} \tag{3}$$

or

$$\phi (\text{Nominal strength}) \geq U \tag{4}$$

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

### 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 described in [8.1.1](#).

### 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 these guidelines for each particular force effect in each of the element types at the critical sections defined by these guidelines, multiplied by the following strength reduction factors  $\phi$ .

- |  |                 |
|--|-----------------|
| a) Flexure, without axial load                           | $\phi = [0,90]$ |
| b) Axial tension and axial tension with flexure          | $\phi = [0,90]$ |
| c) Axial compression and axial compression with flexure: |                 |
| 1) columns with ties and structural concrete walls       | $\phi = [0,65]$ |
| 2) columns with spiral reinforcement                     | $\phi = [0,70]$ |
| d) Shear and torsion                                     | $\phi = [0,75]$ |
| e) Bearing of concrete                                   | $\phi = [0,65]$ |

### 6.4 Serviceability limit state design format

Serviceability limit states correspond to conditions beyond which specified performance requirements for the structure, or the structural elements, are no longer met. The compliance with the serviceability limit state under these guidelines should be obtained indirectly through 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 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 guides

### 7.1 Structural systems and layout

The definition of the structure comprises the the steps shown in [Figure 2](#).

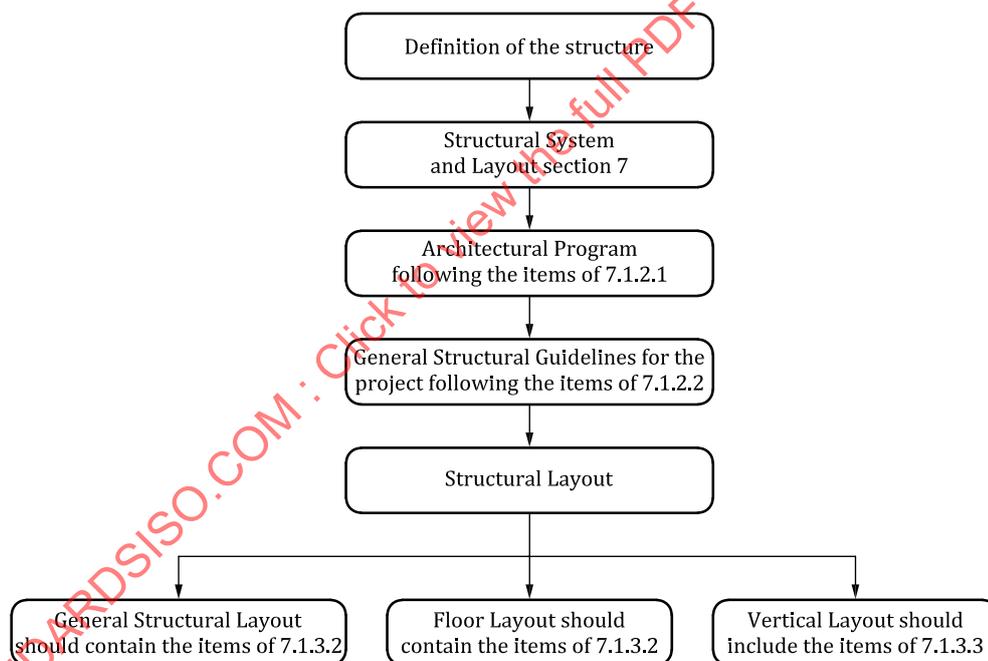


Figure 2 — Definition of the structure

#### 7.1.1 Description of the components of the structure

##### 7.1.1.1 General

For the purposes of these guidelines, the building structure should be divided in the following components.

##### 7.1.1.2 Floor system

The floor system consists of the structural elements that comprise the floor of a story in a building. In [Clause 10](#), the different types of floor systems are covered by these guidelines. 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.

### **7.1.1.3 Vertical supporting elements**

The vertical supporting elements that hold up the floor system at each story, and carry the accumulated gravity loads all the way down to the foundation of the structure. Under these guidelines, they can be either columns or structural concrete walls.

### **7.1.1.4 Foundation**

The foundation comprises all structural elements that serve to transmit loads from the structure to the underlying supporting soil, or are in contact with the soil, or serve to contain it. This includes 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 these guidelines and are not covered herein.

### **7.1.1.5 Lateral load resisting system**

The lateral load resisting system comprises the structural elements that, acting together, 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 carry them down to the foundation, and through the foundation to the underlying soil. Under the requirements of these guidelines, the main vertical elements of the lateral load resisting system should be structural concrete walls.

### **7.1.1.6 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 programme**

### **7.1.2.1 Architectural programme**

A general architectural programme of the building should be coordinated with the structural designer before actual structural design begins. The general architectural programme 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 internal spaces 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 guides for the project

Based in the general architectural programme information, the structural designer should define the general structural input information for the structure being designed under these guidelines. These general structural guides should include, at least, the following items:

- a) intended use of the building;
- b) nominal loads related to the occupation 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 protection 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, and 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 salts;
- 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

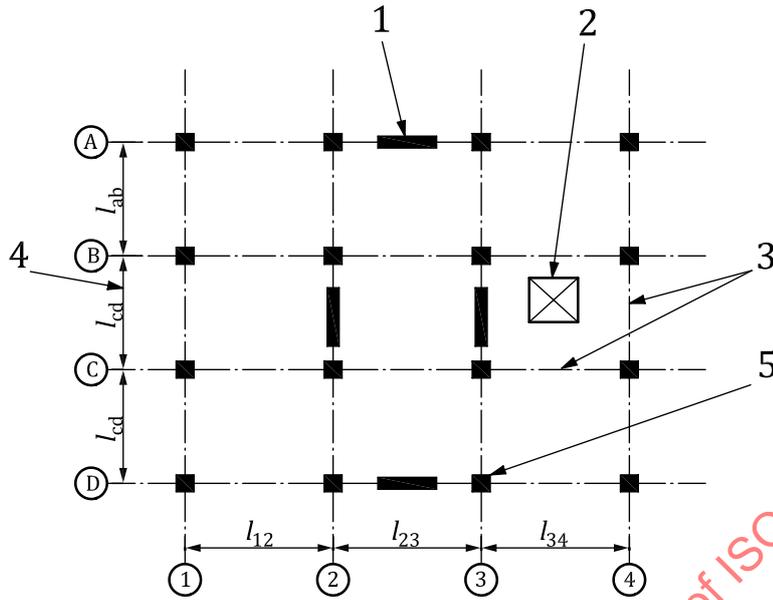
#### 7.1.3.1 General structural layout

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 3](#).

The general structural layout in plan should include the following.

- a) A dimensioned grid of axis, or centrelines, in both principal directions in plan.
- b) These axis should intersect at the location of the vertical supporting elements (columns and structural concrete walls).
- c) Location in plan of all vertical supporting elements, columns and structural concrete walls. These vertical supporting element 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.
- d) Location of all duct, shafts, elevators, and stairways, that are continuous from floor to floor.
- e) Horizontal distance between centrelines,  $l$ , which corresponds to the centre-to-centre span lengths of the floor system.

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



**Key**

- 1 wall
- 2 shaft
- 3 centrelines grid
- 4 span length
- 5 column

**Figure 3 — 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 4](#)). This layout should contain the following:

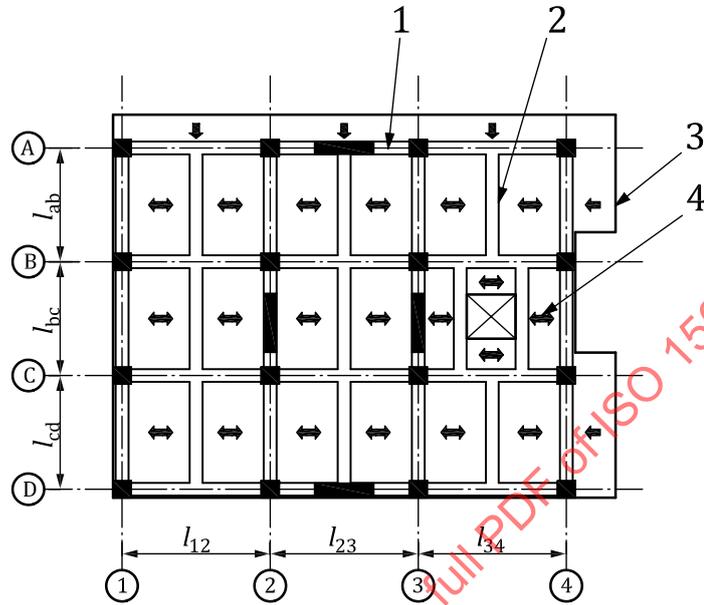
- a) the superposition of the floor perimeter on the general grid of axis;
- b) girder and beam location, or column and middle strips for slab-column systems;
- c) additional information including all substantial architectural openings in the floor;
- d) an approximate load path from all floor areas to the supporting beams and girders.

**7.1.3.3 Vertical layout**

The structural designer should define a general structural vertical layout (see [Figure 5](#)). This vertical layout should include all relevant information in elevation of the structure, including the following:

- a) number of stories;
- b) for all floors, the story height, defined as the vertical distance from finished floor to finished floor 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;

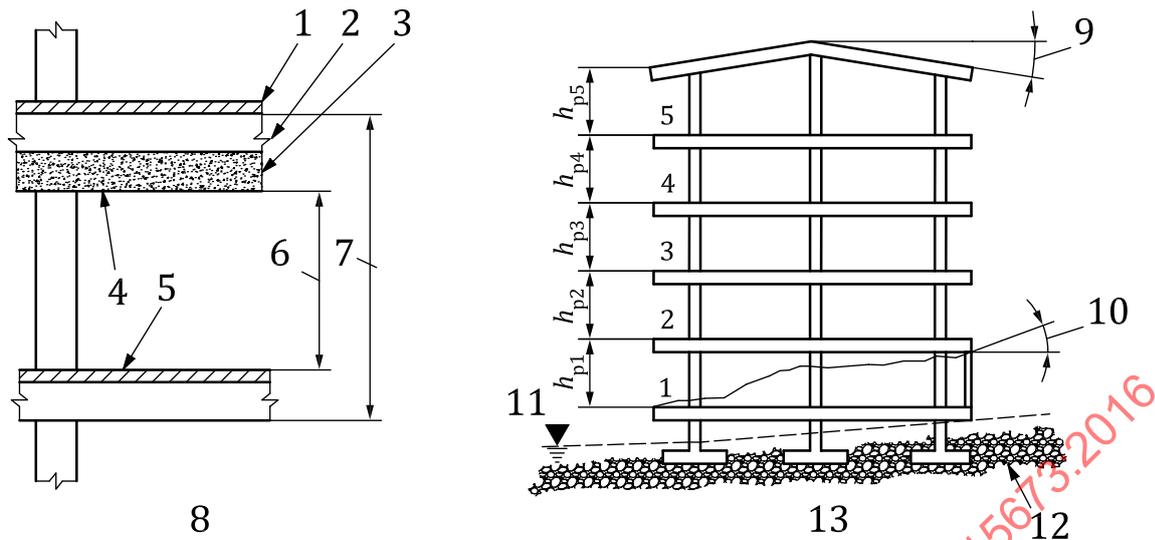
- e) vertical space necessary to accommodate horizontal/vertical service elements for power, water supply and drainage, heating, ventilation, and air conditioning;
- f) slope of the terrain, and its relationship to the ground floor or basement, if any;
- g) supporting soil stratum depth, and water table depth.



**Key**

- 1 girder
- 2 beam
- 3 structural perimeter
- 4 load path

**Figure 4 — Typical floor structural layout**



**Key**

- |   |                                  |    |                       |
|---|----------------------------------|----|-----------------------|
| 1 | floor finish                     | 8  | floor section details |
| 2 | slab                             | 9  | max. slope 15°        |
| 3 | service elements                 | 10 | max. slope 30°        |
| 4 | ceiling                          | 11 | water table           |
| 5 | floor finish                     | 12 | bearing soil stratum  |
| 6 | architectural vertical clearance | 13 | building elevation    |
| 7 | $h_{pi}$ story height            |    |                       |

**Figure 5 — Vertical layout of the building**

**7.1.4 Feasibility under the guidelines**

Based on the layout information, the structural designer should verify the feasibility of performing the structural design under the guidelines. The compliance with the following limitations should be verified.

- The use of the building should be within the accepted occupancies of 6.1.1, and if of mixed use, all types of intended occupancies should be within those permitted.
- The number of stories should not exceed the maximum permitted, given in 6.1.2.
- The area of the largest floor should not exceed the maximum permitted area of 6.1.3.
- The story height of the tallest story, measured from finish-to-finish, should not exceed the maximum permissible story height given in 6.1.4.
- The span lengths should be within the maximum span length prescribed in 6.1.5.
- The difference between adjacent spans should not exceed the limits of 6.1.6.
- The number of spans in both directions, and in all floors, should not be less than two, as required in 6.1.7, or within the exceptions stated there.
- No girder, beam or slab cantilever, should exceed the limits of 6.1.8.
- No sloping girder, beam, joist, or slab should have a slope greater than the maximum permitted value of 6.1.9.

- j) The slope of the terrain, at the building site, should not exceed the maximum prescribed in [6.1.10](#).

## 8 Actions (loads)

### 8.1 General

This subclause provides minimum load requirements for the design of buildings under these guidelines. Loads and the appropriate load combinations, should be used together.

#### 8.1.1 Load factors and load combinations

The following load factors and combinations should be employed to obtain the required factored strength of the structural member or element,  $U$ , as stated in [6.3.1](#). In the following load combinations set forth to obtain the required factored strength  $U$ , the symbol  $\pm$  in 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  $U$ .

##### 8.1.1.1 Dead and live load

Required factored strength,  $U$ , to resist dead load,  $D$ , and live load,  $L$ , should be at least equal to the greater of

$$U = [1, 4] \cdot D \quad (5)$$

$$U = [1, 2] \cdot D + [1, 6] \cdot L \quad (6)$$

##### 8.1.1.2 Rain load, snow load, and sloping roof live load

If resistance to structural effects for a specified rain load,  $R_a$ , snow load,  $S$ , or sloping roof live load,  $L_r$ , are required to be included by the requirements of the present guidelines, the following combinations of  $D$ ,  $L$ , and ( $R_a$ ,  $S$ , or  $L_r$ ) should be investigated to determine the greatest required factored strength  $U$  as given in [Formula \(7\)](#) and [Formula \(8\)](#):

$$U = [1, 2] \cdot D + [1, 6] \cdot L + [0, 5] \cdot (R_a \text{ or } S \text{ or } L_r) \quad (7)$$

$$U = [1, 2] \cdot D + [1, 0] \cdot L + [1, 6] \cdot (R_a \text{ or } S \text{ or } L_r) \quad (8)$$

but for any combination of  $D$ ,  $L$ , and ( $R_a$ ,  $S$ , or  $L_r$ ), the required factored strength  $U$  should not be less than the value obtained employing [Formula \(5\)](#) and [Formula \(6\)](#).

##### 8.1.1.3 Wind

If resistance to structural effects for a specified wind load,  $W$ , is required to be included by the requirements of the present guidelines, the following combinations of  $D$ ,  $L$ ,  $R_a$ ,  $S$ ,  $L_r$  and  $W$  should be investigated to determine the greatest required factored strength  $U$  as given in [Formula \(9a\)](#) and [Formula \(9b\)](#):

$$U = [1, 2] \cdot D + [1, 0] \cdot L \pm [1, 6] \cdot W + [0, 5] \cdot (R_a \text{ or } S \text{ or } L_r) \quad (9a)$$

$$U = [1, 2] \cdot D + [0, 8] \cdot W + [1, 6] \cdot (R_a \text{ or } S \text{ or } L_r) \quad (9b)$$

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

$$U = [0, 9] \cdot D \pm [1, 6] \cdot W \quad (10)$$

but for any combination of  $D$ ,  $L$ , and  $W$ , the required factored strength  $U$  should not be less than the value obtained employing [Formula \(5\)](#) and [Formula \(6\)](#).

#### 8.1.1.4 Earthquake forces

If resistance to specified earthquake forces,  $E$ , is required to be included by the requirements of the present guidelines, the following combinations of  $D$ ,  $L$ , and  $E$  should be investigated to determine the greatest required factored strength  $U$ , as given in [Formula \(11\)](#):

$$U = [1, 2] \cdot D + [1, 0] \cdot L \pm [1, 0] \cdot E + [0, 2] \cdot S \quad (11)$$

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

$$U = [0, 9] \cdot D \pm [1, 0] \cdot E \quad (12)$$

but for any combination of  $D$ ,  $L$ , and  $E$ , the required factored strength  $U$  should not be less than the value obtained employing [Formula \(5\)](#) and [Formula \(6\)](#).

#### 8.1.1.5 Earth pressure

If resistance to earth pressure,  $H$ , is required by the design procedure of these guidelines, the required factored strength  $U$  should be at least equal to

$$U = [1, 2] \cdot D + [1, 6] \cdot L + [1, 6] \cdot H \quad (13)$$

except that where  $D$  or  $L$  reduce the effect of  $H$ , the combination given in [Formula \(14\)](#) should be employed

$$U = [0, 9] \cdot D + [1, 6] \cdot H \quad (14)$$

For any combination of  $D$ ,  $L$  and  $H$ , the required factored strength  $U$  should not be less than the value obtained employing [Formula \(5\)](#) and [Formula \(6\)](#). When the building structure as a whole should resist permanent uncompensated horizontal loads due to lateral soil pressure,  $[1, 6] \cdot H$  should be added to the right side of [Formula \(10\)](#) and [Formula \(12\)](#).

#### 8.1.1.6 Weight and pressure of fluids

If resistance to loadings due to weight and pressure of fluids with well-defined densities and controllable maximum heights,  $F$ , is required by the design procedure of these guidelines,  $[1, 4] \cdot F$  should be added to the right side of [Formula \(5\)](#) and  $[1, 2] F$  should be added to the right side of [Formula \(6\)](#).

#### 8.1.1.7 Other effects

Where structural effects  $T$  from differential settlement, shrinkage, or temperature change are significant in design, the design should not be performed using this document and the appropriate standard of each country should be employed.

### 8.1.2 Mass of materials

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

### 8.1.3 Dead loads

Dead loads consist of the weight 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 purposes of design, the actual weights of materials and constructions should be used. In determining dead loads for purposes of design, the weight of fixed service equipment, such as plumbing stacks and risers, electrical feeders, and heating, ventilating, and air conditioning systems, should be included. See [Table 2](#).

**Table 2 — Minimum density,  $\rho$ , for evaluation of dead loads from materials**

Material	Density, $\rho$ (kg/m <sup>3</sup> )	Material	Density, $\rho$ (kg/m <sup>3</sup> )
Aluminium	2 702	Iron	
Bituminous products		Cast	7 151
Asphalt and tar	1 287	Wrought	7 628
Gasoline	667	Lead	11 283
Graphite	2 145	Lime	
Paraffin	890	Hydrated, loose	509
Petroleum	842	Hydrated, compacted	715
Brass	8 359	Masonry, brick (solid portion)	1 828
Bronze	8 772	Masonry, concrete (solid portion)	1 986
Cement, Portland, loose	1 430	Masonry, grout	2 225
Ceramic tile	2 384	Masonry, stone	2 574
Charcoal	191	Mortar cement or lime	2 066
Cinder fill	906	Particleboard	715
Coal, piled	795	Plywood	572
Concrete, plain	2 288	Sand	
Concrete, reinforced	2 384	Clean and dry	1 430
Copper	8 836	River dry	1 685
Cork, compressed	222	Steel	7 755
Earth		Stone	
Clay, dry	1 001	Basalt, granite, gneiss	1 526
Clay, damp	1 784	Limestone, marble, quartz	1 510
Clay and gravel, dry	1 589	Sandstone	1 303
Silt, moist, packed	1 526	Shale	1 462
Silt, moist, loose	1 240	Terra cotta	
Sand and gravel, dry, loose	1 589	Voids filled	1 907
Sand and gravel, dry packed	1 748	Voind unfillles	1 144
Sand and gravel, wet	1 907	Tin	7 294
Glass	2 543	Water	
Gravel, dry	1 653	Fresh	985

Table 2 (continued)

Gypsum, loose	1 112	Sea	1 017
Gypsum, wallboard	795	Wood, seasoned	445
Ice	906	Zinc, rolled sheet	7 135

#### 8.1.4 Live loads

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

#### 8.1.5 Specified snow load

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

#### 8.1.6 Specified wind forces

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

#### 8.1.7 Specified earthquake forces

##### 8.1.7.1 General

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

The requirements of the corresponding national standard should be met when calculating the mass of building materials. When no national standard is available, the requirements of ISO 9194 may be used.

For buildings which may be designed under these guidelines, an equivalent lateral force applied directly to each story may be employed to represent the dynamic response of the structure to the ground acceleration.

##### 8.1.7.2 Seismic hazard

A level of seismic hazard should be defined for the building in terms of the intensity of the effective peak ground horizontal acceleration in rock at the building site. The peak rock acceleration is calculated as the median spectral acceleration for one degree of freedom systems, with short periods of structural vibration, i.e. periods not exceeding 0,15 s, denoted as  $A_a$ , and usually expressed as a fraction of the acceleration of gravity,  $g$  (acceleration of gravity may be taken as 10 m/s<sup>2</sup>).

For the purpose of the scope of these guidelines, the values for  $A_a$  should be taken from the corresponding national standard having jurisdiction over the site of the considered existing structure. When the national code defines the maximum seismic ground motion for each considered site based on spectral response accelerations at 5 % of critical damping,  $S_S$ ,  $A_a$  may be estimated as the value of  $S_S$  for a period of 0,15 s, divided by 375 ( $A_a = S_S/375$ ). When the national code defines the maximum seismic ground motion for each considered site based on a seismic zone factor  $Z$ , the value of  $A_a$  should be taken equal to  $Z$ . When no national code exists for the site of the building being considered,  $A_a$  may be estimated from the seismic hazard maps shown in [Figure 6](#).

### 8.1.7.3 No seismic hazard zones

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

### 8.1.7.4 Low seismic hazard zones

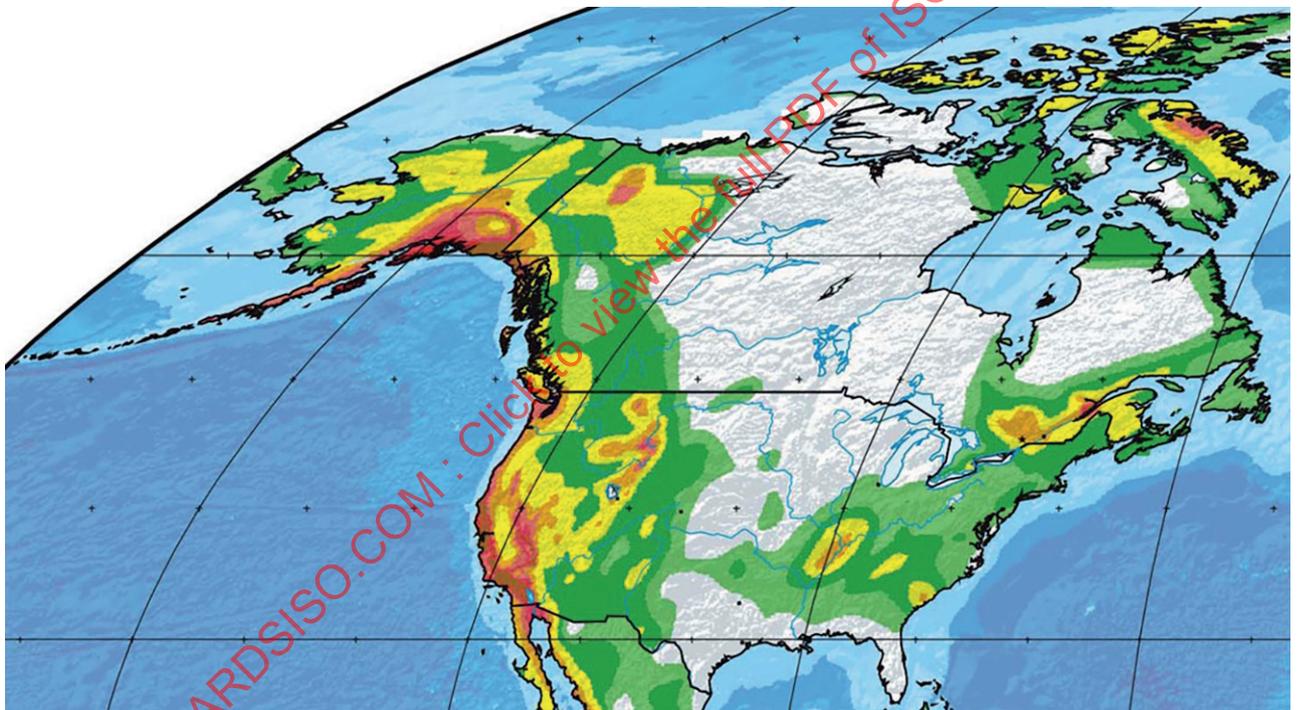
A zone where the value of  $A_a$  is estimated as more than [0,05] but less or equal to [0,10], may be deemed as a **low seismic hazard** zone.

### 8.1.7.5 Intermediate seismic hazard zones

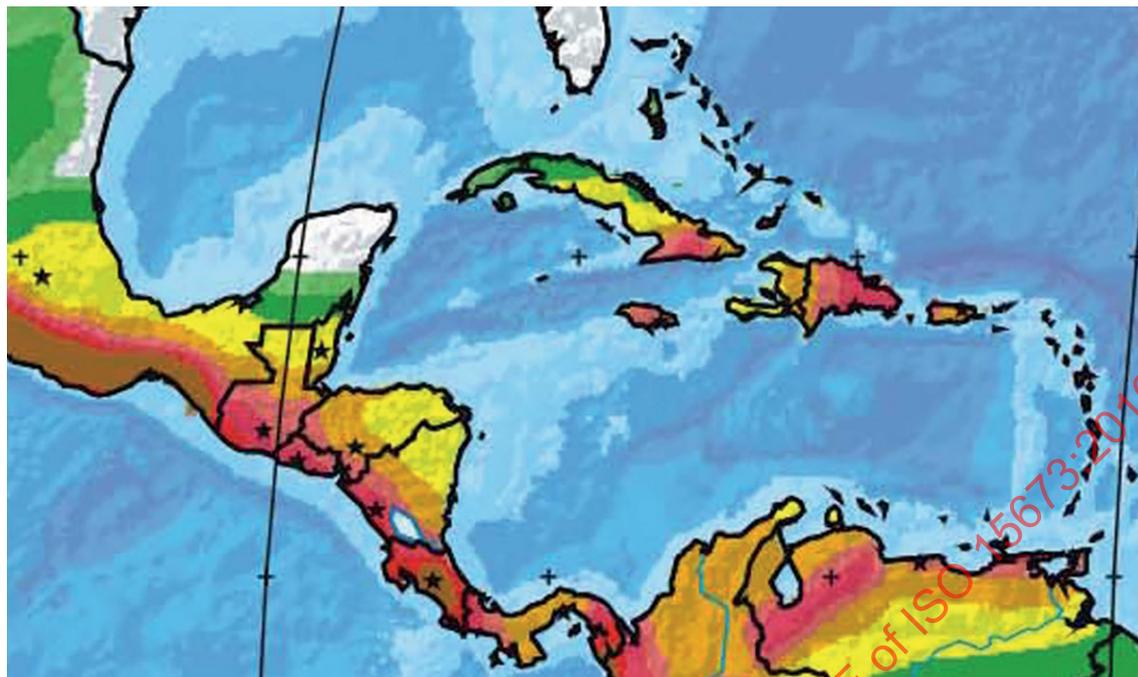
A zone where the value of  $A_a$  is estimated as more than [0,1] but less or equal to [0,20], may be deemed as an **intermediate seismic hazard** zone.

### 8.1.7.6 High seismic hazard zones

A zone where the estimated value of  $A_a$  exceeds [0,20] may be deemed as a **high seismic hazard** zone.

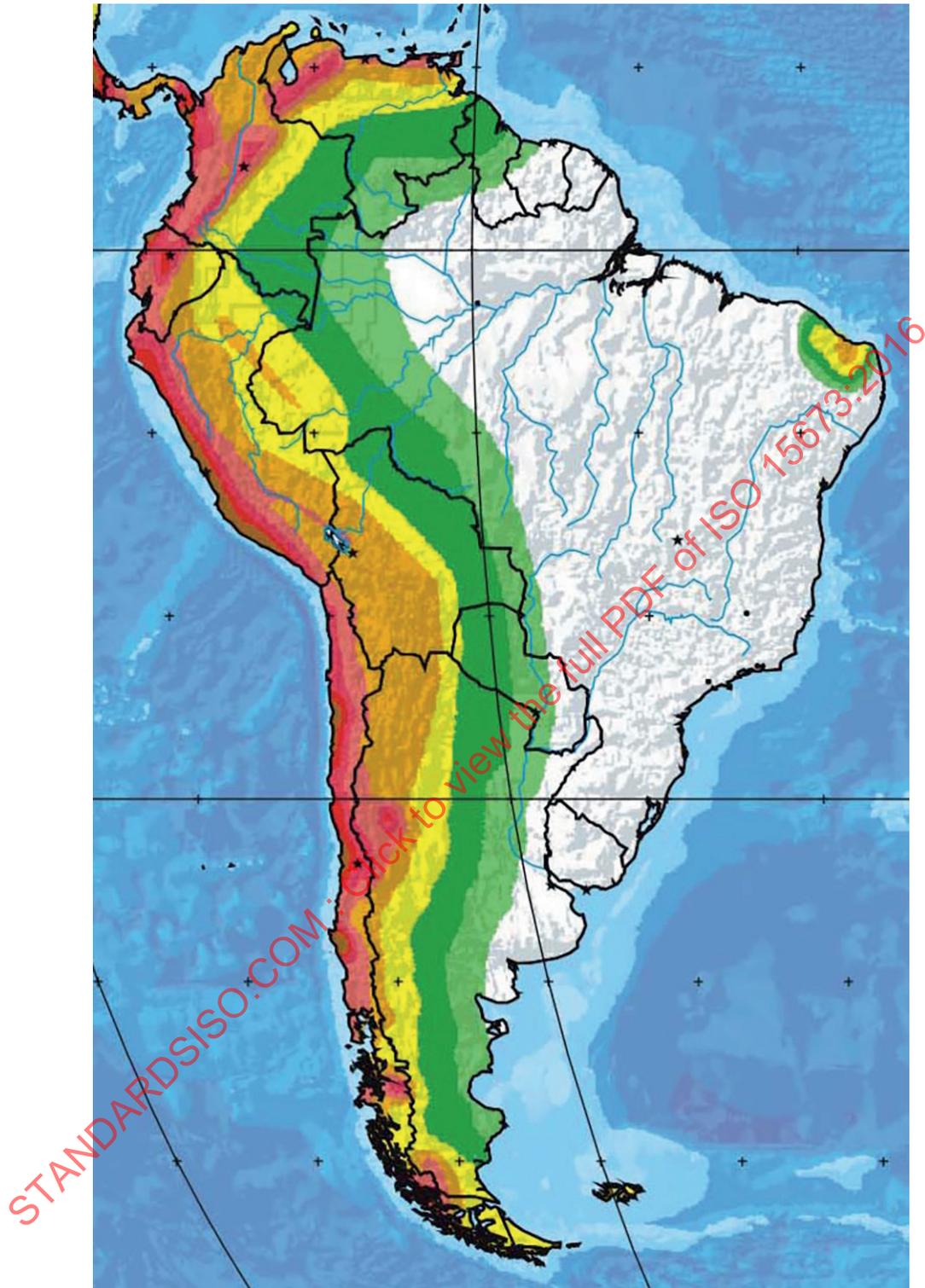


a) North America

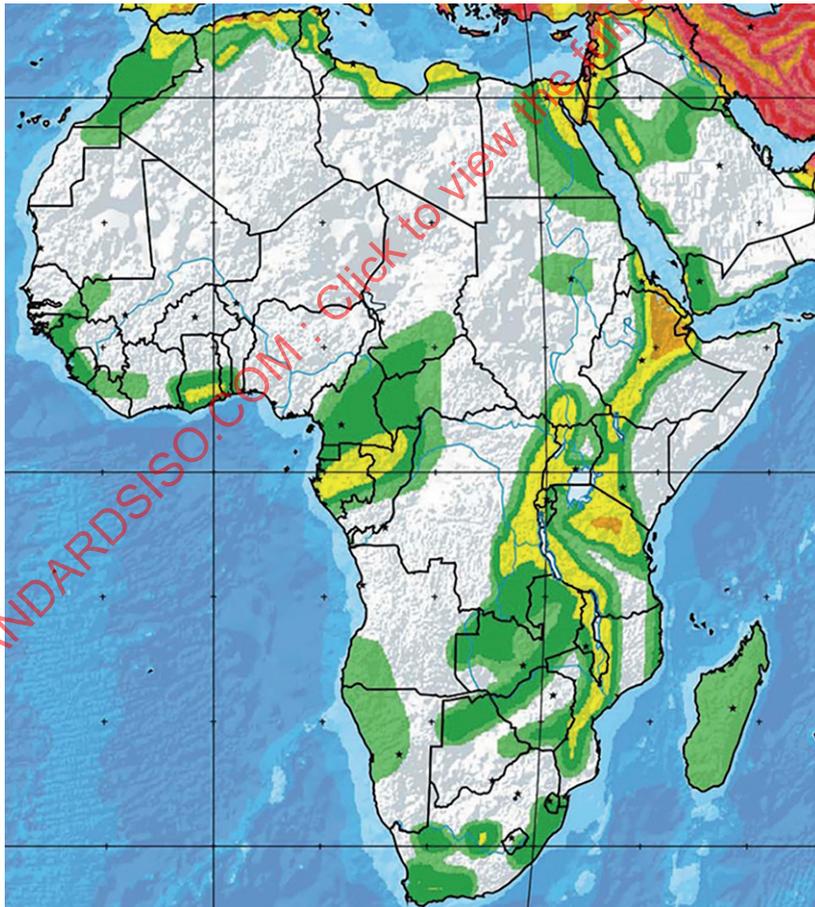
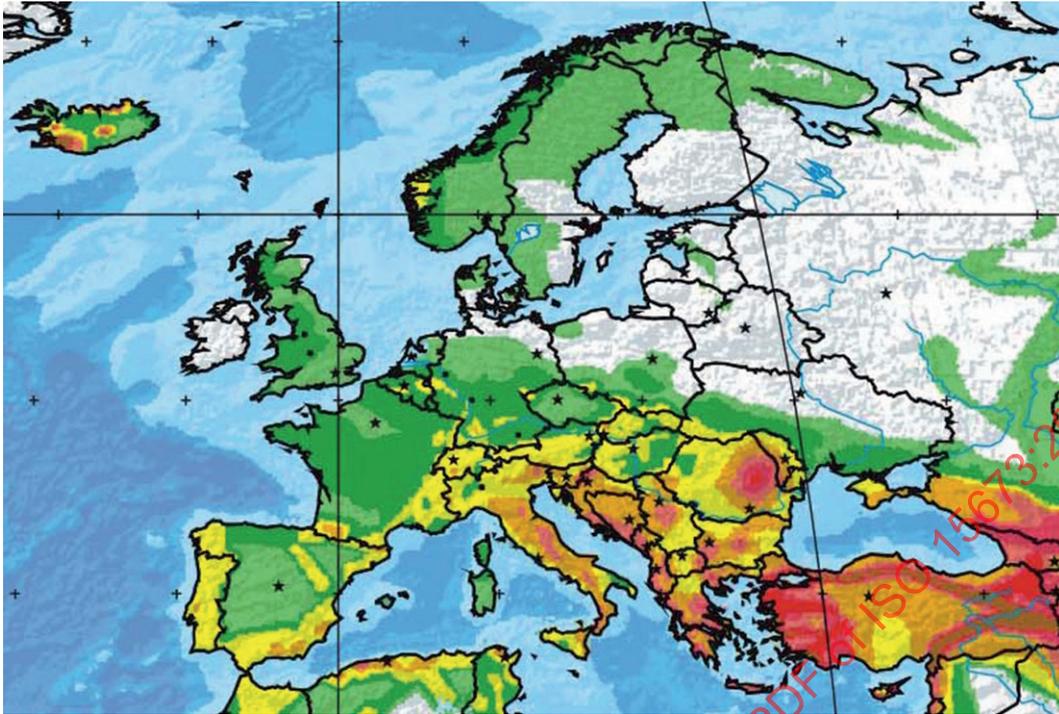


b) Central America and the Caribbean

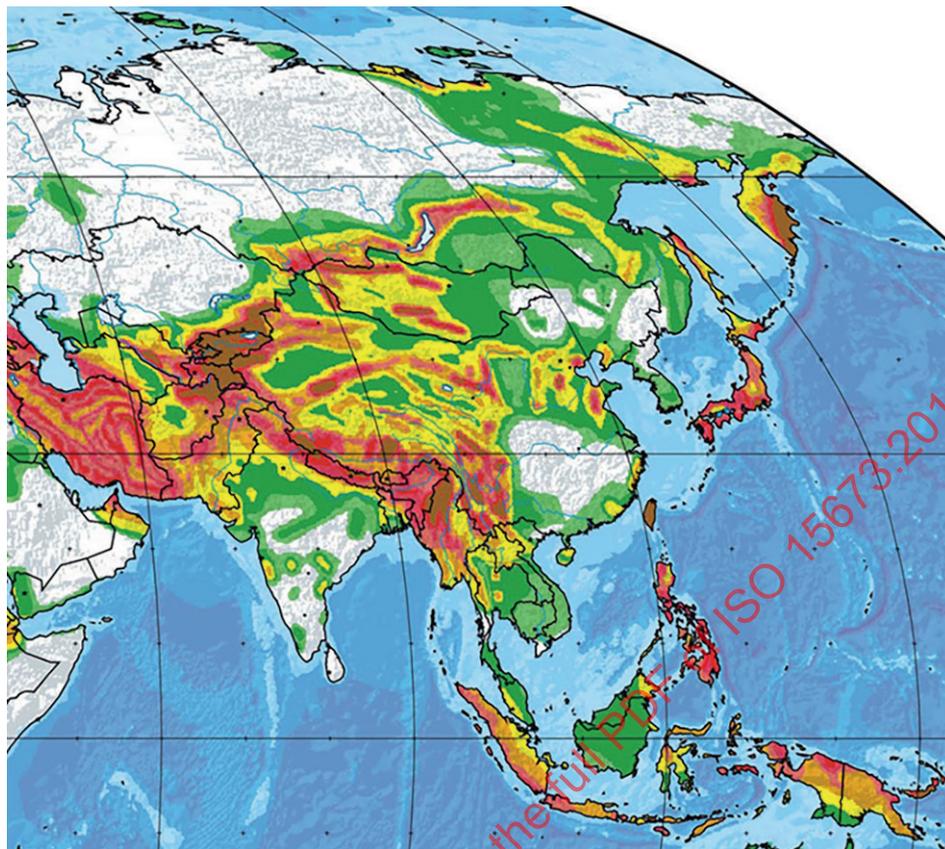
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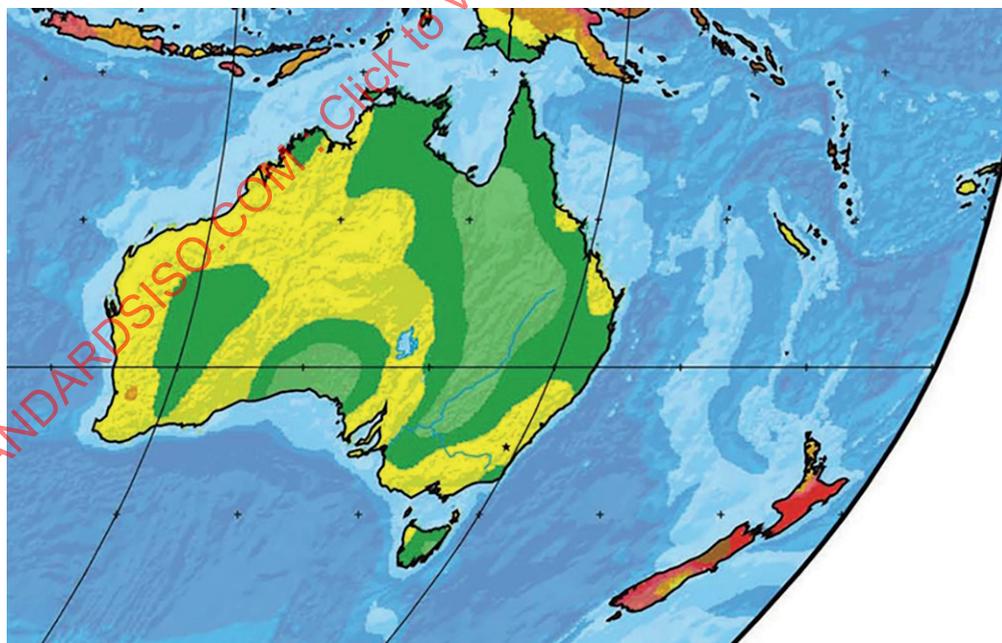
c) South America



**d) Europe and Africa**



e) Asia



f) Oceania

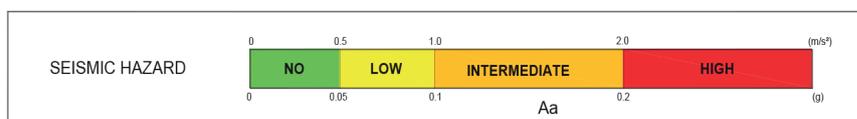


Figure 6 — Global seismic hazard map

**8.1.7.7 Soil profile types**

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

- Soil Profile S<sub>A</sub>: hard rock with a measured shear wave velocity,  $v_s > 1\,500$  m/s.
- Soil Profile S<sub>B</sub>: rock with moderate fracturing and weathering with a measured shear wave velocity in the range ( $1\,500$  m/s  $\geq v_s > 750$  m/s).
- Soil Profile S<sub>C</sub>: soft weathered or fractured rock, or dense or stiff soil, where the measured shear wave velocity is in the range ( $750$  m/s  $\geq v_s > 350$  m/s), or, in the upper 30 m, the standard penetration test resistance has an average value of  $N > 50$  or a shear strength for clays  $s_u \geq 100$  kPa.
- Soil Profile S<sub>D</sub>: predominately medium-dense to dense, or medium stiff to stiff soil, where the measured shear wave velocity is in the range ( $350$  m/s  $\geq v_s > 180$  m/s), or where, in the upper 30 m, the standard penetration test resistance has an average value in the range ( $15 < N \leq 50$ ), or a shear strength for clays in the range ( $50$  kPa  $\leq s_u < 100$  kPa).
- Soil Profile S<sub>E</sub>: a soil profile where the measured shear wave velocity  $v_s \leq 180$  m/s, or the standard penetration test resistance has an average value  $N < 15$  in the upper 30 m, or has more than 3,5 m of plastic ( $PI > 20$ ), high moisture content ( $w > 40$  %) and low shear strength ( $s_u < 25$  kPa) clays.
- Seismically vulnerable soils: sites where the soil profile contains soil having one or more of the following characteristics are beyond the scope of this document:
  - soils vulnerable to potential failure or collapse under seismic motions, such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soil;
  - peats, highly organic clays, or both, with more than 3 m of thickness;
  - very high plasticity clays ( $PI > 75$ ) with more than 8 m of thickness;
  - soft to medium-stiff clays with more than 40 m of thickness.

Soil exploration to obtain the needed values to classify should always be conducted by an engineer familiar with these procedures.

**8.1.7.8 Site effects**

Site effects shall be described through the site soil coefficient for short periods of vibration,  $F_a$ . The values of the site soil coefficient for short periods of vibration,  $F_a$ , shall be determined from Table 3 as a function of  $A_a$ , and the soil profile type from 8.1.7.7. Linear interpolation can be used between values of  $A_a$  in Table 3.

Site effect of seismically vulnerable soils, as described in 8.1.7.7, are beyond the scope of this document and designs should be made under the national standard or other applicable standards.

**Table 3 — Site soil coefficient**

Soil profile	Site coefficient, $F_a$ , for short periods of vibration				
	$A_a \leq [0,1]$	$A_a = [0,2]$	$A_a = [0,3]$	$A_a = [0,4]$	$A_a \geq [0,5]$
S <sub>A</sub>	[0,80]	[0,80]	[0,80]	[0,80]	[0,80]
S <sub>B</sub>	[1,00]	[1,00]	[1,00]	[1,00]	[1,00]
S <sub>C</sub>	[1,20]	[1,20]	[1,10]	[1,00]	[1,00]
S <sub>D</sub>	[1,60]	[1,40]	[1,20]	[1,10]	[1,00]
S <sub>E</sub>	[2,50]	[1,70]	[1,20]	[0,90]	[0,90]

### 8.1.7.9 Design response spectral ordinates

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

The ordinates of the elastic design response spectrum,  $S_a$ , for a 5 % critical damping ratio, expressed as a fraction of the acceleration of gravity, shall be calculated in the short periods of vibration range, using [Formula \(15\)](#):

$$S_a = 2,5 A_a F_a \quad (15)$$

## 8.1.8 Seismic design base shear

### 8.1.8.1 Seismic-resistant structural system

The seismic-resistant structural system shall be classified as a dual building frame system, where an essentially complete moment-resistant space frame provides support for gravity loads, and the resistance to lateral loads is provided by reinforced concrete walls and the moment-resisting space frame provides only a minimum collateral lateral load resistance.

### 8.1.8.2 Energy-dissipation capacity of the seismic-resistant structural system

The energy-dissipation capacity in the inelastic range of the seismic-resistant structural system, described by the response modification factor, shall have a value of  $R = 5,0$ .

### 8.1.8.3 Computation of the seismic design base shear

The seismic design base shear,  $V_s$ , equivalent to the total horizontal inertial effects caused by the seismic ground motions, shall be determined using [Formula \(16\)](#):

$$V_s = \frac{S_a \cdot W}{R} \quad (16)$$

Where  $S_a$  shall be determined from [Formula \(15\)](#),  $R$  is the response modification factor determined from 8.1.8.2, and  $W$  corresponds to the total weight of the building.

$W$  shall include the total weight of the structure, plus the weight of all non-structural elements, such as walls and partitions, permanent equipment, tanks and the contained liquid, in storage occupancies 25 % of the live load and the snow load when the snow load exceeds 1,5 kN/m<sup>2</sup>.

### 8.1.8.4 Vertical distribution of the design seismic forces

The total seismic design base shear shall be distributed over the height of the building using [Formula \(17\)](#) and [Formula \(18\)](#). At each floor level designated as  $x$ ,  $F_x$  shall be applied over the area of the building in accordance with the mass distributions at that level.

$$F_x = C_{vx} \cdot V_s \quad (17)$$

and

$$C_{vx} = \frac{W_x \cdot H_x}{\sum_{i=1}^n (w_i \cdot h_i)} \quad (18)$$

## 9 General reinforced concrete requirements

### 9.1 General

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

### 9.2 Additional requirements

The designer should comply with the additional requirements for each individual element type described in [Clause 10](#) to [Clause 16](#).

### 9.3 Materials for reinforced concrete

#### 9.3.1 General

All materials employed in the construction of the structure designed following these guidelines shall conform to the International Standards listed below.

#### 9.3.2 Cement

Cement shall conform to the following International Standards or corresponding national cement standards:

- ISO 679;
- ISO 29581-1;
- ISO 863.

#### 9.3.3 Aggregates

Aggregates shall conform to the following International Standards or corresponding national aggregate standards:

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

#### 9.3.4 Water

Water used in mixing concrete should be potable, clean and free from injurious amounts of oils, acids, alkalis, salts, organic materials, or other substances deleterious to concrete or reinforcement, and shall conform to the applicable International Standards, specifically ISO 12439 or corresponding national concrete mixing water standard.

#### 9.3.5 Steel reinforcement

##### 9.3.5.1 General

Steel reinforcement should be deformed reinforcement, with the exceptions noted in [9.3.5.4](#), and shall conform to the following limitations, and comply to the corresponding International Standard,

especially ISO 10144. Welded-wire reinforcement should be considered deformed reinforcement, under the present guidelines.

### 9.3.5.2 Deformed reinforcement

The maximum specified yield strength for deformed reinforcement should be 420 MPa. Deformed reinforcing bars shall conform to ISO 6935-2 or corresponding national deformed reinforcement standard. ISO 6935-2 covers grades RB 300, RB 400, and RB 500 (300 MPa, 400 MPa, and 500 MPa characteristic upper yield stress, respectively) and nominal diameters of 10 mm, 12 mm, 16 mm, 20 mm, 25 mm, 32 mm and 40 mm, although under the present guidelines, the nominal diameter of deformed reinforcement bars is limited to 25 mm (see [9.3.8](#)).

### 9.3.5.3 Welded-wire reinforcement

The maximum specified yield strength for wires being part of welded-wire reinforcement should be 500 MPa. Welded wire reinforcement shall conform to ISO 6935-3 or corresponding national welded-wire reinforcement standard. Under the present guidelines, the nominal diameter of wire for welded-wire reinforcement is limited to 10 mm (see [9.3.8](#)).

### 9.3.5.4 Plain reinforcement

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

### 9.3.6 Admixtures

Admixtures shall conform to the applicable International Standards or corresponding national admixtures standard.

### 9.3.7 Storage of materials

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

### 9.3.8 Minimum and maximum reinforcement bar diameter

Reinforcement employed in structures designed under these guidelines should not have a nominal diameter,  $d_b$ , less than the minimum diameter nor should it be larger than the maximum diameter permitted.

	Minimum bar diameter $d_b$	Maximum bar diameter $d_b$
a) Deformed reinforcing bars (see <a href="#">9.3.5.2</a> )	10 mm	25 mm
b) Wire for welded-wire reinforcement (see <a href="#">9.3.5.3</a> )	4 mm	10 mm
c) For stirrups and ties	6 mm	16 mm
d) Plain reinforcing bars (see <a href="#">9.3.5.4</a> )	6 mm	16 mm

9.3.9 Concrete mixture specification

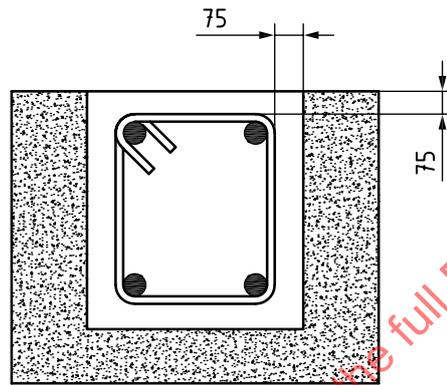
Concrete mixture proportioning procedure should conform to the following International Standard or corresponding national concrete mixture proportioning standards:

- ISO 22965-1;
- ISO 22965-2.

9.3.10 Concrete cover to reinforcement

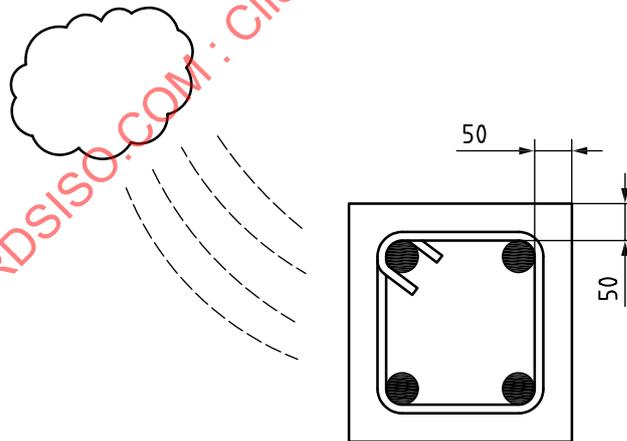
9.3.10.1 Minimum concrete cover

The minimum concrete cover given in Figure 7 to Figure 10 should be provided for reinforcement.



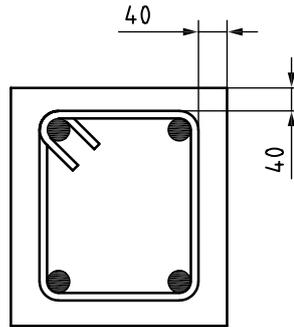
NOTE Minimum concrete cover 75 mm.

Figure 7 — All types of reinforcement of elements cast against and permanently exposed to earth



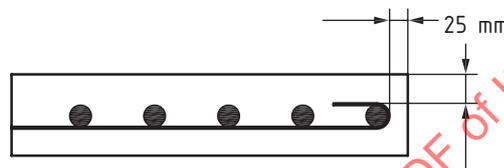
NOTE Minimum concrete cover 50 mm.

Figure 8 — All types of reinforcement of elements exposed to weather or earth



NOTE Minimum concrete cover 40 mm.

**Figure 9 — All types of reinforcement of girders, beams, or columns, when not exposed to weather or in contact with ground**



NOTE Minimum concrete cover 25 mm.

**Figure 10 — All types of reinforcement of solid slabs, structural concrete walls or joists, when not exposed to weather or in contact with ground**

### 9.3.10.2 Special fire protection

When the designated fire protection in hours, for the building, is greater than 1 h, the concrete cover guides shown in [9.3.10.1](#) should be increased by 6 mm per each additional hour of fire protection beyond the first hour.

### 9.3.10.3 Special corrosion protection

In very aggressive environments, special corrosion protection of the reinforcement should be employed, such as epoxy-coated bars, air-entrained concrete concrete coating, special steels and other means. These types of protections are beyond the scope of these guidelines.

### 9.3.11 Minimum reinforcement bend diameter

Diameter of bend of the reinforcement, measured on the inside of the bar, should not be less than the values given in [Figure 11](#).

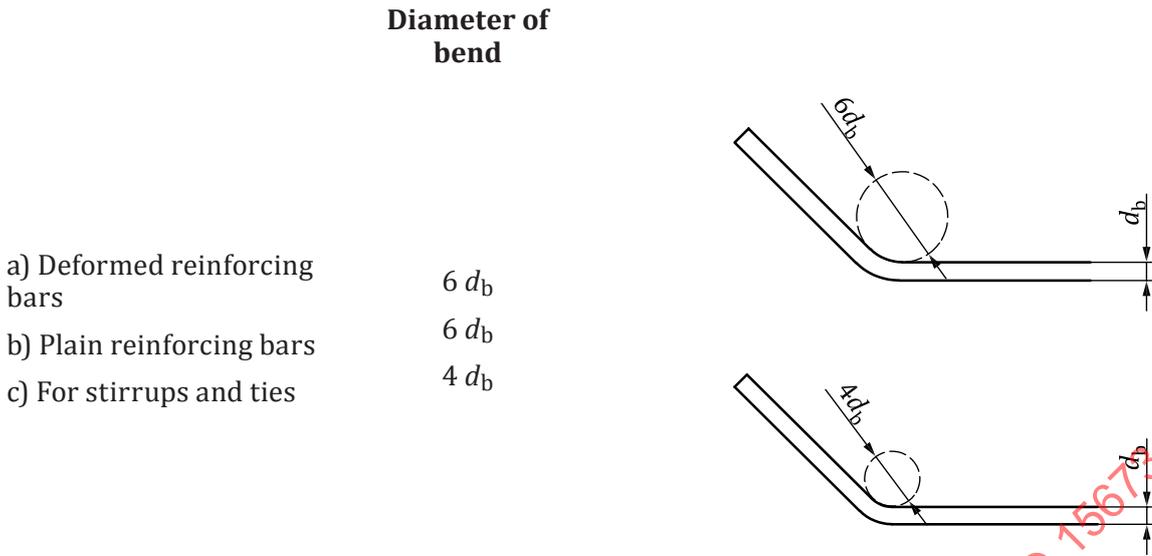


Figure 11 — Minimum reinforcement bend diameter

9.3.12 Standard hook dimensions

The term “standard hook” as used in these guidelines should mean one of the following (see [Figure 12](#) to [Figure 15](#)).

- a) **90° hook** a 90° bend plus a  $12 d_b$  extension at free end of bar

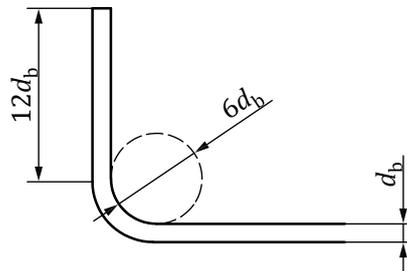


Figure 12 — 90° hook

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

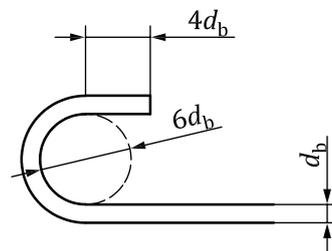
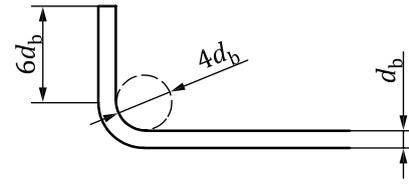


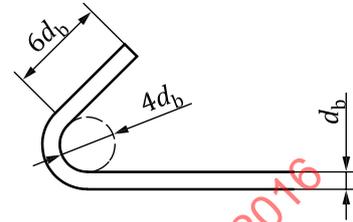
Figure 13 — 180° hook

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



**c) For stirrup and tie hooks**

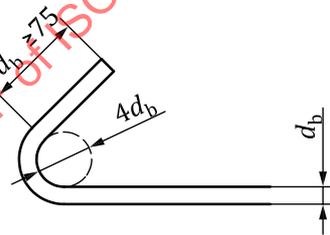
a 135° bend plus  $6 d_b$  extension at free end of bar



**Figure 14 — For stirrup and tie hooks**

**d) For confinement stirrups, ties and crossties in seismic zones**

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



**Figure 15 — For confinement stirrups and ties in seismic zones**

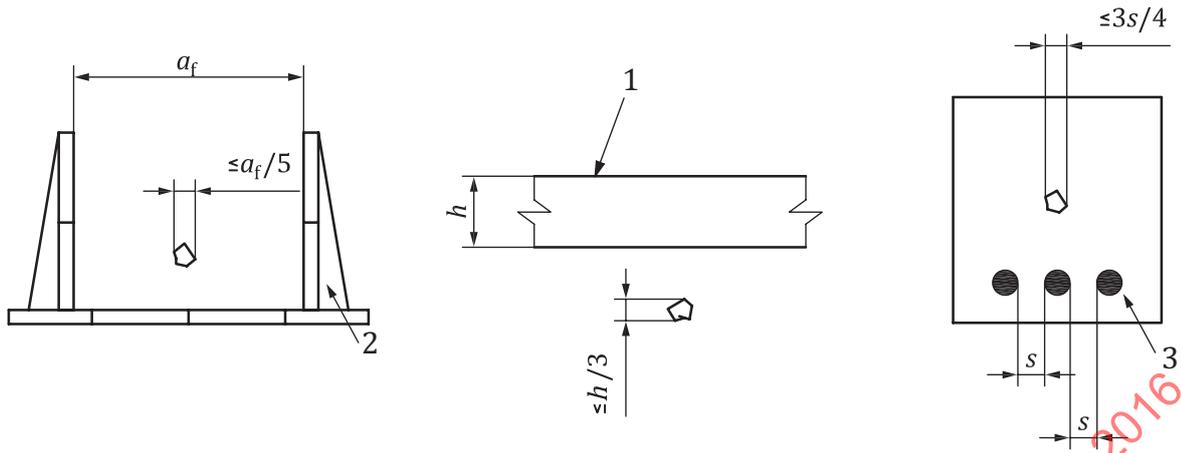
**9.3.13 Bar spacing and maximum aggregate size**

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

**9.3.14 Maximum nominal coarse aggregate size**

Maximum nominal coarse aggregate size, see [Figure 16](#), should be not larger than either of the following:

- 1/5 of the narrowest dimension between sides of forms;
- 1/3 of the depth of slabs;
- 3/4 the minimum clear spacing between parallel reinforcing bars or wires.



**Key**

- 1 slab
- 2 formwork
- 3 parallel bars

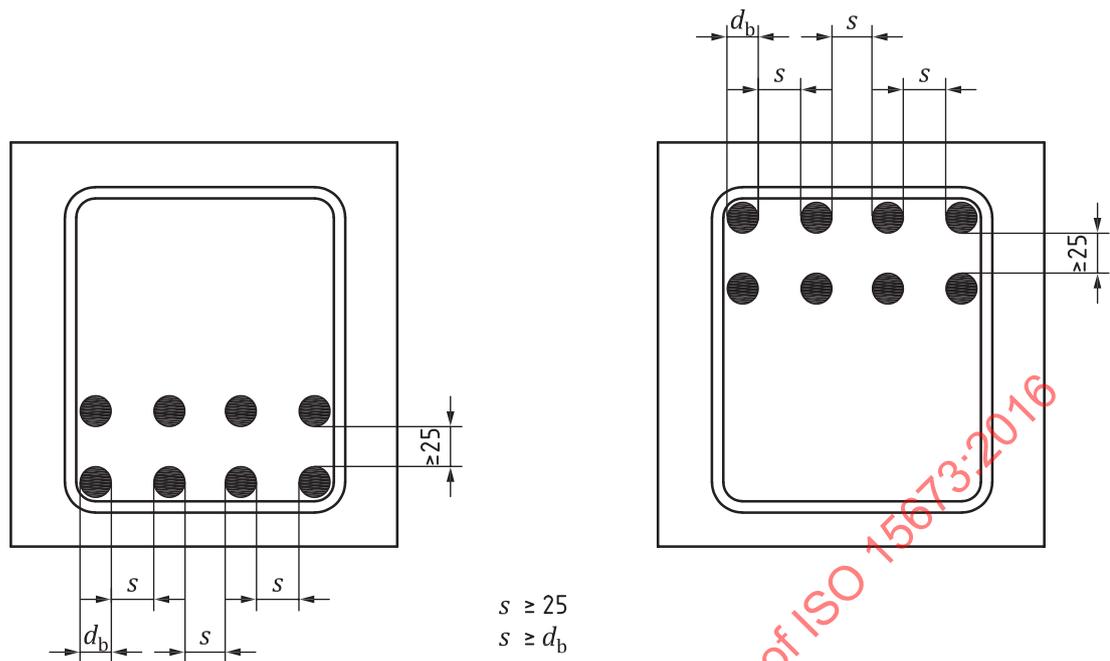
**Figure 16 — Maximum nominal coarse aggregate size**

**9.3.15 Minimum clear spacing between parallel bars in a layer**

In solid slabs, girders, beams and joists, the minimum clear spacing between parallel bars in a layer should be the largest nominal bar diameter,  $d_b$ , but not less than 25 mm (see [Figure 17](#)). These guides should apply also for the spacing between parallel stirrups or ties.

**9.3.16 Minimum clear spacing between parallel layers of reinforcement**

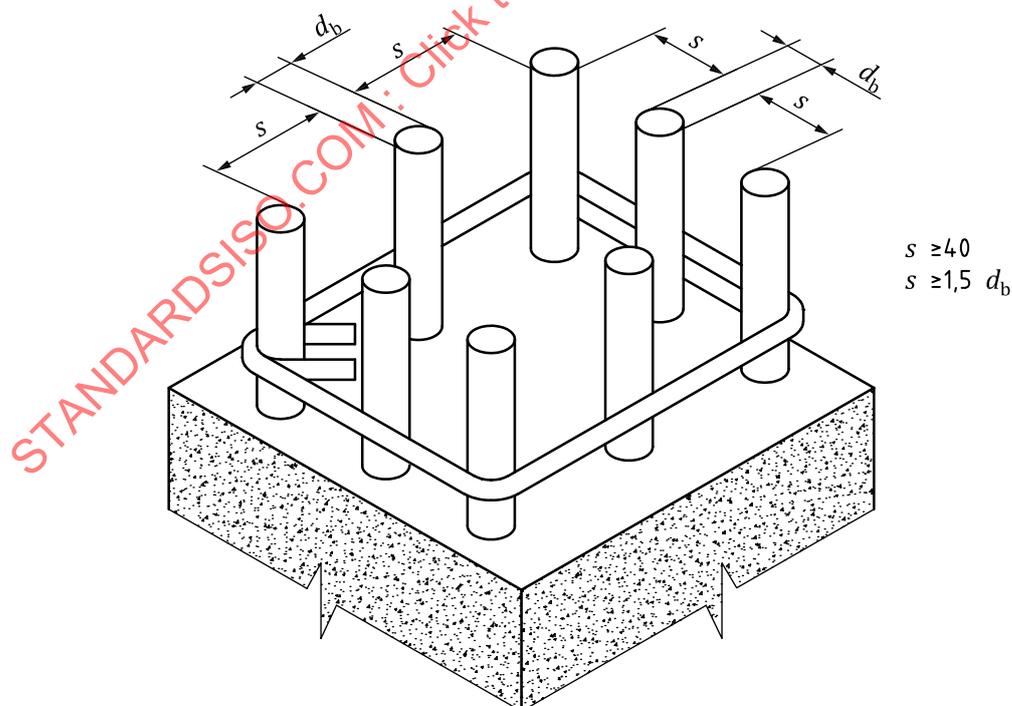
In girders, beams and joists, where parallel reinforcement is placed in two or more layers, bars in the upper layer should be placed directly above bars in the bottom layer with clear distance between layers not less than 25 mm. See [Figure 17](#).



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

### 9.3.17 Minimum clear spacing between longitudinal bars in columns

In columns, clear distance between longitudinal bars should not be less than  $1,5 d_b$  or 40 mm. See [Figure 18](#).



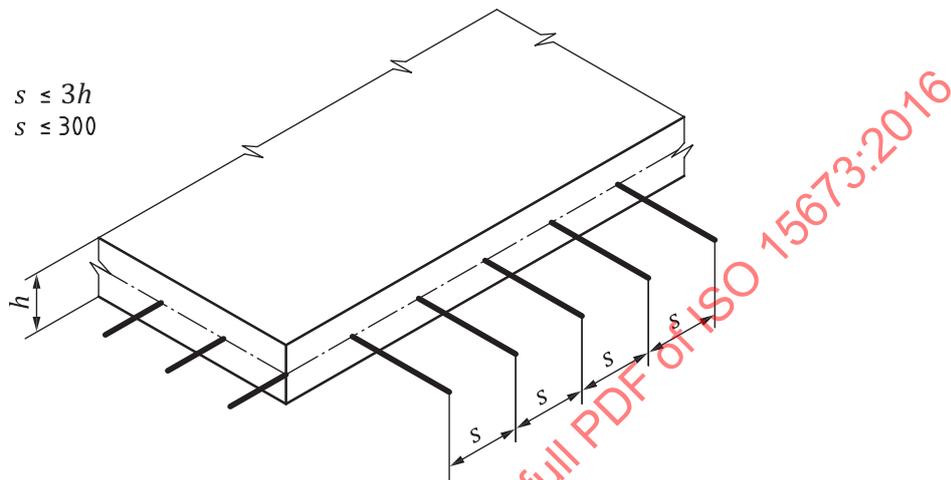
**Figure 18 — Clear distance between longitudinal bars in columns**

**9.3.18 Clear spacing between parallel lap splices**

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

**9.3.19 Maximum flexural reinforcement spacing in solid slabs**

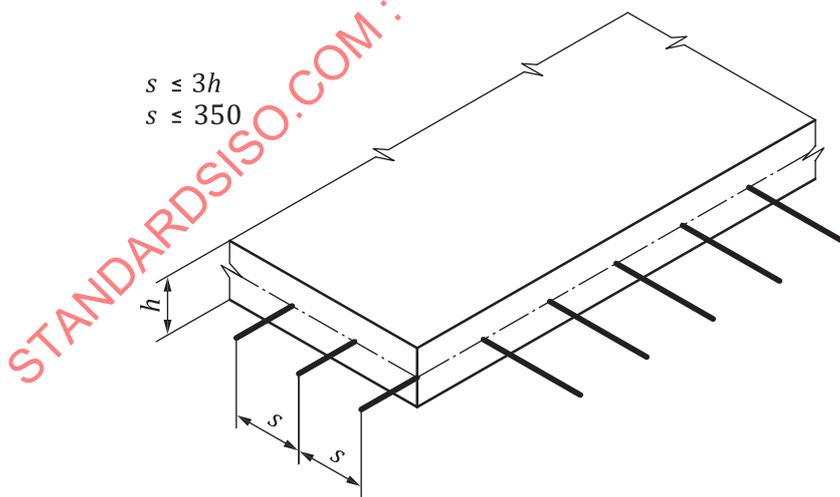
In solid slabs, primary flexural reinforcement should be spaced no farther apart than three times the slab thickness, nor more than 300 mm (see [Figure 19](#)).



**Figure 19 — Spacing between flexural reinforcement in solid slabs**

**9.3.20 Maximum shrinkage and temperature reinforcement spacing in solid slabs**

In slabs, shrinkage and temperature reinforcement, should be spaced no farther apart than four times the slab thickness, nor more than 350 mm (see [Figure 20](#)).



**Figure 20 — Spacing between shrinkage and temperature reinforcement in slabs**

### 9.3.21 Maximum reinforcement spacing in structural concrete walls

#### 9.3.21.1 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 350 mm (see [Figure 21](#)).

#### 9.3.21.2 Number of layers of reinforcement

Structural concrete walls more than 250 mm thick should have vertical and horizontal reinforcement placed in two layers parallel with faces of wall. Each layer should have approximately half of the reinforcement in that direction. The layers should be placed no less than 30 mm nor more than one-third of the thickness of the wall from the surface of the wall. For exterior exposure, the exterior surface layer should be placed no less than 50 mm, instead of the 30 mm prescribed. See additional requirements in [14.4.2](#).

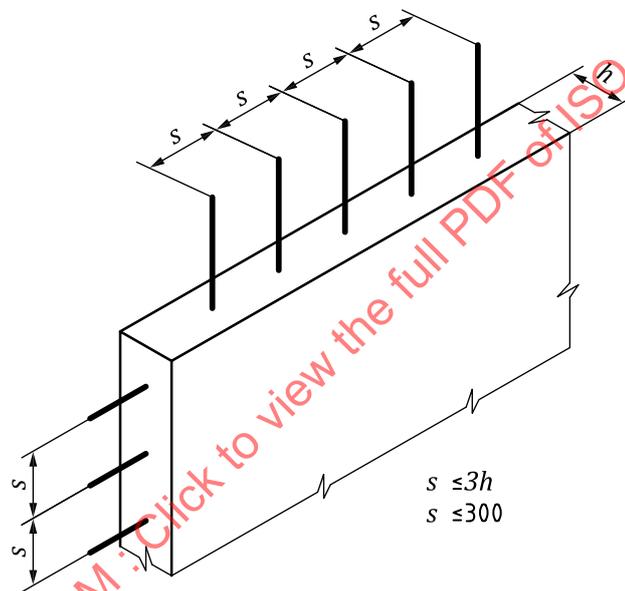


Figure 21 — Spacing between reinforcement in structural concrete walls

### 9.3.22 Special details per element type

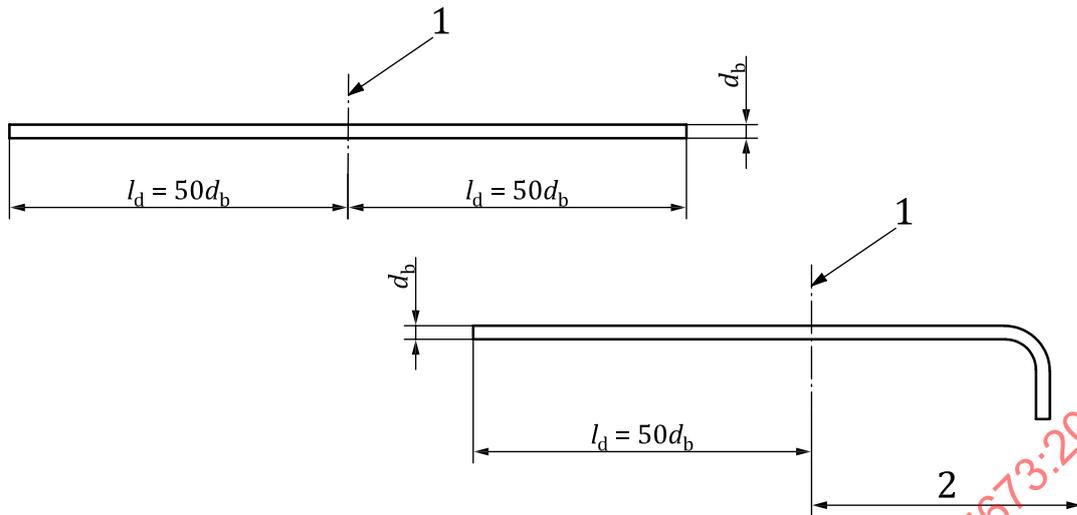
The designer should comply with the additional reinforcement details required for each individual element type, as required by [Clause 9](#) to [Clause 16](#).

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

### 9.4.1 Development length

#### 9.4.1.1 Reinforcing bars

The minimum length of embedment,  $l_d$ , required on each side of a critical section for a reinforcing bar to develop its full strength in tension should be  $50 d_b$ , for the bar diameters permitted by these guidelines in [9.3.8](#). It should be permitted to replace development length in one side of the critical section by a length of bar ending in a standard hook complying with the minimum anchorage distance of [9.4.3](#). See [Figure 22](#).



- Key**
- 1 critical section
  - 2 anchorage distance (see 9.4.3)

**Figure 22 — Required development length for reinforcing bars**

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

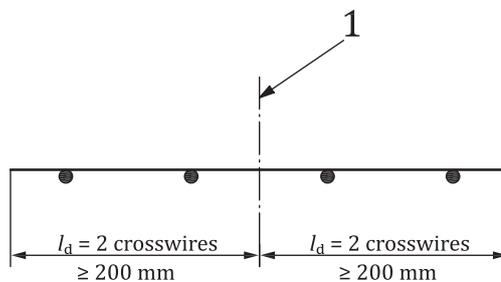
In a more accurate manner, the minimum length of embedment,  $l_d$ , required on each side of a critical section, for a reinforcing bar to develop its full strength in tension may be calculated to a more accurate value by using [Formula 19](#):

$$l_d = \frac{0,25 d_b f_y}{\sqrt{f'_c} \geq 0,04 d_b f_y} \tag{19}$$

The yield strength and the compressive strength are expressed in megapascals, MPa, while the diameter of bar is expressed in millimetres, mm.

**9.4.1.2 Welded-wire reinforcement**

The development length,  $l_d$ , of welded-wire reinforcement measured on each side of the critical section to the end of wire should contain two cross-wires, but should not be less than 200 mm, for the wire diameters permitted by these guidelines in 9.3.8. See [Figure 23](#).



- Key**
- 1 critical section

**Figure 23 — Required development length for welded-wire reinforcement**

## 9.4.2 Lap splice dimensions

### 9.4.2.1 Reinforcing bars

The minimum length of lap for splicing of reinforcing bars should be  $50 d_b$ , for the bar diameters permitted by these guidelines in 9.3.8. See Figure 24.

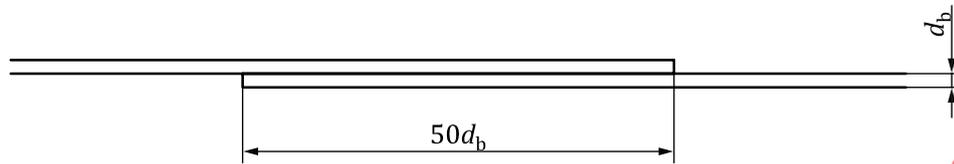


Figure 24 — Minimum lap splice length for reinforcing bars

### 9.4.2.2 Welded-wire reinforcement

Welded-wire reinforcement splicing should be attained by superimposing two cross-wires, but the distance between the edge cross-wires should not be less than 250 mm, for the wire diameters permitted by these guidelines in 9.3.8. See Figure 25.

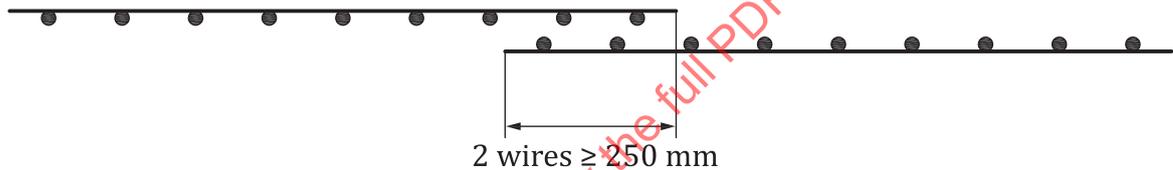
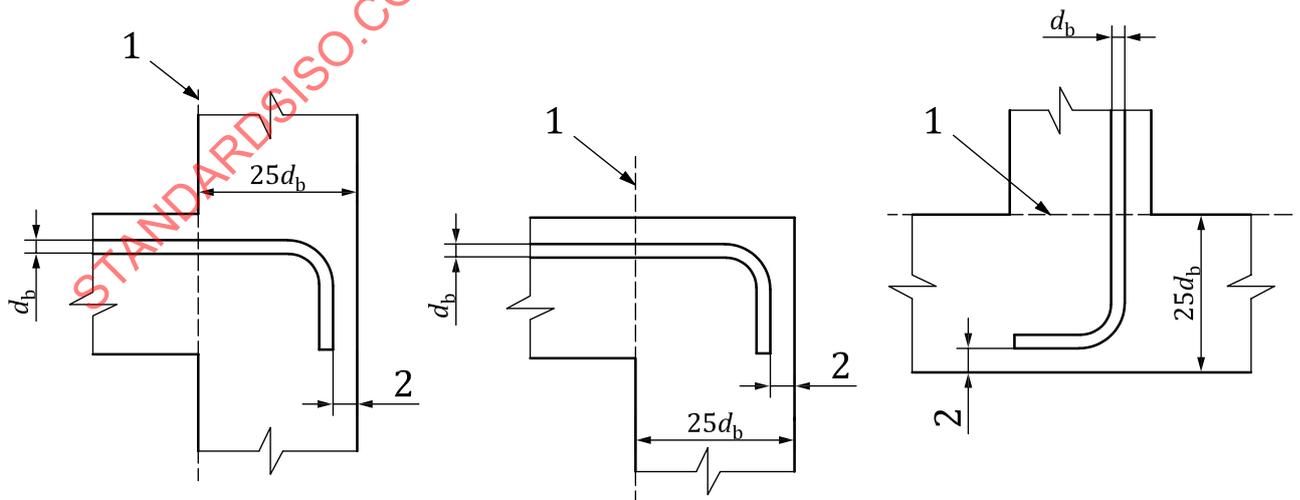


Figure 25 — Minimum lap splice length for welded-wire reinforcement

## 9.4.3 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  $25 d_b$ . See Figure 26.



### Key

- 1 critical section
- 2 cover requirement

Figure 26 — Minimum standard hook anchorage distance

## 9.5 Limits for longitudinal reinforcement

### 9.5.1 General

Longitudinal reinforcement in reinforced concrete structural elements should be provided to resist axial tension, axial compression, flexural induced tension and compression, and/or stresses induced by variation of temperature and drying shrinkage from the concrete. The amount of longitudinal reinforcement employed in the structural elements covered by these guidelines should be that required to resist the factored loads and forces, but should be not less than the minimum values given in 9.5. The dimensions of the structural element should be appropriately modified when the amount of calculated reinforcement required to resist the factored loads and forces exceed the maximum amounts permitted by 9.5.

### 9.5.2 Solid slabs and footings

#### 9.5.2.1 Minimum area of shrinkage and temperature reinforcement

Reinforcement for shrinkage and temperature stresses normal to flexural reinforcement should be provided in structural solid slabs and footings where flexural reinforcement extends in one direction only (see Figure 27). The maximum spacing for this reinforcement should comply with 9.3.20. The following minimum ratios of reinforcement area to gross concrete area,  $\rho_t$ , should be provided for shrinkage and temperature:

- a) where deformed bars with  $f_y < 350$  MPa are used  $\rho_t \geq 0,0020$ ;
- b) where deformed bars or welded-wire reinforcement with  $f_y \geq 350$  MPa are used  $\rho_t \geq 0,0018$ .

#### 9.5.2.2 Minimum area of tension flexural reinforcement

The minimum area of tension flexural reinforcement,  $A_{s,min}$ , in structural solid slabs and footings should be greater than or equal to the reinforcement area required for shrinkage and temperature stresses as required by 9.5.2.1 ( $A_{s,min} \geq \rho_t \cdot b \cdot h$ ). See Figure 27. The maximum spacing of this reinforcement should comply with 9.3.19.

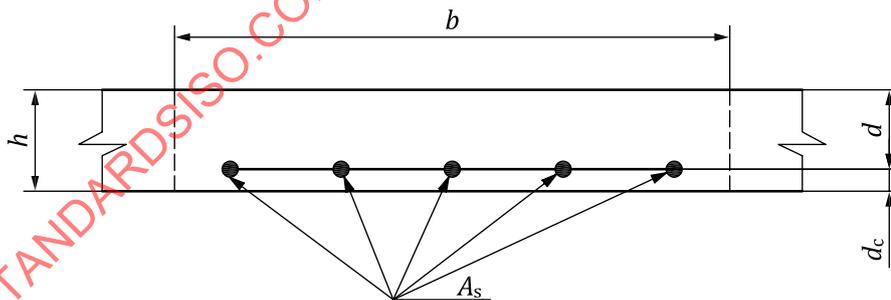


Figure 27 — Slab or footing section

#### 9.5.2.3 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  $\rho_{max}$ , stipulated in Table 4. In solid slabs and footings, flexural reinforcement in compression should not be taken into account in the computation of design moment strength.

**Table 4 — Maximum flexural reinforcement ratio,  $\rho_{\max}$  for solid slabs and footings**

		$f_y$ (MPa)		
		240	300	400
$f'_c$ (MPa)	20	0,022 0	0,016 0	0,011 0
	25	0,027 0	0,020 0	0,014 0
	30	0,032 0	0,024 0	0,016 0
	35	0,036 0	0,027 0	0,018 0
It should be permitted to interpolate for different values of $f_y$ and $f'_c$ .				

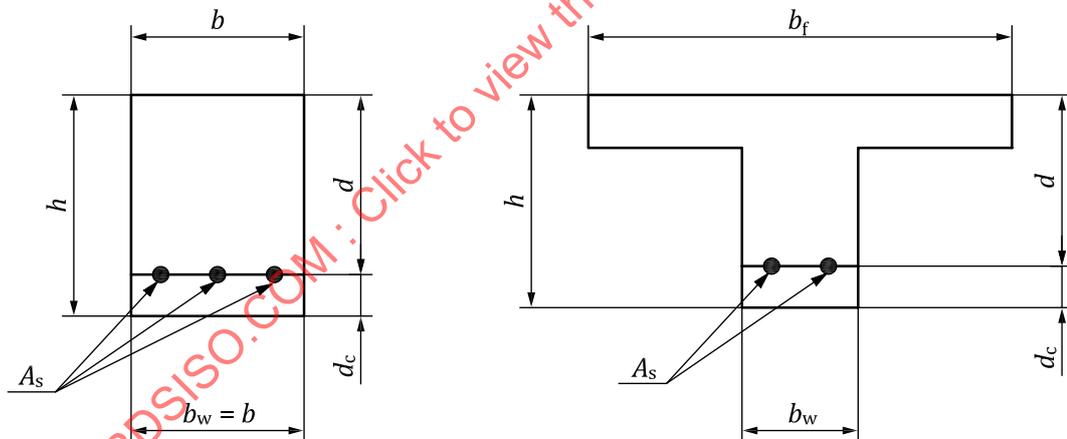
### 9.5.3 Girders, beams and joists

#### 9.5.3.1 Minimum area of tension flexural reinforcement

At every section of a girder, beam or joist, where tension flexural reinforcement is required by [Clause 9](#), the minimum area of tension flexural reinforcement,  $A_{s,\min}$ , should be greater than or equal to the following values, where  $\rho_{\min}$ , is the value stipulated in [Table 5](#).

- a) For rectangular sections and for T sections where the flange is in compression (see [Figure 28](#)):

$$A_{s,\min} = \rho_{\min} \cdot d \cdot b_w \quad (20)$$


**Figure 28 — Rectangular section and T-shaped section with flange in compression**

- b) For T sections where the flange is in tension (see [Figure 29](#)), should be greater than or equal to the smaller value obtained from [Formula \(21\)](#) or [Formula \(22\)](#):

$$A_{s,\min} = 2 \cdot \rho_{\min} \cdot d \cdot b_w \quad (21)$$

$$A_{s,\min} = \rho_{\min} \cdot d \cdot b_f \quad (22)$$

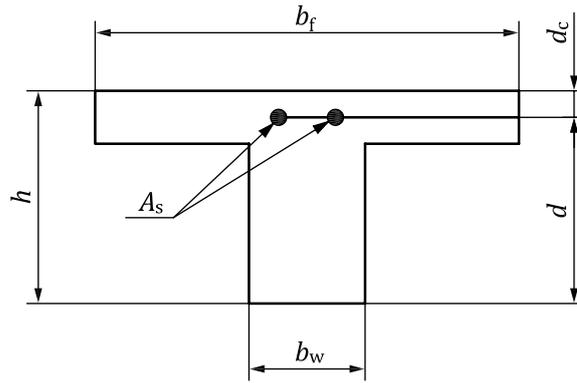


Figure 29 — T-shaped section with flange in tension

Table 5 — Minimum flexural reinforcement ratio,  $\rho_{min}$ , for girders, beams and joists

	$f_y$ (MPa)		
	240	300	400
$\rho_{min}$	0,005 8	0,004 7	0,003 4

It should be permitted to interpolate for different values of  $f_y$  and  $f_c$ , or use the [Formula \(23\)](#):

$$\rho_{min} \geq 0,25 \frac{\sqrt{f_c'}}{\sqrt{f_y}} \geq \frac{1,4}{f_y} \tag{23}$$

9.5.3.2 Maximum flexural reinforcement ratios

The ratio of tension flexural reinforcement,  $\rho$ , should not exceed the following values expressed in function of  $\rho_{max}$  as given in [Table 6](#):

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

$$\rho = \frac{A_s}{b \cdot d} \leq \rho_{max} \tag{24}$$

b) in girders, beams and joists, having tension and compression flexural reinforcement (see [Figure 30](#)):

$$\rho - \rho' = \frac{A_s - A_s'}{b \cdot d} \leq \rho_{max} \tag{25}$$

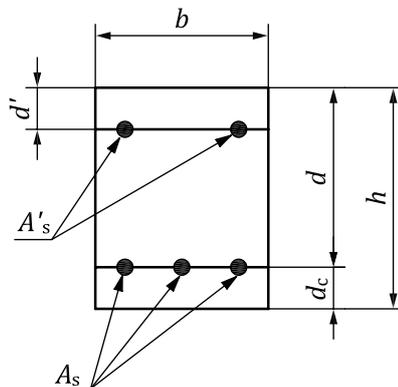


Figure 30 — Section with tension and compression reinforcement

**Table 6 — Maximum flexural reinforcement ratio,  $\rho_{\max}$ , for girders, beams and joists**

		$f_y$ (MPa)		
		240	300	400
$f'_c$ (MPa)	20	0,032 0	0,024 0	0,016 0
	25	0,040 0	0,030 0	0,020 0
	30	0,048 0	0,036 0	0,024 0
	35	0,056 0	0,042 0	0,028 0

It should be permitted to interpolate for different values of  $f_y$  and  $f'_c$  or use [Formula \(26\)](#):

$$\rho_{\max} \geq 0,55 \frac{f'_c}{f_y} \cdot \frac{600}{600 + f_y} \quad (26)$$

#### 9.5.4 Columns

##### 9.5.4.1 Minimum and maximum area of longitudinal reinforcement

The total area of longitudinal reinforcement for columns,  $A_{st}$ , should not be less than 0,01 nor more than 0,06 times the gross area,  $A_g$ , of section as given in [Formula \(27\)](#):

$$0,01 \leq \rho_t = \left[ \frac{A_{st}}{A_g} \right] \leq 0,06 \quad (27)$$

##### 9.5.4.2 Minimum diameter of longitudinal bars

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

##### 9.5.4.3 Minimum number of longitudinal bars

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

##### 9.5.4.4 Distribution of longitudinal bars

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

#### 9.5.5 Structural concrete walls

##### 9.5.5.1 Minimum area of vertical reinforcement

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

### 9.5.5.2 Maximum area of vertical reinforcement

The maximum ratio,  $\rho_v$ , of vertical reinforcement area to gross structural concrete wall horizontal section area should be 0,06, but when the ratio,  $\rho_v$ , exceeds 0,01, the vertical reinforcement should be enclosed with ties as prescribed for columns in [9.6.4.1](#).

$$0,0025 \leq \rho_v = \left[ \frac{A_{st}}{b_w \cdot l_w} \right] \leq 0,06 \quad (28)$$

## 9.6 Minimum amounts of transverse reinforcement

### 9.6.1 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 nonlinear range. The amount of transverse reinforcement employed in the structural elements covered by these guidelines should be that required to resist the factored loads, forces, and stresses, but should be not less than the minimum values given by [9.6](#). The dimensions of the structural element should be appropriately modified when the amount of calculated reinforcement required to resist the factored loads, forces and stresses, exceed the maximum amounts permitted in these guidelines.

### 9.6.2 Slabs

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

### 9.6.3 Girders, beams and joists

#### 9.6.3.1 Minimum transverse reinforcement

The minimum transverse reinforcement in girders, beams and joist should be the required for shear, as specified in [9.8.4.3](#) and [9.8.4.4](#), with the exceptions noted in [9.6.3.2](#).

#### 9.6.3.2 Girders and beams in seismic zones

Girders and beams framing into columns and structural concrete walls located in seismic zones should be provided with confining transverse reinforcement as required in [Clause 17](#).

### 9.6.4 Columns

All columns should have transverse reinforcement in the form of either tie reinforcement or spiral reinforcement conforming to [9.6.4.1](#) or [9.6.4.2](#), respectively.

#### 9.6.4.1 Ties

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

- a) All longitudinal columns bars should be enclosed by lateral ties made with bars at least 10 mm in diameter ( $d_b \geq 10$  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 31](#)).
- 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 31](#)).
- d) The vertical spacing of ties,  $s$ , should not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or the least dimensions of the columns section (see [Figure 32](#)).
- e) The first tie should be located one-half spacing from the top of the slab, beam or footing, where the column is supported, and the uppermost one should be located no more than one-half tie spacing below the lowest horizontal reinforcement of shallowest member supported above.

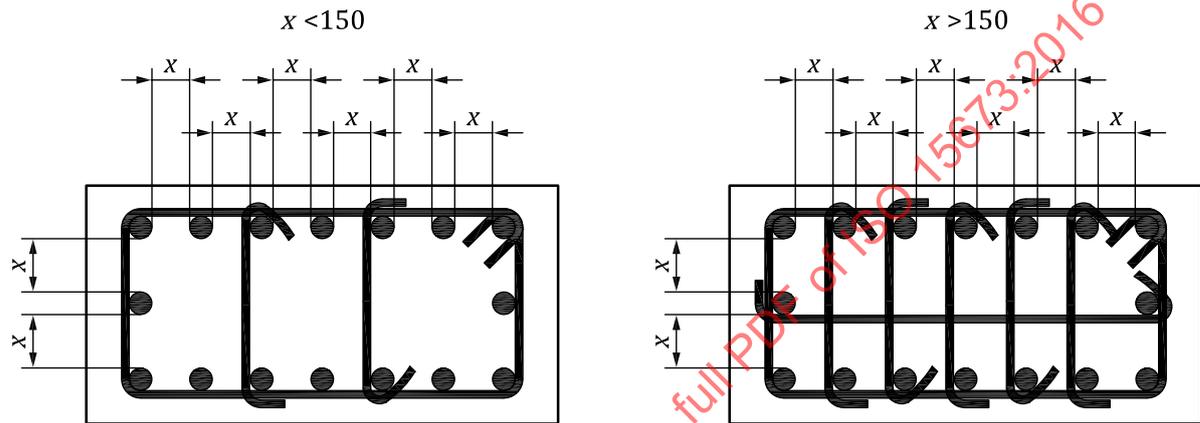
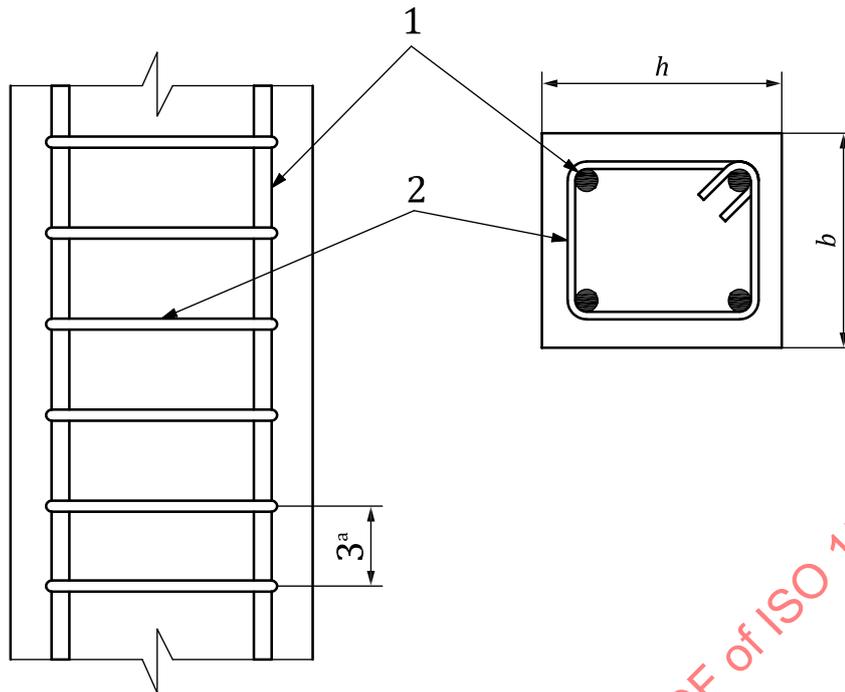


Figure 31 — Arrangement of ties in a tied column section



**Key**

- 1 longitudinal bars
- 2 tie
- 3 smaller of
  - a  $16 d_b$  (longitudinal bars)
  - $48 d_b$  (tie bar)
  - b

**Figure 32 — Vertical spacing of ties in a tied column**

**9.6.4.2 Spirals**

Columns with spiral reinforcement should comply with the following guides.

- a) All longitudinal column bars should be enclosed by a spiral consisting of an evenly spaced continuous bar at least 10 mm in diameter ( $d_b \geq 10$  mm).
- a) Clear spacing between spirals should not exceed 80 mm, nor be less than 25 mm, and should comply with 9.3.13.
- c) Anchorage of the spiral reinforcement should be provided by  $11/2$  extra turns at each end of a spiral unit.
- d) Splices in spiral reinforcement should comply with 9.4.
- e) Spirals should extend from top of footing or slab to level of lowest horizontal reinforcement of shallowest member supported above. In columns with capitals, the spiral should extend to a level at which the diameter or width of capital is two times that of the column.
- f) Ratio of spiral reinforcement,  $\rho_s$ , defined as ratio of the volume of reinforcement contained in one loop of the spiral to the volume of concrete in the core of the column confined by the same loop of spiral, should be not less than any of the values given by Formula (29) (see Figure 33):

$$\rho_s = \frac{A_b \cdot \pi \cdot d_c}{A_c \cdot s} \geq \begin{cases} 0,12 \cdot \frac{f'_c}{f_{ys}} \\ 0,45 \cdot \left[ \frac{A_g}{A_c} - 1 \right] \cdot \frac{f'_c}{f_{ys}} \end{cases} \quad (29)$$

where

$A_b$  is the area of the bar of spiral;

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

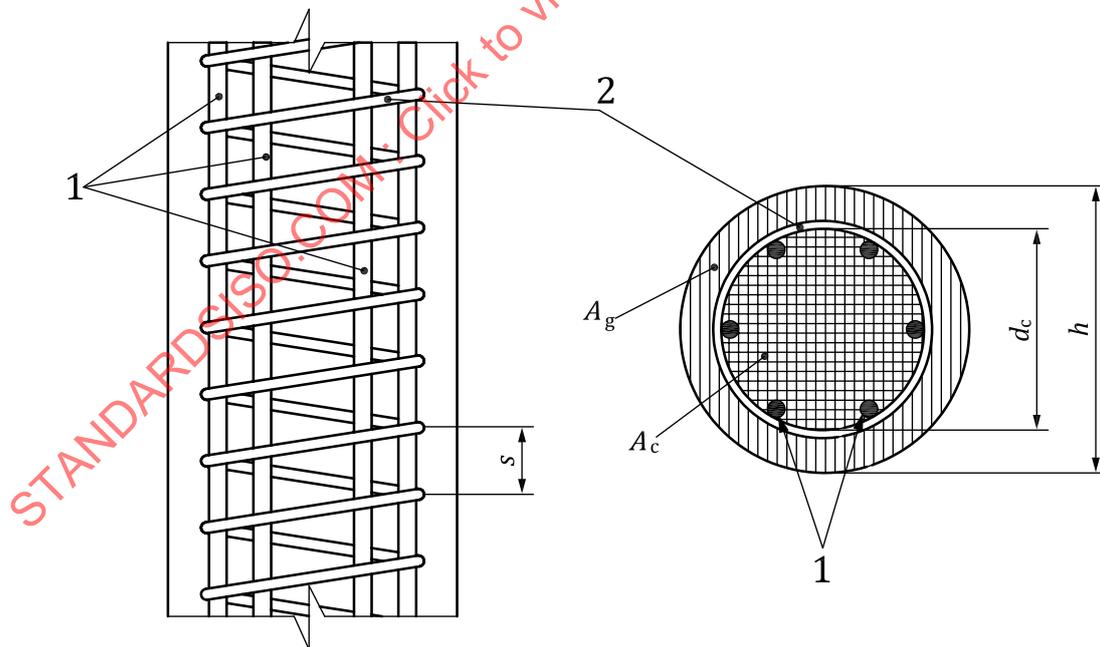
$s$  is the vertical spacing of the spiral;

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

$A_g$  is the gross column section area;

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

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



**Key**

- 1 longitudinal bars
- 2 spiral

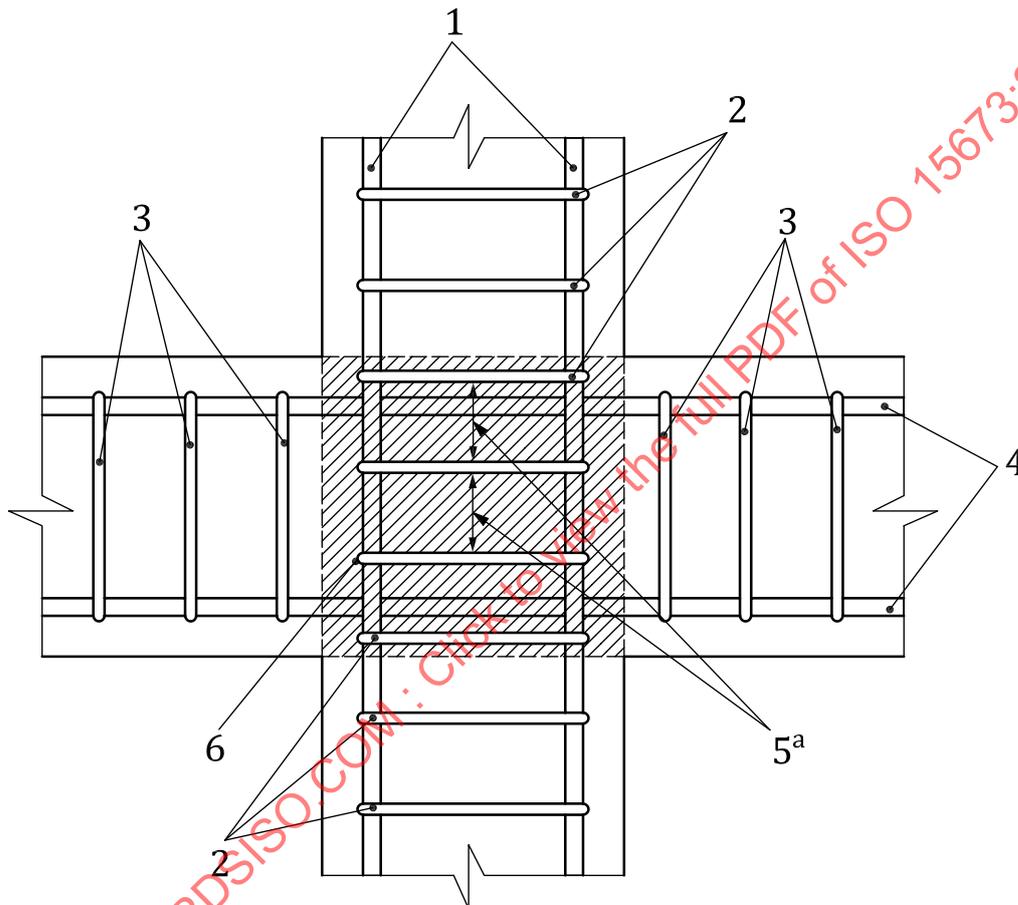
**Figure 33 — Spiral reinforcement of column**

9.6.4.3 Column-girder joints

At joints of frames where columns and girders meet, a minimum of three column ties, complying with a) to c) in 9.6.4.1, should be provided within the joint and the maximum vertical spacing between ties should be 150 mm. As many ties as necessary to comply with the maximum spacing should be provided. See Figure 34.

9.6.5 Structural concrete walls

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



- Key**
- 1 column longitudinal reinforcement
  - 2 column ties
  - 3 girder stirrups
  - 4 girder longitudinal reinforcement
  - 5 joint ties
  - 6 joint
  - a  $s \leq 150$  mm.

Figure 34 — Column ties in column-girder joints

## 9.7 Strength of members subjected to bending moments

### 9.7.1 General

Calculation of the design strength of member sections subjected to bending moments should be performed employing the requirements of 9.7. If the factored axial compressive load on the member,  $P_u$ , exceeds  $(0,10 \cdot f'_c \cdot A_g)$ , the calculation of the design strength should be performed employing the requirements of Clause 13.

### 9.7.2 Factored bending moment at section

The factored bending moment at section,  $M_u$ , caused by the factored loads applied to the structure should be determined, for the particular element type, from the requirements of Clause 8, Clause 10, Clause 11, Clause 12, Clause 15 and Clause 16.

### 9.7.3 Minimum design bending moment strength

The design bending moment strength of the section,  $(\phi \cdot M_n)$ , should be greater than or equal than the factored bending moment at that section,  $M_u$ , as shown in Formula (30).

$$\phi \cdot M_n \geq M_u \quad (30)$$

### 9.7.4 Design moment strength for rectangular sections with tension reinforcement only

#### 9.7.4.1 Design moment strength

For a section with tension reinforcement only, the design moment strength at the section, should be obtained using Formula (31):

$$\phi \cdot M_n = \phi \cdot A_s \cdot f_y \left( d - \frac{a}{2} \right) \quad (31)$$

where the depth of the equivalent uniform stress block,  $a$ , should be (see Figure 35) as given in Formula (32):

$$a = \frac{A_s \cdot f_y}{0,85 \cdot f'_c \cdot b} \quad (32)$$

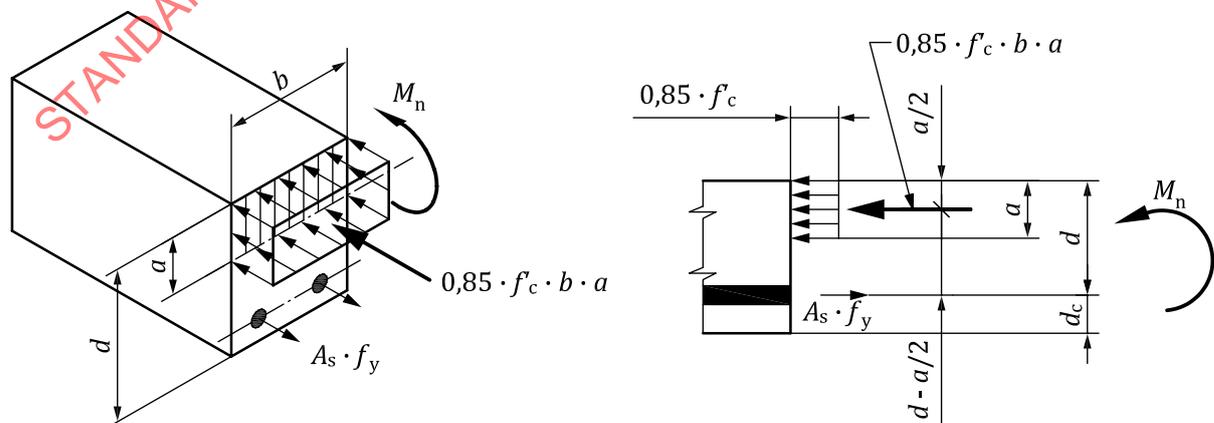


Figure 35 — Flexural nominal moment strength

It should be permitted to use [Formula \(33\)](#), where the value of  $a$  has been introduced in [Formula \(31\)](#) to [Formula 32](#):

$$\phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot d \cdot \left( 1 - 0,59 \cdot \frac{A_s \cdot f_y}{b \cdot d \cdot f'_c} \right) \quad (33)$$

For the purposes of these guidelines, it should be permitted to approximate the design moment strength in slabs, and also in girders, beams and joists where  $\rho < \frac{\rho_{\max}}{2}$ , with  $\rho_{\max}$  from [Tables 4](#) and [6](#), respectively, as [Formula \(34\)](#):

$$\phi \cdot M_n \approx \phi \cdot A_s \cdot f_y \cdot 0,85 \cdot d \quad (34)$$

#### 9.7.4.2 Obtaining the flexural tension reinforcement area

The required ratio of flexural reinforcement,  $\rho = \frac{A_s}{b \cdot d}$ , should be obtained combining [Formula \(30\)](#) with [Formula \(31\)](#), and using the factored flexural moment,  $M_u$ , as given in [Formula \(35\)](#):

$$\rho = \frac{A_s}{b \cdot d} = \alpha - \sqrt{\alpha^2 - \left( \frac{M_u}{\phi \cdot b \cdot d^2} \cdot \frac{2 \cdot \alpha}{f_y} \right)} \quad (35)$$

where  $\alpha = \frac{f'_c}{1,18 \cdot f_y}$ , or using the approximate [Formula \(34\)](#), in slabs where  $\rho < \rho_{\max}$ , with  $\rho_{\max}$  from

[Table 4](#), and in girders, beams and joists where  $\rho < \frac{\rho_{\max}}{2}$ , with  $\rho_{\max}$  from [Table 6](#), as given in [Formula \(36\)](#):

$$\rho = \frac{A_s}{b \cdot d} \approx \frac{M_u}{\phi \cdot b \cdot d^2 \cdot 0,85 \cdot f_y} \quad (36)$$

In [Formula \(35\)](#) and [Formula \(36\)](#),  $\phi = [0,90]$  (see [6.3.3](#)). If the value obtained from [Formula \(35\)](#) or [Formula \(36\)](#) is smaller than  $\rho_{\min}$  from [9.5.2.2](#) and [9.5.3.1](#),  $\rho$  should be increased to that value. For slabs, if the obtained value of  $\rho$  is greater than  $\rho_{\max}$  from [Table 4](#), the slab depth,  $h$ , should be increased, correcting the selfweight of the slab. For girders, beams, and joists, if the obtained value of  $\rho$  is greater than  $\rho_{\max}$  from [Table 6](#), the possibility of either using compression reinforcement (see [9.7.5](#)), or changing dimensions, making the appropriate correction for the selfweight, should be investigated.

### 9.7.5 Use of compression reinforcement in girders, beams, and joists

#### 9.7.5.1 Tension reinforcement less than maximum

If the ratio of tension reinforcement,  $\rho$ , is less than  $\rho_{\max}$  as given in [9.5.3.2](#), the effect of reinforcement in the compression face of the element should be permitted to be disregarded.

#### 9.7.5.2 Shallow doubly reinforced sections

If the ratio of  $\frac{d'}{d}$  is greater than the values given in [Table 7](#), the compression reinforcement should be considered not to be effective.

**Table 7 — Maximum values of  $\frac{d'}{d}$  for compression reinforcement to be effective**

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

**9.7.5.3 Design moment strength of sections with compression reinforcement**

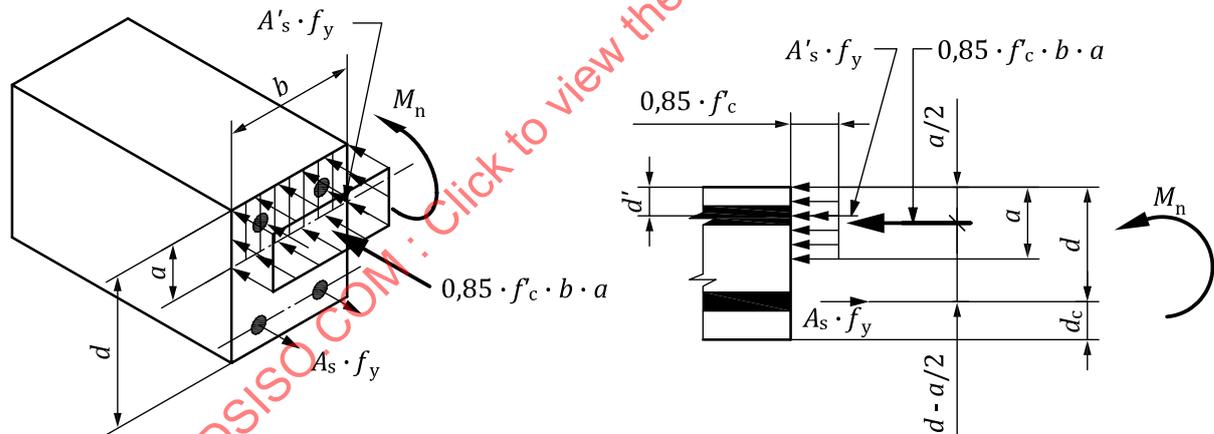
When the condition of  $\frac{d'}{d}$  is met, the design moment strength at the section should be as given in [Formula \(37\)](#) (see [Figure 36](#)):

$$\phi \cdot M_n = \phi \cdot \left[ (A_s - A'_s) \cdot f_y (0,85d) + A'_s \cdot f_y \cdot (d - d') \right] \tag{37}$$

where the depth of the equivalent uniform stress block,  $a$ , should be in this case as given in [Formula \(38\)](#):

$$a = \frac{(A_s - A'_s) \cdot f_y}{0,85 \cdot f'_c \cdot b} \tag{38}$$

Since [Formula \(37\)](#) assumes the compression reinforcement yields, this should be verified.



**Figure 36 — Flexural nominal moment strength for doubly reinforced sections**

9.7.5.4 Obtaining the flexural tension and compression reinforcement area

The required area of flexural tension reinforcement,  $A_s$ , and compression reinforcement,  $A'_s$ , should be obtained combining [Formula \(30\)](#) with [Formula \(37\)](#), and using the factored flexural moment,  $M_u$ , as follows:

$$A'_s = \frac{M_u}{\phi \cdot f_y \cdot (d - d')} - \left[ \frac{b \cdot d^2 \cdot \rho_{\max} \cdot 0,85}{(d - d')} \right] \tag{39}$$

and

$$A_s = A'_s + \rho_{\max} \cdot b \cdot d \tag{40}$$

In [Formula \(39\)](#) and [Formula \(40\)](#),  $\phi = [0,90]$ ; see [6.3.3 a\)](#). The steel ratio,  $\rho_{\max}$ , should be obtained from [Table 6](#). This procedure should be used only when the condition of  $\frac{d'}{d}$  of [9.7.5.2](#) is met. Compression reinforcement should be enclosed by ties as required by [9.6.4.1](#).

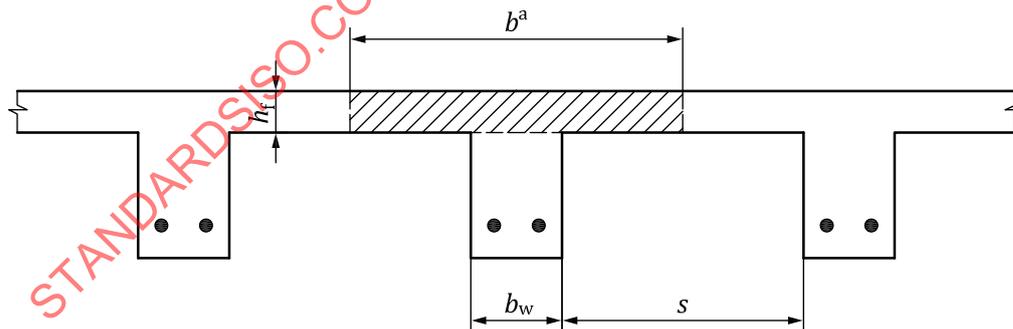
9.7.6 T-beam effect

In beams that are cast monolithically with a slab and when subjected to flexural moments that induce compression stresses in the slab, a portion of the slab should be permitted to act as a flange of the beam, and the flexural design should comply with the requirements of [9.7.6.1](#) to [9.7.6.5](#).

9.7.6.1 Effective flange width for beams with slab in both sides

The width of slab effective as a T-beam flange,  $b$ , should not exceed the following (see [Figure 37](#)):

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



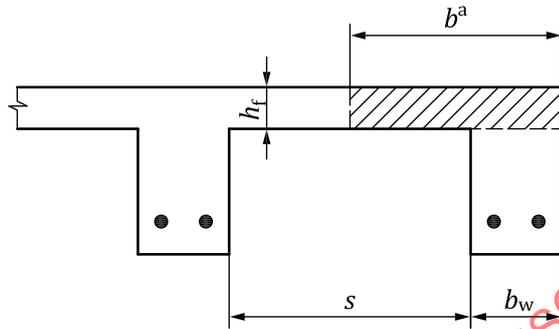
$$a \quad b \leq \min. \text{ of } \begin{cases} l/4 \\ 16 \cdot h_f + b_w \\ s + b_w \end{cases}$$

Figure 37 — Effective flange width for T-beams with slab in both sides

**9.7.6.2 Effective flange width for beams with slab in one side only**

The width of slab effective as a T-beam flange,  $b$ , should not exceed the following (see [Figure 38](#)):

- 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,  $b_w$ .

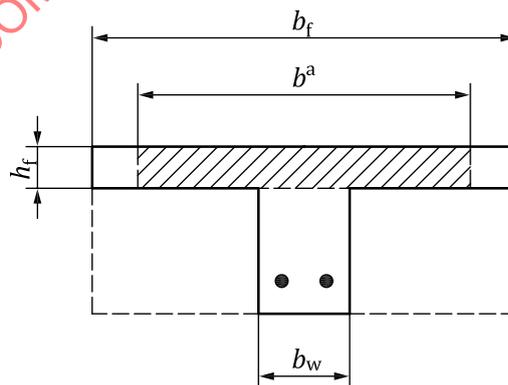


$$a \quad b \leq \min. \text{ of } \begin{cases} l/12 + b_w \\ 6 \cdot h_f + b_w \\ s/2 + b_w \end{cases}$$

**Figure 38 — Effective flange width for T-beams with slab in one side only**

**9.7.6.3 Isolated T-beams**

The flange thickness,  $h_f$ , in isolated T-beams should be at least one-half of the web thickness,  $b_w$ , and the effective flange width,  $b$ , should not exceed  $4 b_w$  nor  $b_f$  (see [Figure 39](#)).



$$a \quad b \leq \min. \text{ of } \begin{cases} 4 \cdot b_w \\ b_f \end{cases}$$

**Figure 39 — Effective flange width for isolated T-beams**

9.7.6.4 Design moment strength of T-beams

When the flange is in compression the moment strength should be calculated as for a rectangular beam using 9.7.4.1, as long as the depth of the equivalent uniform stress block,  $a$ , lies within the flange thickness,  $h_f$ . See Figure 40. The last condition should be verified using Formula 41.

$$h_f \geq a \text{ and } a = \frac{A_s \cdot f_y}{0,85 \cdot f'_c \cdot b} \tag{41}$$

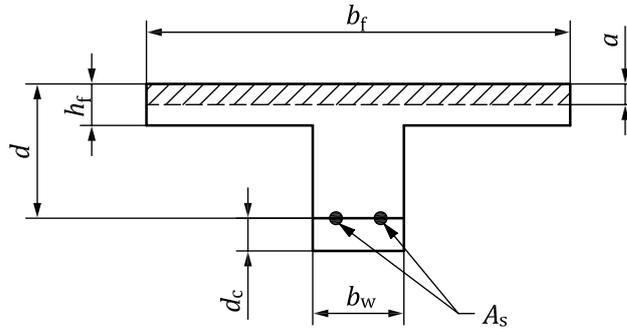


Figure 40 — Effective cross section for moment strength calculation of T-beams

9.7.6.5 Obtaining the flexural tension reinforcement area

The required ratio of flexural reinforcement,  $\rho < \frac{A_s}{(b \cdot d)}$  for T-beams, should be obtained from

Formula (35) or Formula (36), and the flexural reinforcement ratio,  $\rho$ , should not exceed the value given by Formula (42) in order for the depth of the equivalent uniform stress block,  $a$ , to lie within the flange thickness,  $h_f$ .

$$\rho \leq \frac{0,85 \cdot f'_c \cdot h_f}{f_y \cdot d} \tag{42}$$

If the value obtained from Formula (35) or Formula (36) is smaller than  $\rho_{min}$  from 9.5.3.1,  $\rho$  should be increased to that value. If the obtained value of  $\rho$  is greater than  $\rho_{max}$  from Table 6, the dimensions should be changed, making the appropriate correction for the selfweight.

9.8 Strength of members subjected to shear stresses

9.8.1 General

Calculation of the design strength of member sections subjected to diagonal tension or shear stresses should be performed employing the requirements of 9.8. Two type of shear stress effects are covered by these guidelines:

- a) beam-action shear that accompany 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 document.

## 9.8.2 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 [Clause 11](#) to [Clause 16](#).

## 9.8.3 Design shear strength

The design shear strength at the section of the element,  $(\phi \cdot V_n)$ , should be greater than or equal than the factored shear,  $V_u$ , as shown in [Formula \(43\)](#):

$$\phi \cdot V_n \geq V_u \quad (43)$$

In [Formula \(43\)](#),  $\phi = [0,85]$ ; see [6.3.3 d](#)).

## 9.8.4 Beam-action shear

### 9.8.4.1 General

The guides in [9.8.4](#) should be applied to the design of members for beam-action shear. The following general guides should be employed.

- a) Where shear reinforcement is used the design shear strength,  $\phi \cdot V_n$ , should be computed using [Formula \(44\)](#):

$$\phi \cdot V_n = \phi \cdot (V_c + V_s) \quad (44)$$

In [Formula \(44\)](#),  $\phi \cdot V_c$  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 [Formula \(44\)](#),  $\phi = [0,85]$ ; see [6.3.3 d](#)).

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

### 9.8.4.2 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 41](#)) and it should be computed using [Formula \(45\)](#) with  $\phi = [0,85]$ ; see [6.3.3 d](#)).

$$\phi \cdot V_c = \phi \cdot 2 \cdot \left[ \frac{\sqrt{f'_c}}{6} \right] \cdot b_w \cdot d \quad (45)$$

In [Formula 45](#), for solid slabs and footings,  $b_w$  should be taken as the width of the section,  $b$ . See [Figure 42](#).

### 9.8.4.3 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 as given in [Formula \(46\)](#):

$$\phi \cdot V_s = \phi \cdot \left[ \frac{A_v \cdot f_{ys} \cdot d}{s} \right] \quad (46)$$

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 [Formula \(46\)](#),  $\phi = [0,85]$ ; see [6.3.3 d](#)).

The contribution of the shear reinforcement to the design shear strength should not be taken greater than [Formula \(47\)](#):

$$\phi \cdot V_s \leq \phi \cdot \left[ \frac{2}{3} \cdot \sqrt{f'_c} \cdot b_w \cdot d \right] = 4 \cdot \phi \cdot V_c \tag{47}$$

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

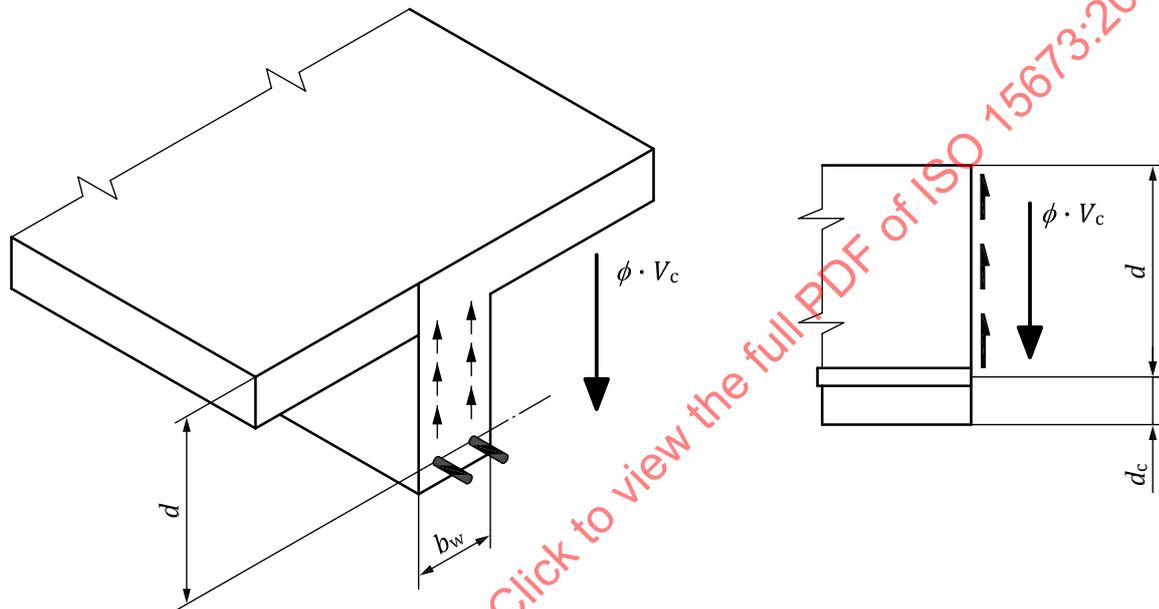


Figure 41 — Contribution of concrete to beam-action shear strength in girders, beams, and joists

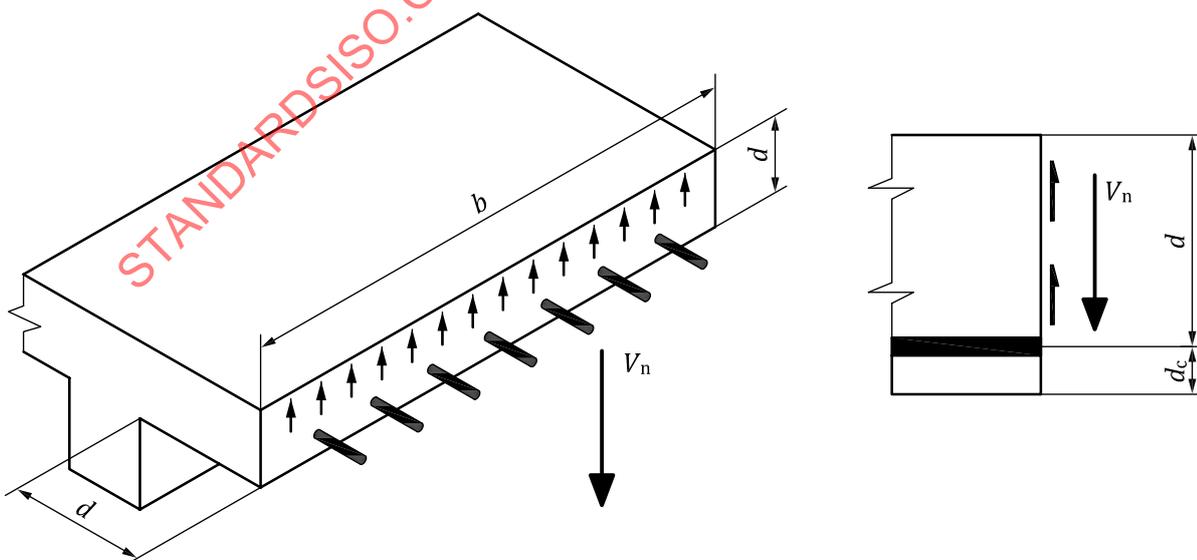


Figure 42 — Contribution of concrete to beam-action shear strength in solid slabs

9.8.4.4 Design of shear reinforcement

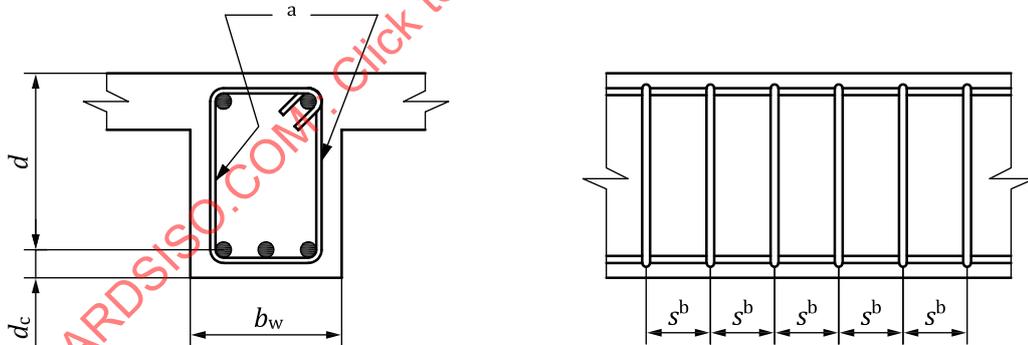
Shear reinforcement in girders, beams and joists, should be provided using stirrups perpendicular to the axis of the member with a maximum spacing  $s$  measured along the axis of the element.

- a) Where the factored shear,  $V_u$ , is less than one-half  $\phi \cdot V_c$ , it should be permitted to waive the use of shear reinforcement.
- b) Where the factored shear,  $V_u$ , exceeds one-half  $\phi \cdot V_c$ , and is less than  $\phi \cdot V_c$ , a minimum amount of shear reinforcement should be employed as specified by [Formula \(48\)](#). The maximum spacing  $s$  along the axis of the element should not exceed  $d/2$ , nor 600 mm. See [Figure 43](#).

$$A_v = \frac{1}{16} \sqrt{f'_c} \frac{b_w \cdot s}{f_{ys}} \geq \frac{b_w \cdot s}{3 \cdot f_{ys}} \tag{48}$$

In [Formula \(48\)](#),  $A_v$  corresponds to the product of the area of the bar of the stirrup,  $A_b$ , multiplied by the number of vertical legs of the stirrup.

- 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 [Formula \(44\)](#), [Formula \(45\)](#) and [Formula \(46\)](#) and the following limitations should be employed (see [Table 8](#)).
  - 1) The amount of shear reinforcement should not be less than that determined using [Formula 46](#).
  - 2) If the value of  $\phi \cdot V_s$ , calculated using [Formula \(46\)](#) is less than  $(2 \cdot \phi \cdot V_c)$  the spacing limits of [9.8.4.4 b\)](#) should be employed.
  - 3) If the value of  $\phi \cdot V_s$ , calculated using [Formula \(46\)](#) is greater than  $(2 \cdot \phi \cdot V_c)$  the spacing limits should be half of the values of [9.8.4.4 b\)](#).
  - 4) The value of  $\phi \cdot V_s$ , calculated using [Formula \(46\)](#) should not be taken greater than  $(4 \cdot \phi \cdot V_c)$ .



- a  $A_v = 1 \cdot A_b$
- b  $s \leq \min. \begin{cases} d/2 \\ 600 \text{ mm} \\ 3 \cdot A_v \cdot f_y / b_w \end{cases}$

**Figure 43 — Minimum shear reinforcement in girders, beams, and joists when  $(\phi \cdot V_c/2 \leq V_u < \phi \cdot V_c)$**

**Table 8 — Shear reinforcement in girders, beams, and joists, maximum spacing**

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

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

**9.8.5.1 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.

**9.8.5.2 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.

### 9.8.5.3 Two-way action shear design strength

The design shear strength should be the smallest of the values obtained from [Formula \(49\)](#), [Formula \(50\)](#) and [Formula \(51\)](#), with  $\phi = [0,85]$ ; see [6.3.3 d](#)):

$$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[ 1 + \frac{2}{\beta_c} \right] \cdot \left[ \frac{\sqrt{f'_c}}{6} \right] \cdot b_0 \cdot d \quad (49)$$

where  $\beta_c$  is the ratio of long side to short side of the column, concentrated load or reaction area, as given in [Formula \(50\)](#):

$$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[ 2 + \frac{\alpha_s \cdot d}{b_0} \right] \cdot \left[ \frac{\sqrt{f'_c}}{12} \right] \cdot b_0 \cdot d \quad (50)$$

where  $\alpha_s$  is 40 for interior columns, 30 for edge columns, and 20 for corner columns, and

$$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[ \frac{\sqrt{f'_c}}{3} \right] \cdot b_0 \cdot d \quad (51)$$

## 10 Floor systems

### 10.1 Types of floor systems

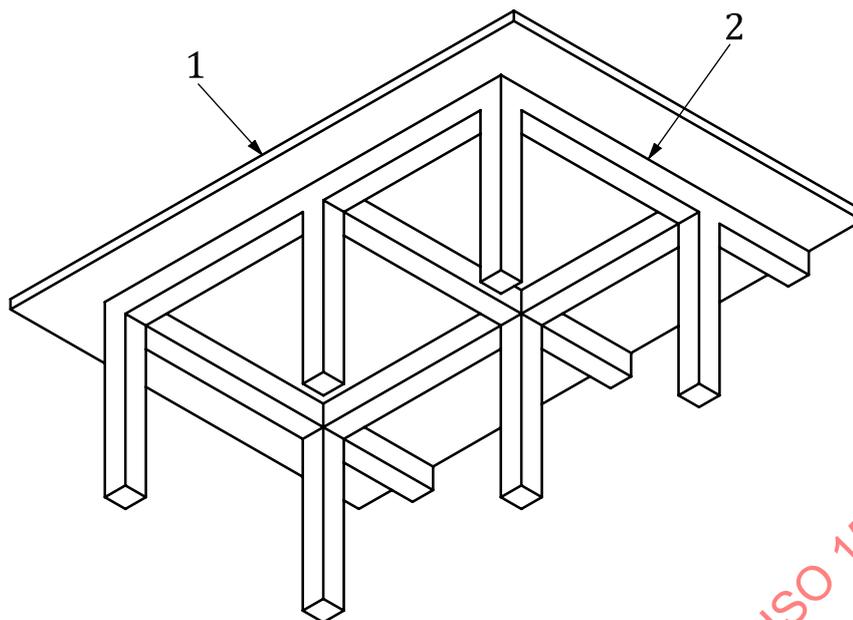
#### 10.1.1 General

This clause describes the floor systems covered by the scope of this document. The floor systems employed by a building designed under this document should be one of the systems covered or their permitted variations. The selection of an appropriate floor systems should be performed studying several alternatives.

#### 10.1.2 Slab-on-girder system

#### 10.1.3 Description of the basic system

This system consists of a grid of girders in both main plan directions with a slab spanning the space between girders. These girders are located in the column lines or axis, spanning the distance between columns. A solid slab is supported by the girders. The slab can cantilever out of the edge beam. In this system, the slab has a shallower depth than the girders (see [Figure 44](#)). For this system, the guides for structural integrity of [10.3](#) should be complied with.

**Key**

- 1 slab
- 2 girder

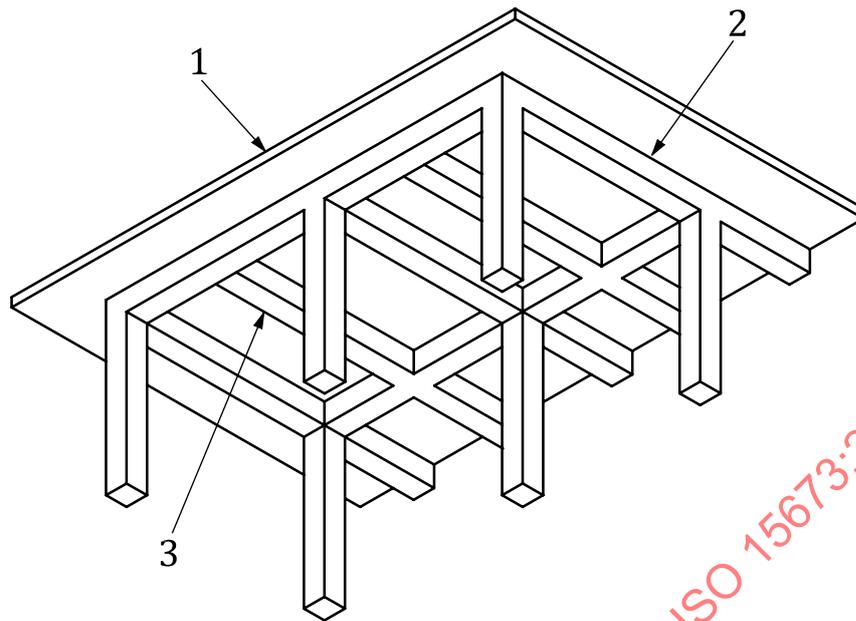
**Figure 44 — Slab-on-girder floor system**

### 10.1.3.1 Use of intermediate beams

One of the main variations of the system is the use of intermediate beams, supported on the girders. One or several beams can be employed per span. The intermediate beams can be of the same height of the girders or shallower. These intermediate beams can be used in one direction, as shown in [Figure 45](#), or in two directions, as shown in [Figure 46](#). The use of too many intermediate beams will make the system gravitate to the joist system, described in [10.1.4](#).

### 10.1.3.2 Advantages of slab-on-girder system

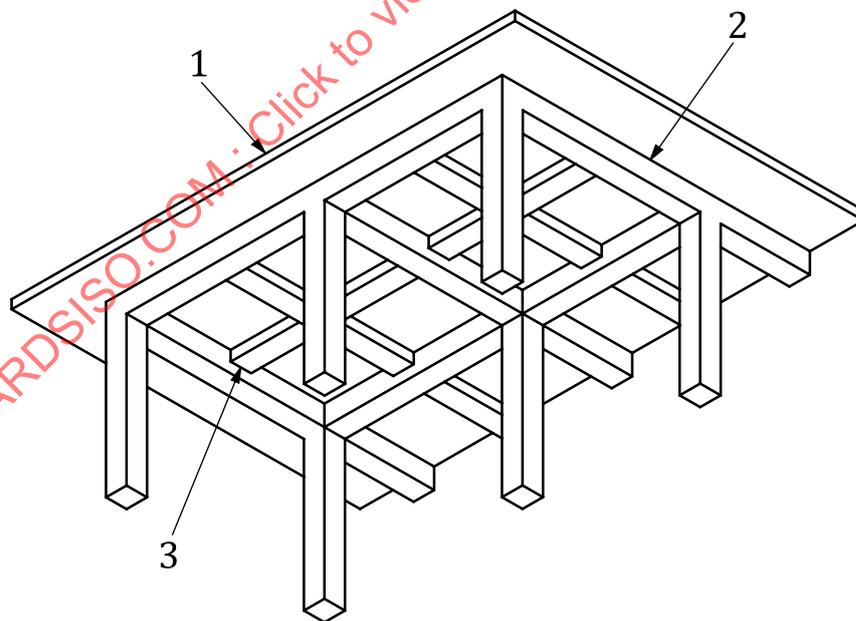
For the slab-on-girder system, each component has the appropriate minimum depth and width to comply with the strength or serviceability guides; therefore, having a relatively low selfweight. The system can accommodate spans of any size, can easily be adapted to any plan shape, and large perforations,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



**Key**

- 1 slab
- 2 girder
- 3 intermediate one-direction beam

**Figure 45 — Use of one-direction intermediate beams in the slab-on-girder floor system**



**Key**

- 1 slab
- 2 girder
- 3 intermediate two-direction beam

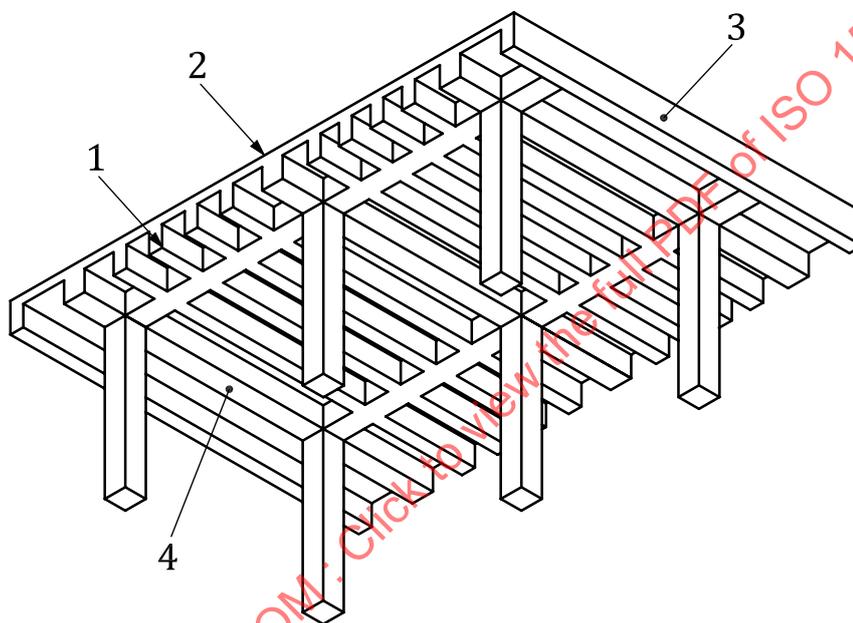
**Figure 46 — Use of two-direction intermediate beams in the slab-on-girder floor system**

## 10.1.4 Joist systems

### 10.1.4.1 Description of the basic system

The joist system consists of a series of parallel ribs, or joists, supported by girders. The girders are located in the column lines or axis, spanning the distance between columns. A thin solid slab spans the space between joists. See [Figure 47](#).

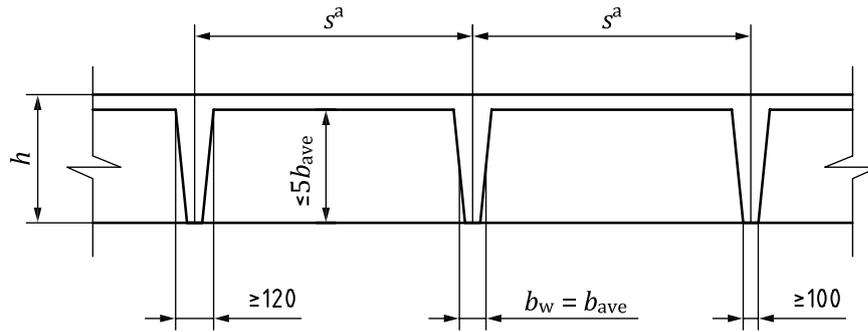
For this system, the guides for structural integrity of [10.3](#) should be complied with. The thin slab cannot cantilever out of the edge joist. In this system, the joists area usually of the same depth of the girders, but can have a shallower depth. The separation between parallel joists, measured 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 48](#). The thin slab should comply with the minimum thickness guides of [10.5.2.1](#).



#### Key

- 1 thin top slab
- 2 joist
- 3 edge joist
- 4 girder or bean

Figure 47 — Joist floor system



- <sup>a</sup>  $s \leq 2,5 h$   
 $s \leq 1,2 m$

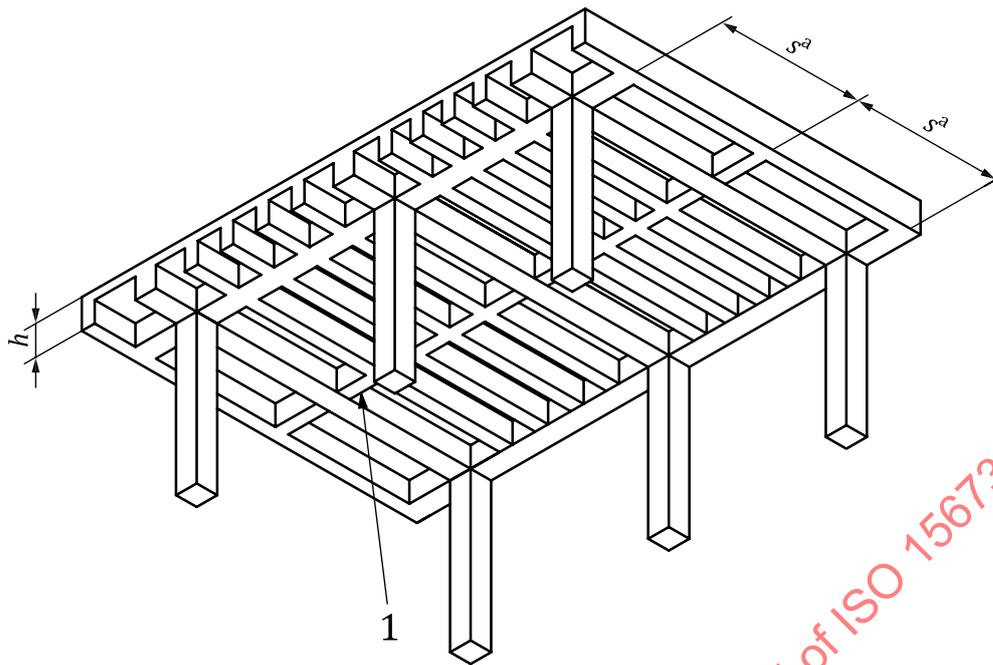
Figure 48 — Joist section dimension guides

#### 10.1.4.2 Type of formwork

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

#### 10.1.4.3 Distribution ribs

In joist systems that span in only one direction, in order to avoid that a concentrated load be carried by just one joist, transverse distribution ribs should be employed with separations of no more than 10 times the total depth,  $h$ , of the joist, without exceeding 4 m. See [Figure 49](#).



**Key**

1 distribution rib

a  $s \leq 10 h$

$s \leq 4 m$

**Figure 49 — Distribution ribs**

**10.1.4.4 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 should be supported on girders. This system is referred as a waffle-on-beams slab. See [Figure 50](#). If the beams are omitted, the system should be classified as the waffle-slab system described in [10.1.4.4](#).

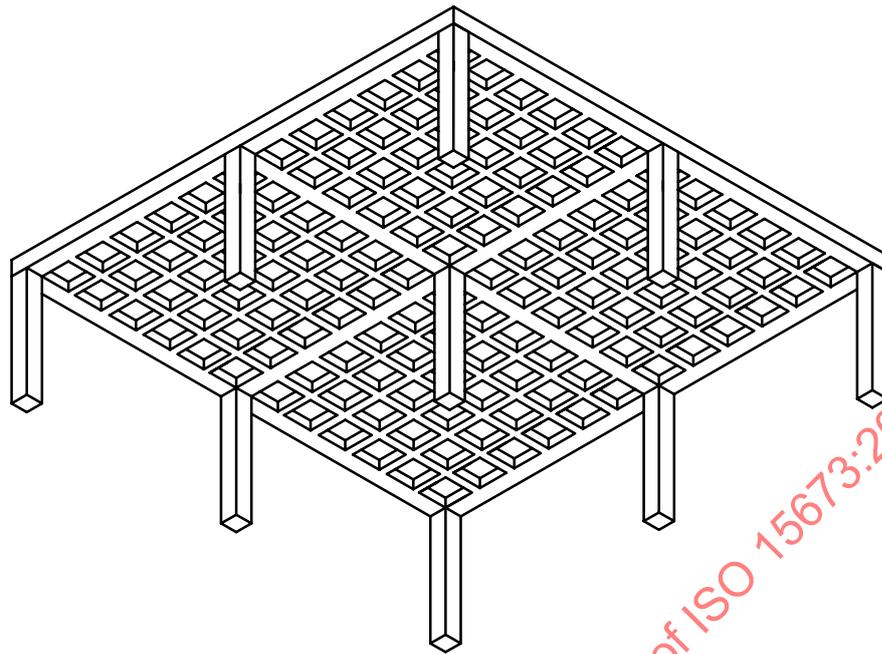


Figure 50 — Two-way joist system or waffle-on-beams system

#### 10.1.4.5 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 guides can easily be accommodated because of the relatively high depth of the system. The clear spacing between joists is a tradeoff between a thinner top slab and requiring a larger amount of joists, thus allowing the designer great freedom in the choice of appropriate dimensions.

### 10.2 Criteria for the selection of the floor system

The structural designer should select a floor system from the systems covered by these guidelines, as presented in [10.1](#). Several alternatives should be studied and the final selection should be performed taking into account the merits of each of them in terms of the following:

- a) the magnitude of the dead and live loads and especially the selfweight of the system;
- b) the geometry of the structural plan layout especially the span lengths in both plan directions and the ratio between them;
- c) the presence of cantilevers and their maximum span and direction;
- d) the type of occupancy of the building;
- e) the available material strengths, both for concrete and reinforcing steel;
- f) the expected behaviour of the slab system, and the adequacy to comply with the serviceability and deflection criteria;
- g) the amount of materials (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 plays an important role in the selection, and following it might simplify construction coordination;

- i) workmanship training and proficiency should affect the selection, thus avoiding systems that require more training and proficiency than what the local workers can comply with;
- j) the relative cost of the alternatives, but the economical advantages should be pondered against the expected behaviour and safety of the system.

### 10.3 Guides for structural integrity

#### 10.3.1 General

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

#### 10.3.2 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, as guided by [12.3.5.5](#). This reinforcement should always be lap-spliced using the minimum lap spliced length of [9.4.2](#).

#### 10.3.3 Other beams and girders

All beams and girders, except the perimeter girders guide by [10.3.2](#) should have closed stirrups and a minimum area of continuous bottom longitudinal reinforcement, as required by [12.3.5.5](#). This reinforcement should always be lap-spliced, at or close to the supports, using the minimum lap splice length of [9.4.2](#).

#### 10.3.4 Joists

In joists, at least one bottom bar should be continuous over the support or should be spliced there using the minimum lap splice length of [9.4.2](#), and at not continuous supports should be terminated with a standard hook. See [12.3.5.5](#).

### 10.4 One-way and two-way slabs and load path

#### 10.4.1 General

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

#### 10.4.2 One-way behaviour

A slab, solid or with joists, should be considered to work in one-way when either of the following conditions have been met:

- 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 in all edges, and the long slab span is greater than twice the short slab span;

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

### 10.4.3 Two-way behaviour

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

### 10.4.4 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 assigning tributary load to all slab-supporting elements, and also in obtaining the preliminary dimensions of the slab and the supporting elements. The load path and the tributary load on the supporting elements should be verified, and corrected as needed, during the design and dimensioning stage of each of the structural elements.

## 10.5 Minimum depth for elements of the floor system

### 10.5.1 General

The following minimum 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.

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

#### 10.5.2.1 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.

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

When the slab 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 given in [Table 9](#), where the span length,  $l$ , should be taken as the centre-to-centre distance between supports, except that when the span is less than 3 m, it should be permitted to take  $l$  as the clear span.

**Table 9 — 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	$\frac{l}{10}$

**10.5.2.3 Non-structural elements likely be damaged by large deflections**

When the slab 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 10](#), where the span length,  $l$ , should be taken as the centre-to-centre distance between supports, except that when the span is less than 3 m, it should be permitted to take  $l$  as the clear span.

**Table 10 — 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	$\frac{l}{7}$

**10.5.3 Girders, beams and one-way joists supporting the slab**

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

When the 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 given in [Table 11](#), where the span length,  $l$ , should be taken as the centre-to-centre distance between supports, except that for joists when the span is less than 3 m, it should be permitted to take  $l$  as the clear span.

**Table 11 — 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	$\frac{l}{8}$

**10.5.3.2 Non-structural elements likely be damaged by large deflections**

When the 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 12](#), where the span length,  $l$ , should be taken as the centre-to-centre distance between supports, except that for joists when the span is less than 3 m, it should be permitted to take  $l$  as the clear span.

**Table 12 — 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	$\frac{l}{5}$

**10.5.4 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 in all edges of the panel, should be as given in [Formula \(52\)](#), 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.

$$h = \frac{l_n}{30 + 3 \cdot \beta} \quad (52)$$

Where the span length,  $l_n$ , should be taken as the clear span in the long direction, measured face-to-face of the supporting beams, and  $\beta$  is the ratio of long clear span to short clear span of the slab panel. The

procedure for design of two-way slabs-on-girders of this document guide that the supporting girders or beams should have a depth not less than three times the slab thickness (see [11.8.1](#)).

## 10.6 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 [10.5](#). For beam and girders the initial trial width,  $b_w$ , should be taken as one half of the depth,  $h$ , of the element but not less than 200 mm, and for joists should be defined using the minimum width dimensions given in [10.1.4.1](#).

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.

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

### 11.1 General

The design of one-way and two-way solid slabs supported by girders, beams, or structural concrete walls in their edges should be performed employing the guides of [Clause 11](#). Guides for the top thin solid slab that span between joists are also included.

### 11.2 Design load definition

#### 11.2.1 Loads to be included

The design load for solid slabs supported on girders, beams, joists, or structural concrete walls should be established from the requirements of [Clause 8](#). 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) if the slab is part of the roof system, the appropriate values of roof live load, rain load and snow load, should be employed.

#### 11.2.2 Dead load and live load

The values of  $q_d$  for dead load and  $q_l$  for live load should be in  $N/m^2$ .  $q_d$  should include the selfweight of the solid slab, at 24  $N/m^2$  per mm of thickness, and the weight of the flat and standing non-structural elements also in  $N/m^2$ . Live load should be determined as guided by [8.1.4](#). If the slab is part of the roof system, the specified snow loads in [8.1.5](#) should be included, if appropriate.

#### 11.2.3 Factored design loads

The value of the factored design loads,  $q_u$  in  $N/m^2$ , should be the greater value obtained combining  $q_d$  and  $q_l$  using [Formula \(5\)](#) and [Formula \(6\)](#). If the slab is part of a roof system, [Formula \(7\)](#) and [Formula \(8\)](#) should also be investigated, choosing the greatest value of all four formulae.

## 11.3 Details of reinforcement

### 11.3.1 General

For the purposes of the present guidelines, the reinforcement of solid slabs-on-girders should be of the types described and should comply with the guides of [11.3.2](#) to [11.3.6.4](#).

### 11.3.2 Shrinkage and temperature reinforcement

#### 11.3.2.1 Description

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

#### 11.3.2.2 Location

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

#### 11.3.2.3 Minimum reinforcement area

Shrinkage and temperature reinforcement should comply with the minimum reinforcement steel ratio,  $\rho_t$ , of [9.5.2.1](#).

#### 11.3.2.4 Maximum and minimum reinforcement spacing

Shrinkage and temperature reinforcement should not be spaced further apart than guide by [9.3.20](#), nor should it be placed closer than guided by [9.3.15](#).

#### 11.3.2.5 Reinforcement splicing

It should be permitted to lap-splice shrinkage and temperature reinforcement at any location. The splice length should comply with [9.4.2](#).

#### 11.3.2.6 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.

### 11.3.3 Positive flexural reinforcement

#### 11.3.3.1 Description

Positive flexural reinforcement should be provided in the lower part of the slab section, as guided in [Clause 11](#), and should comply with the general guides of [11.3.3](#), and the particular guides for each slab type as set forth in [11.4](#) to [11.8](#).

#### 11.3.3.2 Location

Positive flexural reinforcement should be provided parallel to the short span in one-way solid slabs-on-girders, and in both directions in two-way-slabs. Positive flexural reinforcement should be located as close as concrete cover guides permit (see [9.3.10.1](#)) to the bottom surface of the slab. In two-way systems, the short span positive flexural reinforcement should be located under the long span positive flexural reinforcement. The amount of positive flexural reinforcement should be that required to resist the factored positive design moment at the section.

#### 11.3.3.3 Minimum reinforcement area

Positive flexural reinforcement should have an area at least equal to the area guide by [9.5.2.2](#).

#### 11.3.3.4 Maximum reinforcement area

Positive flexural reinforcement area should not exceed the values set forth in [9.5.2.3](#).

#### 11.3.3.5 Maximum and minimum reinforcement spacing

Positive flexural reinforcement should not be spaced further apart than required by [9.3.19](#) nor should it be placed closer than permitted by [9.3.15](#).

#### 11.3.3.6 Cut off points

It should be permitted to suspend at the locations indicated in [11.6](#) to [11.8](#) 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.

#### 11.3.3.7 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.

#### 11.3.3.8 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.

#### 11.3.3.9 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.

### 11.3.4 Negative flexural reinforcement

#### 11.3.4.1 Description

Negative flexural reinforcement should be provided in the upper part of the slab section, at edges and supports, in the amounts and lengths required in [Clause 11](#), and should comply with the general guides of [11.3.4](#), and the particular guides for each slab type as set forth in [11.4](#) to [11.8](#).

#### 11.3.4.2 Location

Negative flexural reinforcement should be provided perpendicular to edge and intermediate supporting girders, beams, and structural concrete walls. Negative flexural reinforcement should be located as close as concrete cover guides permit (see [9.3.10.1](#)) to the upper surface of the slab. In two-way systems the short span negative flexural reinforcement should be located above the long span negative flexural reinforcement. The amount of negative flexural reinforcement should be that required to resist the factored negative design moment at the section.

#### 11.3.4.3 Minimum reinforcement area

Negative flexural reinforcement should have an area at least equal to the area guide by [9.5.2.2](#).

**11.3.4.4 Maximum reinforcement area**

Negative flexural reinforcement area should not exceed the values set forth in [9.5.2.3](#).

**11.3.4.5 Maximum and minimum reinforcement spacing**

Negative flexural reinforcement should not be spaced further apart than guide by [9.3.19](#), nor should it be placed closer than permitted by [9.3.15](#).

**11.3.4.6 Cut off points**

It should be permitted to suspend all the negative flexural reinforcement, except for cantilevers, at the locations indicated in [11.6](#) to [11.8](#) for each slab type. Where adjacent spans are unequal, negative flexural reinforcement cut-off points should be based on the guides for the longer span.

**11.3.4.7 Reinforcement splicing**

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

**11.3.4.8 End anchorage of reinforcement**

Negative flexural reinforcement perpendicular to a discontinuous edge should be anchored with a standard hook into the edge girder, beam, or structural concrete wall that provides support at the edge, complying with the anchorage distance guide by [9.4.3](#). 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.

**11.3.5 Shear reinforcement**

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

**11.3.6 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 51](#)).

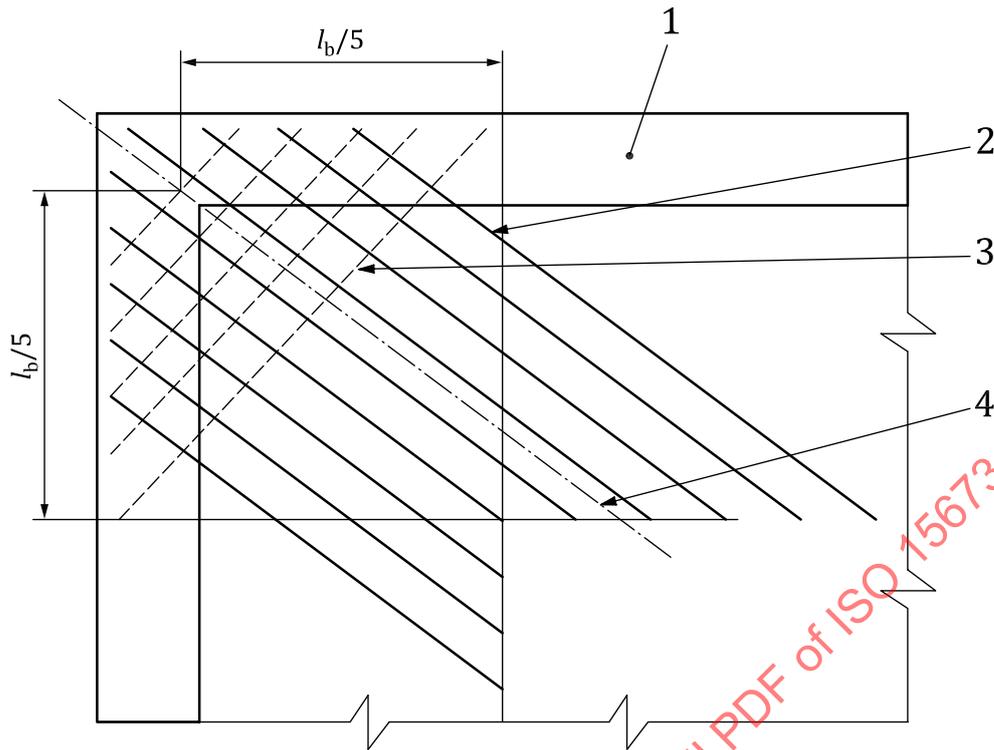
The amount of reinforcement, top and bottom, should be sufficient to resist a moment equal to the maximum positive factored design moment, per meter of width, in the slab panel, in accordance with [11.3.6.1](#) and [11.3.6.2](#).

**11.3.6.1 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.

**11.3.6.2 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**

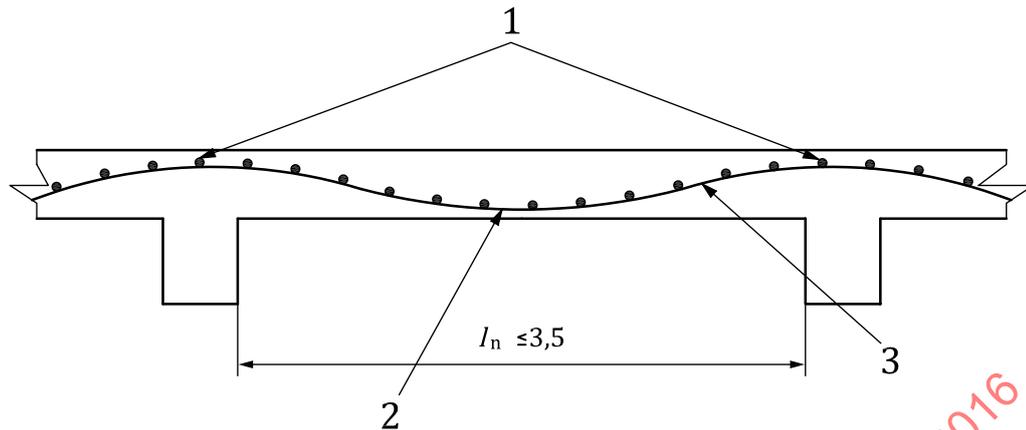
- 1 girder, beam or wall
- 2 corner top reinforcement
- 3 corner bottom reinforcement
- 4 slab panel diagonal

**Figure 51 — Special slab corner reinforcement**

**11.3.6.3 Welded-wire fabric used in short span slabs**

Welded-wire reinforcement 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 midspan, provided that such reinforcement is either continuous over or anchored at support. The area of longitudinal reinforcement should comply with the maximum required factored moment, either positive or the negative. See [Figure 52](#).

In two-way slabs, the wires of the welded-wire reinforcement 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.

**Key**

- 1 placed close to top of slab over supports
- 2 placed close to bottom of slab near midspan
- 3 welded - wire reinforcement as positive and negative reinforcement

**Figure 52 — Welded-wire reinforcement in short spans**

#### 11.3.6.4 Practical considerations for the value of $d_c$ and $d$ to employ in solid slabs

The determination of the distance from extreme tension fibre to centroid of tension reinforcement,  $[d_c]$ , should include the appropriate concrete cover from 9.3.10, the bar diameters employed, and the existence of reinforcement in the perpendicular direction placed between the reinforcement under study and the concrete surface. It should be permitted to use the following values of  $d_c$  to compute  $d$  as  $d = h - d_c$ . For one-way slabs and for the reinforcement in the short direction in two way slabs,  $d_c = 40$  mm for internal exposure and  $d_c = 60$  mm for external exposure. For reinforcement in the long direction of two-way slabs,  $d_c = 55$  mm for internal exposure and  $d_c = 75$  mm for external exposure.

### 11.4 Top thin solid slab that spans between joists

#### 11.4.1 Dimensional guidelines

The thin solid slab that spans between joists should comply with the minimum thickness guidelines of 10.5.2.1. The top thin slab should not be permitted to cantilever out of the edge joist (see 10.1.4.1).

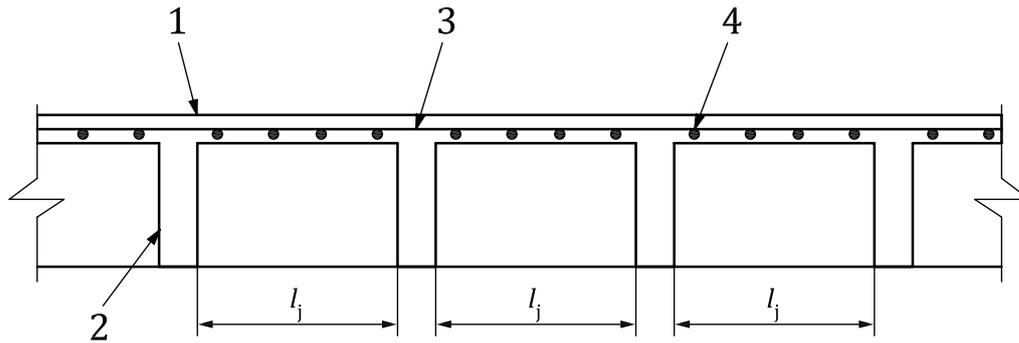
#### 11.4.2 Factored bending moment

The factored bending moment,  $M_u$ , in  $N \cdot m/m$ , for negative and positive bending moment in the thin slab that spans between joist in joist construction should be calculated using Formula (53), where  $l_j$  is the clear spacing between joists in m and  $q_u$  should be employed in  $N/m^2$ . See Figure 53.

$$M_u^+ = M_u^- = \frac{q_u \cdot l_j^2}{12} \quad (53)$$

#### 11.4.3 Reinforcement

The flexural reinforcement ratio,  $\rho$ , perpendicular to the joist direction should be determined employing Formula (35) or Formula (36), with the value of  $M_u$  obtained from Formula (53), converted to  $N \cdot mm$  ( $1 N \cdot m/m = 10^3 \cdot N \cdot mm/m$ ), using  $d$  in mm as one-half the thickness of the thin slab, and  $b = 1\,000$  mm.  $\rho$  should be made greater than or equal to the shrinkage and temperature ratio prescribed in 9.5.2.1. See Figure 53. The flexural reinforcing bar separations should meet the guides of 9.3.19. The reinforcement parallel to the joist direction should meet the guides of 11.3.2.



**Key**

- 1 top thin slab
- 2 joist
- 3 negative and positive flexural reinforcement
- 4 shrinkage and temperature reinforcement

**Figure 53 — Reinforcement of the thin solid slab that span between joists**

**11.4.4 Shear strength verification**

The factored shear  $V_u$ , in N/m, for the thin slab that span between joists in joist construction should be calculated using [Formula \(54\)](#), where  $l_j$  is the clear spacing between joists in m and  $q_u$  should be employed in N/m<sup>2</sup>. See [Figure 53](#).

$$V_u = \frac{q_u \cdot l_j}{2} \tag{54}$$

The design shear strength  $\phi \cdot V_n$ , in N/m, should be calculated using [Formula \(45\)](#), with  $d$  as one-half the thickness of the thin slab, in mm, and  $b_w = b = 1\,000$  mm. [Formula \(43\)](#) should be complied with.

**11.4.5 Calculation of the reactions on the joists**

Factored uniformly distributed reaction on the supporting joists  $r_u$ , in N/m, should be the value obtained from [Formula \(55\)](#), where  $V_u$  is the factored shear from [11.4.4](#), in N/m,  $l$  is the centre-to-centre spacing of the joist, in m, and  $l_j$  is the clear spacing between joists, also in m.

$$r_u = \frac{2 \cdot V_u \cdot l}{l_j} \tag{55}$$

**11.5 Cantilevers of slabs supported on girders, beams or walls**

**11.5.1 Dimensional guidelines**

Solid slab cantilevers spanning out of the edge girder, beam or structural concrete wall, should comply with the minimum thickness guidelines of [10.5.2](#). The cantilever span should not exceed the limits of [6.1](#). No openings for ducts or shafts should be permitted in the internal one-half span of the cantilever. It should be permitted for the slab to cantilever in two directions at corners, with the same limitations for single cantilevers. The thin top slab that spans between joists should not cantilever out of the edge joist.

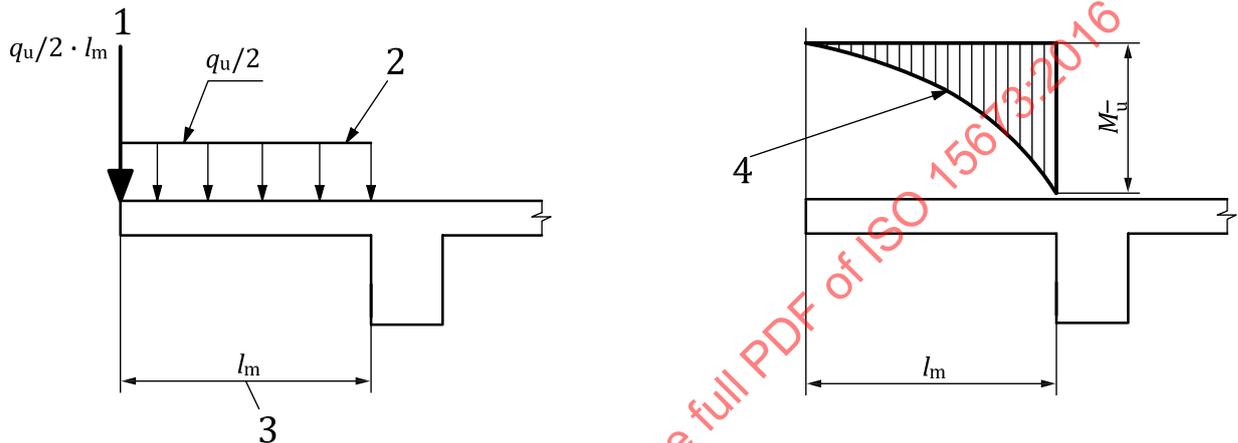
**11.5.2 Factored negative bending moment**

The factored negative bending moment,  $M_u$ , for slab cantilevers that span out of the edge supporting girders, beams or structural concrete walls, should be calculated supposing that one-half of the distributed factored load,  $q_u$ , acts as a concentrated load at the tip of the cantilever, and the other one-

half acts as uniformly distributed load over the full span, using [Formula \(56\)](#), but it should not be less than the factored negative bending moment of the first interior span at the exterior supporting girder, beam or structural concrete wall, nor less than one-third of the positive bending moment, in the same direction, of the first interior span. See [Figure 54](#).

$$M_u^- = \frac{3 \cdot q_u \cdot l_m^2}{4} \quad (56)$$

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



#### Key

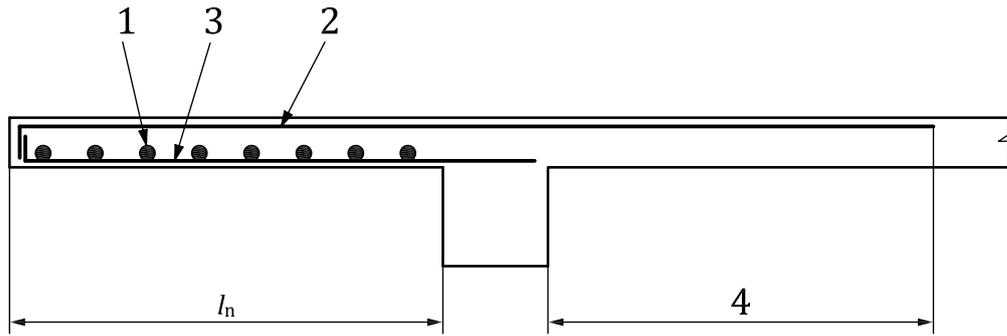
- 1 concentrated load
- 2 uniform load
- 3 cantilever clear span
- 4 moment diagram

Figure 54 — Calculation of negative moment at slab cantilevers

### 11.5.3 Reinforcement

#### 11.5.3.1 Negative flexural reinforcement

The negative flexural reinforcement ratio,  $\rho$ , in the direction of the cantilever should be determined employing [Formula 35](#) or [Formula 36](#), using the value of  $M_u$  obtained from [Formula 56](#), converted to N · mm/m (1 N · m/m = 10<sup>3</sup> · N · mm/m), with the appropriate value of  $d$  in mm, and  $b = 1\,000$  mm. The negative flexural reinforcement should comply with [11.3.4](#). This reinforcement should be anchored in the first interior span not less than  $l_d$  for the reinforcing bar (see [9.4.1](#)), nor the distance required for the negative reinforcement of the interior slab panel at the edge support. See [Figure 55](#).



**Key**

- 1 shrinkage and temperature reinforcement
- 2 negative cantilever reinforcement
- 3 minimum positive reinforcement
- 4 distance as required for negative reinforcement of first interior span, but not less than  $l_n$  bar

**Figure 55 — Reinforcement for slab cantilevers**

**11.5.3.2 Positive flexural reinforcement**

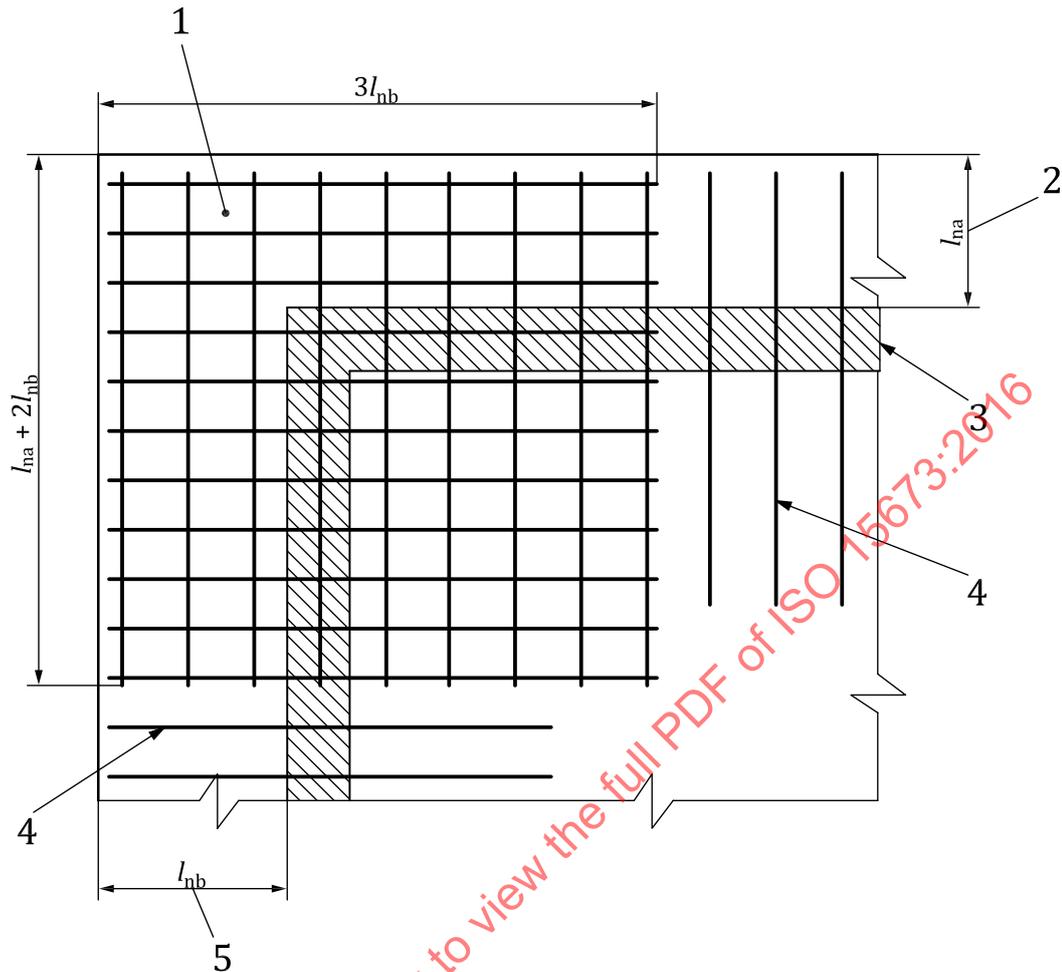
A minimum amount of positive flexural reinforcement with an area greater than or equal to the shrinkage and temperature reinforcement, complying with 11.3.2 should be provided in the direction of the cantilever. See Figure 55.

**11.5.3.3 Shrinkage and temperature reinforcement**

Reinforcement parallel to the edge of the cantilever complying with 11.3.1 should be provided. See Figure 55.

**11.5.3.4 Reinforcement of two-way cantilevers**

At corners where the slab cantilevers in two-directions, the negative flexural reinforcement should be calculated for the larger span cantilever, using 11.5.3.1. This reinforcement should be placed in both directions (see Figure 56), for a distance measured from the corner equal to the cantilever clear span plus two times the larger cantilever span, but not less than the distance required for the negative flexural reinforcement of the first interior span plus the cantilever span. Reinforcement as guided by 11.5.3.4 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 56 — Negative flexural reinforcement in two-way slab cantilevers**

#### 11.5.4 Shear verification

The factored shear  $V_u$ , in N/m, at the support of cantilever slabs should be calculated using [Formula \(57\)](#), where  $l_m$  should be the clear span of the cantilever in m, and  $q_u$  should be employed in N/m<sup>2</sup>.

$$V_u = q_u \cdot l_m \quad (57)$$

For two-way cantilevers, the value of  $V_u$  should be taken as twice the value obtained from [Formula \(57\)](#) using the larger cantilever span.

The design shear strength  $\phi \cdot V_{tr}$ , in N/m, should be calculated using [Formula \(45\)](#) with the appropriate value of  $d$  in mm, and  $b = 1\,000$  mm. [Formula \(43\)](#) should be complied with.

**11.5.5 Calculation of reactions on the supports**

Uniformly distributed factored reaction on the support of the cantilever  $r_u$ , in N/m, should be the value obtained from [Formula \(58\)](#):

$$r_u = \frac{V_u \cdot l}{l_m} \tag{58}$$

where

$V_u$  is the factored shear from [11.5.4](#), in N/m;

$l$  is the span of the canteliver measured from the centreline of the supporting element, in m;

$l_m$  is the clear span of the canteliver, in m.

Where two-way cantilevers exists, it should be permitted in the calculation of the value of  $R_u$  to use [Formula \(58\)](#) employing the value of  $V_u$  obtained from [Formula \(57\)](#) for the larger cantilever span, without doubling it.

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

**11.6.1 Dimensional guidelines**

One-way one-span solid slabs should comply with the minimum thickness guidelines of [10.5.2](#). In addition to the appropriate guidelines of [11.6](#), these slabs should comply with the general dimensional guidelines set forth in [6.1](#), and the particular guidelines of [10.1.2](#) for slab-on-girder systems.

**11.6.2 Factored bending moment**

The factored positive and negative bending moment,  $M_u$ , in N · m/m, for one-span one-way slabs should be calculated using the formulae given in [Table 13](#).

**Table 13 — Factored flexural moment for one-way, one-span slabs**

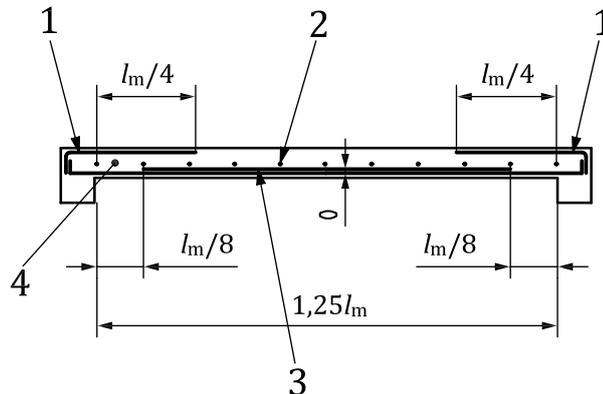
<p><b>Positive moment:</b></p> $M_u^+ = \frac{q_u \cdot l_m^2}{8}$	Formula (59)
<p><b>Negative moment at supports:</b></p> $M_u^- = \frac{q_u \cdot l_m^2}{24}$	Formula (60)

**11.6.3 Longitudinal flexural reinforcement**

**11.6.3.1 Positive flexural reinforcement**

The positive reinforcement ratio,  $\rho$ , in the direction of the span  $l_m$ , should be determined employing [Formula \(35\)](#) or [Formula \(36\)](#), with the value of  $M_u^+$  obtained from Formula (59) converted to N · mm (1 N · m/m = 10<sup>3</sup> · N · mm/m), using  $d$  in mm, and  $b = 1\,000$  mm. This reinforcement should comply with the guides of [11.3.3](#). 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  $l_m/8$  measured from the internal face of the supports into the span. See [Figure 57](#).



#### Key

- 1 negative flexural reinforcement
- 2 shrinkage and temperature reinforcement
- 3 positive flexural reinforcement
- 4 positive flexural reinforcement suspension, only if slab built monolithically with support at least three times deeper than slab

**Figure 57 — Reinforcement for one-span one-way slabs**

#### 11.6.3.2 Negative flexural reinforcement

The negative flexural reinforcement ratio,  $\rho$  in the direction of the span  $l_m$ , should be determined employing [Formula \(35\)](#) or [Formula \(36\)](#), with the value of  $M_u^-$  obtained from [Formula \(60\)](#) converted to  $N \cdot mm$  ( $1 N \cdot m/m = 10^3 \cdot N \cdot mm/m$ ), using  $d$  in  $mm$ , and  $b = 1\ 000$   $mm$ . This reinforcement should comply with [11.3.4](#). At a distance equal to  $l_m/4$  measured from the internal face of the supports into the span, all the negative flexural reinforcement should be permitted to be suspended. See [Figure 57](#).

#### 11.6.3.3 Shrinkage and temperature reinforcement

The reinforcement perpendicular to the span should meet the guides for shrinkage and temperature reinforcement of [11.3.2](#). See [Figure 57](#).

#### 11.6.4 Shear verification

The factored shear,  $V_u$ , in  $N/m$ , for the one-span one-way slab should be calculated at the face of the supports using [Formula \(61\)](#), where  $l_m$  is the clear span in  $m$  and  $q_u$  should be employed in  $N/m^2$ . See [Figure 57](#).

$$V_u = \frac{q_u \cdot l_m}{2} \quad (61)$$

The design shear strength,  $\phi \cdot V_n$ , in  $N/m$ , should be calculated using [Formula \(45\)](#), with  $d$  in  $mm$ , and  $b_w = b = 1\ 000$   $mm$ . [Formula \(43\)](#) should be complied with.

### 11.6.5 Calculation of the reactions on the supports

Uniformly distributed factored reaction on the supports of one-way one-span slabs,  $r_u$ , in N/m, should be the value obtained from [Formula \(62\)](#) plus the uniformly distributed reaction from any cantilever spanning from that support:

$$r_u = \frac{V_u \cdot l}{l_m} \quad (62)$$

where

$V_u$  is the factored shear from [11.6.4](#), in N/m;

$l$  is the centre-to-centre span of the slab, in m;

$l_m$  is the clear span of the canteliver, in m.

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

### 11.7.1 Dimensional guidelines

One-way solid slabs with two or more spans should comply with the minimum thickness guidelines of [10.5.2](#). In addition to the appropriate guidelines of [Clause 11](#), slabs should comply with the general dimensional guidelines in [6.1](#), and the particular guides of [10.1.2](#) for slab-on-girder systems.

The following restrictions should be in effect for slabs designed under [11.7](#):

- 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) loads are uniformly distributed;
- d) unit live load,  $q_l$ , does not exceed three times unit dead load,  $q_d$ ;
- e) for negative moment evaluation at internal supports,  $l_m$  should correspond to the largest of the neighbouring spans.

### 11.7.2 Factored bending moment

The factored positive and negative bending moment,  $M_u$ , in N · m/m, for one-way slabs should be calculated using the [Formula \(63\)](#) to [Formula \(68\)](#) for slabs with two or more spans.

Positive moment:

— at end spans:

$$M_u^+ = \frac{q_u \cdot l_n^2}{11} \quad (63)$$

— at interior spans:

$$M_u^+ = \frac{q_u \cdot l_n^2}{16} \quad (64)$$

Negative moment at supports:

— at interior face of external support:

$$M_u^- = \frac{q_u \cdot l_n^2}{24} \quad (65)$$

— at exterior face of first internal support, only two spans:

$$M_u^- = \frac{q_u \cdot l_n^2}{9} \quad (66)$$

— at faces of internal supports, more than two spans:

$$M_u^- = \frac{q_u \cdot l_n^2}{10} \quad (67)$$

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

$$M_u^- = \frac{q_u \cdot l_n^2}{12} \quad (68)$$

### 11.7.3 Longitudinal flexural reinforcement

#### 11.7.3.1 Positive flexural reinforcement

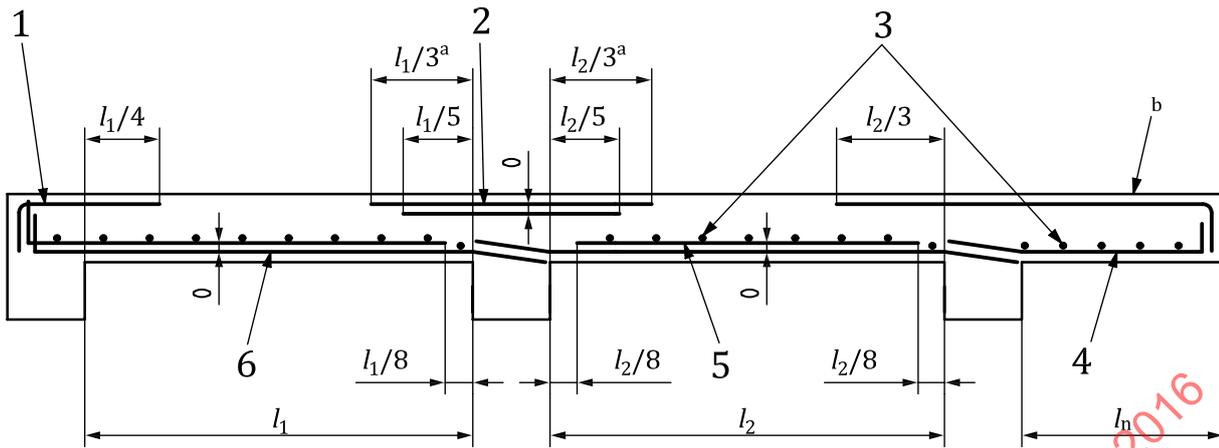
The positive reinforcement ratio,  $\rho$ , in the direction of the span  $l_m$ , should be determined employing [Formula \(35\)](#) or [Formula \(36\)](#), with the appropriate value of  $M_u^+$  obtained from [Formula \(63\)](#) or [Formula \(64\)](#), converted to  $N \cdot mm/m$  ( $1 N \cdot m/m = 10^3 \cdot N \cdot mm/m$ ), using  $d$  in mm, and  $b = 1\,000$  mm. This reinforcement should comply with [11.3.3](#). At internal supports, at a distance equal to  $l_m/8$  measured from the face of the supports into the span, up to one-half of the positive flexural reinforcement should be permitted to be suspended. See [Figure 58](#).

#### 11.7.3.2 Negative flexural reinforcement

The negative flexural reinforcement ratio,  $\rho$ , in the direction of the span,  $l_m$ , should be determined employing [Formula \(35\)](#) or [Formula \(36\)](#), with the appropriate value of  $M_u^-$  obtained, from [Formula \(65\)](#) to [Formula \(68\)](#), converted to  $N \cdot mm/m$  ( $1 N \cdot m/m = 10^3 \cdot N \cdot mm/m$ ), using  $d$  in mm, and  $b = 1\,000$  mm. This reinforcement should comply with [11.3.4](#). At internal supports, at a distance equal to  $l_m/3$ , where  $l_m$  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  $l_m/4$  measured from the internal face of the support into the span, all the negative flexural reinforcement should be permitted to be suspended. See [Figure 58](#) and [Figure 59](#).

#### 11.7.3.3 Shrinkage and temperature reinforcement

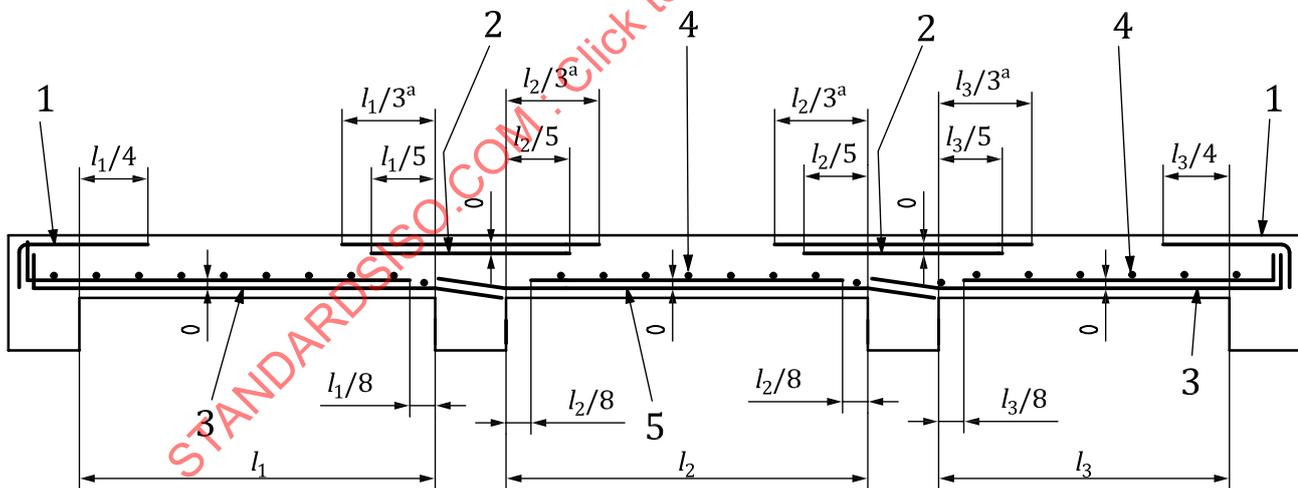
The reinforcement perpendicular to the span should meet the guides for shrinkage and temperature reinforcement of [11.3.2](#). See [Figure 58](#) and [Figure 59](#).



**Key**

- 1 negative reinforcement at interior face of external support
- 2 negative reinforcement 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 greater of the two neighbouring spans.
- b Greater negative reinforcement from that required for the external support or for the cantilever.

**Figure 58 — Reinforcement for two-span one-way slabs supported by girders, beams, or structural concrete walls**



**Key**

- 1 negative reinforcement at interior face of external support
- 2 negative reinforcement at faces of internal support more than two spans
- 3 positive reinforcement end span
- 4 shrinkage and temperature reinforcement
- 5 positive reinforcement interior span
- a Negative reinforcement cut-off points should be based upon greater of the two neighbouring spans.

**Figure 59 — Reinforcement one-way slabs supported by girders, beams, or structural concrete walls, with three or more spans**

#### 11.7.4 Shear verification

The factored shear,  $V_u$ , in N/m, for the slab should be calculated at the faces of all supports using [Formula \(69\)](#) and [Formula \(70\)](#), where  $l_m$  is the clear span in m and  $q_u$  should be employed in N/m<sup>2</sup>. See [Figure 58](#) and [Figure 59](#).

At exterior face of first interior support:

$$V_u = 1,15 \cdot \frac{q_u \cdot l_m}{2} \quad (69)$$

At faces of all other supports

$$V_u = \frac{q_u \cdot l_m}{2} \quad (70)$$

The design shear strength,  $\phi \cdot V_n$ , in N/m, should be calculated using [Formula \(45\)](#), with  $d$  in mm, and  $b_w = b = 1\,000$  mm. [Formula \(43\)](#) should be complied with at all faces of supports.

#### 11.7.5 Calculation of reactions on the supports

Uniformly distributed factored reaction on the support contributed by any span of one-way slabs,  $r_u$ , in N/m, should be the value obtained from [Formula \(71\)](#):

$$r_u = \frac{V_u \cdot l}{l_m} \quad (71)$$

where

$V_u$  is the factored shear from [11.7.4](#), in N/m;

$l$  is the centre-to-centre span of the slab, in m;

$l_m$  is the clear span of the cantilever, in m.

Total factored uniformly distributed reaction on the external supports should be equal to the value of the factored uniformly distributed reaction from the span,  $r_u$ , obtained from [Formula \(71\)](#) at the support, plus the uniformly distributed reaction of any cantilever spanning from that support. Total factored uniformly distributed reactions on internal supports should be the sum of the factored uniformly distributed reactions,  $r_u$ , obtained using [Formula \(71\)](#) for both neighbouring spans at that support.

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

#### 11.8.1 Dimensional guides

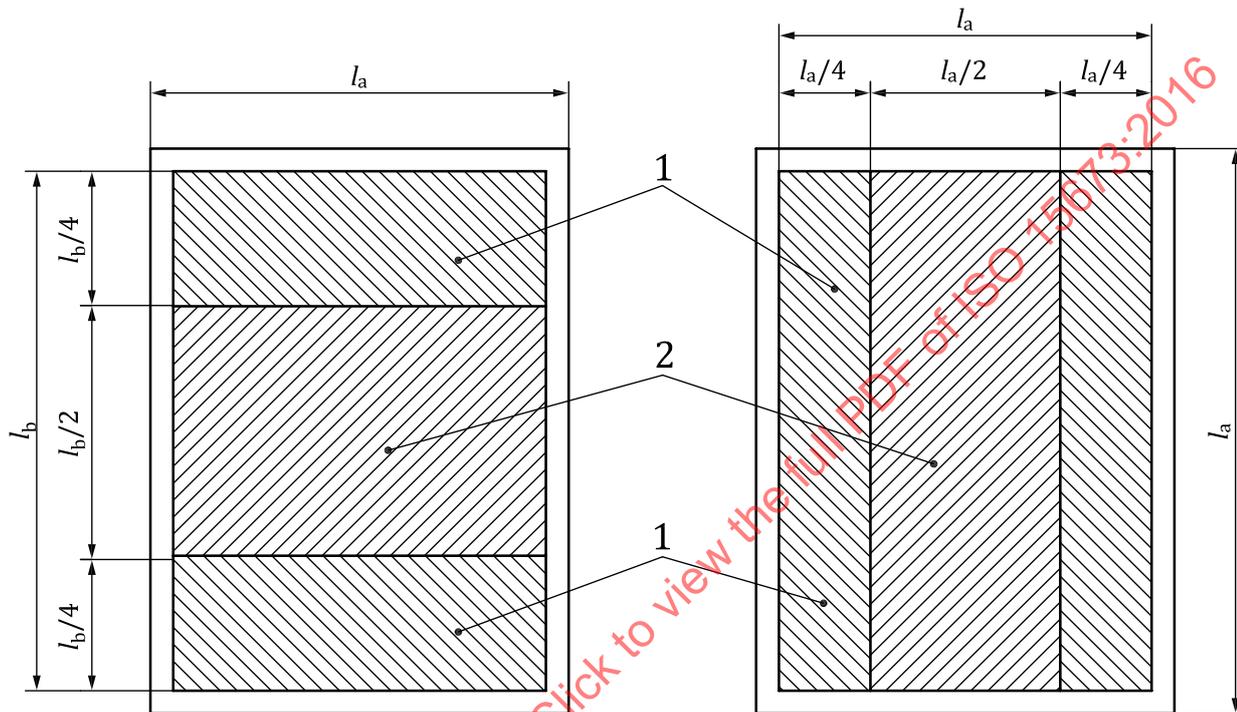
Two-way solid slabs having girders, beams, or structural concrete walls in all edges should comply with the minimum thickness guides of [10.5.4](#). In addition to the appropriate guides of [Clause 11](#), two-way slabs should comply with the general dimensional guides in [6.1](#), and the particular guides of [10.1.2](#) for slab-on-girder systems.

The following restrictions should be in effect for the use of the procedure of [11.8](#):

- 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 three 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 while the border regions should be two one-quarter regions adjacent on both sides of the central region. See [Figure 60](#).



**Key**

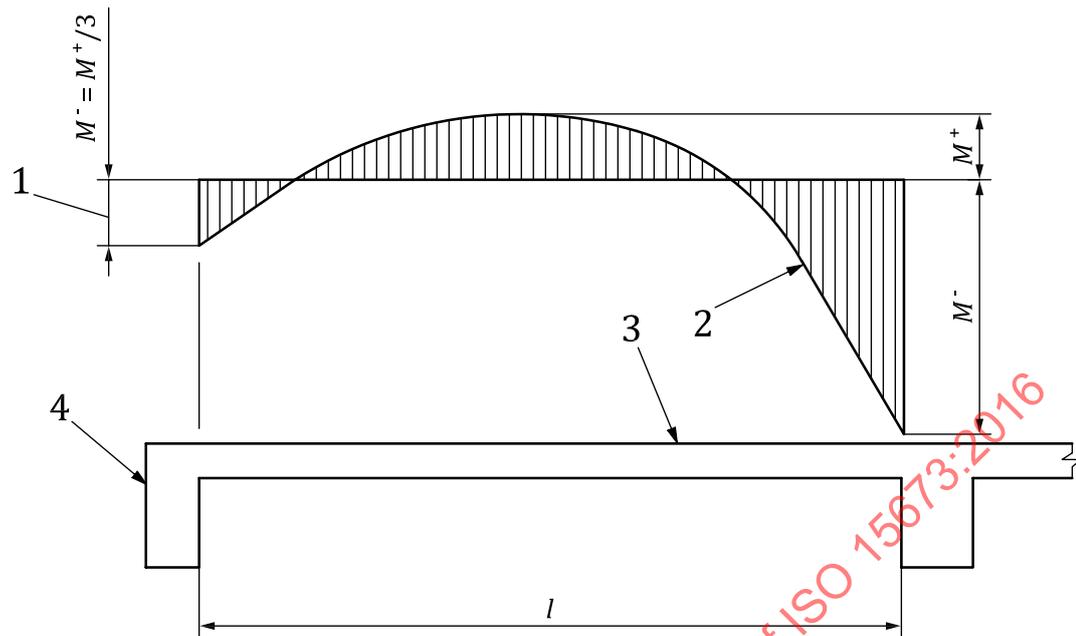
- 1 border region
- 2 central region

**Figure 60 — Central and border regions for two-way slabs supported on girders, beams, or structural concrete walls**

**11.8.2 Factored flexural moment**

The factored positive and negative moment,  $M_u$ , for two-way solid slabs should be calculated using the procedure in [11.8.2](#). The negative and positive factored flexural moment for the central region of the panel, in each direction, should be calculated using the formulae given in [Table 14](#) for central panels, in [Table 15](#) for edge panels with the short span at the edge, in [Table 16](#) for edge panels with the long span at the edge, and in [Table 17](#) for corner panels. In each table, the values of the factored flexural moments should be obtained for the appropriate ratio,  $\beta$ , of long clear span,  $l_b$ , to short clear span,  $l_a$ , and the corresponding edge continuity conditions.

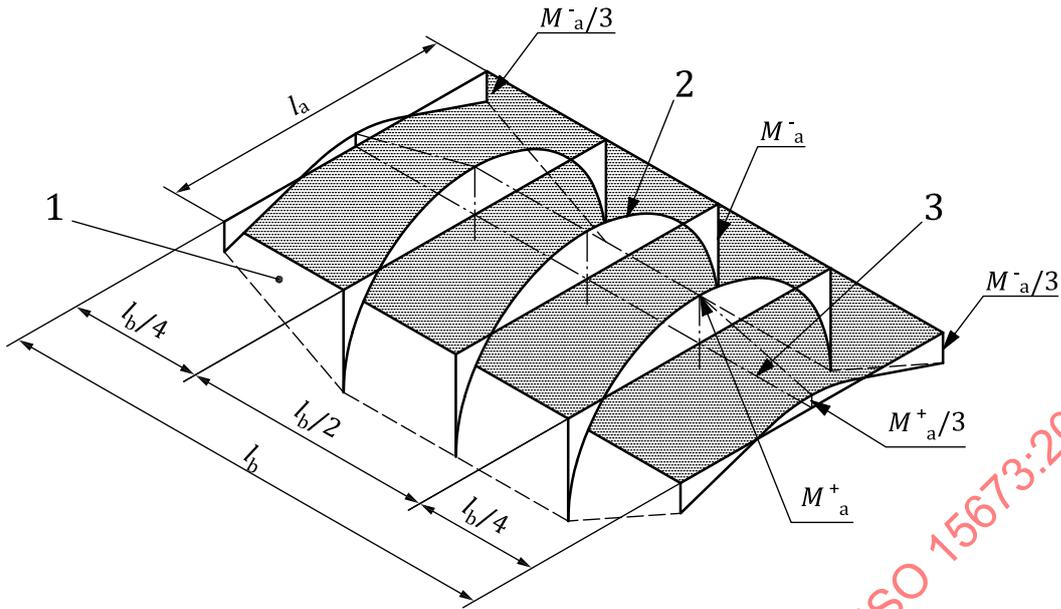
The negative moment at discontinuous edges should be one-third of the positive moment in the same direction. See [Figure 61](#).

**Key**

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

**Figure 61 — Negative moment at discontinuous edges of two-way solid slabs-on-girders**

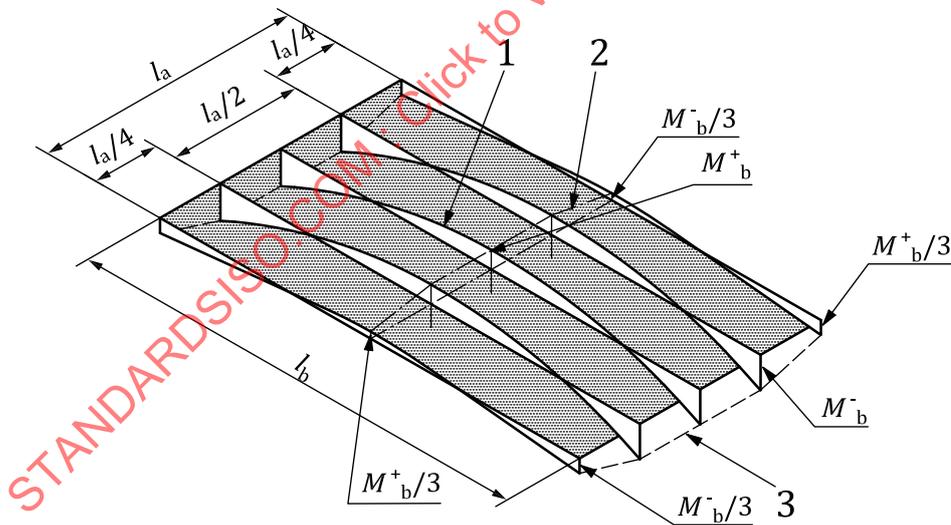
It should be permitted to decrease the moment strength values at the edge of the central regions to one-third of this value at the edge of the panel, as shown in [Figure 62](#) for moments in the short directions, and in [Figure 63](#) for moments in the long direction.



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



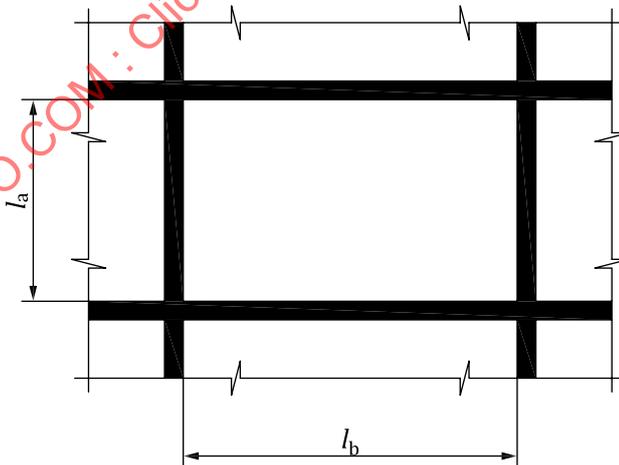
**Key**

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

**Table 14 — Central panel of two-way slabs supported on girders, beams or structural concrete walls**

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



**Figure 64 — Central panel of two-way slabs supported on girders, beams or structural concrete walls**

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

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

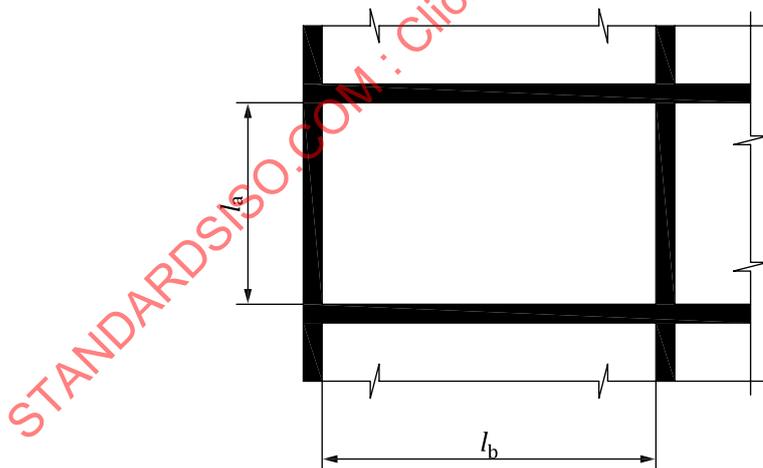


Figure 65 — Edge panel with  $l_a$  parallel to edge, of two-way slabs supported on girders, beams or walls

Table 16 — Edge panel with  $l_b$  parallel to edge, of two-way slabs supported on girders, beams or walls

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

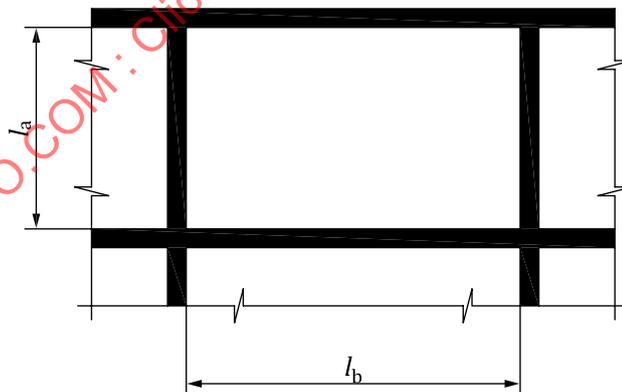


Figure 66 — Edge panel with  $l_b$  parallel to edge, of two-way slabs supported on girders, beams or walls

Table 17 — Corner panel of two-way slabs supported on girders, beams or structural concrete walls

$\beta = l_b/l_a$	Short direction ( $l_a$ )			Long direction ( $l_b$ )		
Panel span ratio	Negative moment	Positive moment	Load fraction	Negative moment	Positive moment	Load fraction
1,0	$M_a^- = \frac{q_u l_b^2}{20}$	$M_a^+ = \frac{q_u l_b^2}{31}$	$\alpha_a = 0,50$	$M_b^- = \frac{q_u l_b^2}{20}$	$M_b^+ = \frac{q_u l_b^2}{31}$	$\alpha_b = 0,50$
1,2	$M_a^- = \frac{q_u l_b^2}{15}$	$M_a^+ = \frac{q_u l_b^2}{23}$	$\alpha_a = 0,67$	$M_b^- = \frac{q_u l_b^2}{30}$	$M_b^+ = \frac{q_u l_b^2}{45}$	$\alpha_b = 0,33$
1,4	$M_a^- = \frac{q_u l_b^2}{13}$	$M_a^+ = \frac{q_u l_b^2}{19}$	$\alpha_a = 0,80$	$M_b^- = \frac{q_u l_b^2}{50}$	$M_b^+ = \frac{q_u l_b^2}{70}$	$\alpha_b = 0,20$
1,6	$M_a^- = \frac{q_u l_b^2}{11}$	$M_a^+ = \frac{q_u l_b^2}{16}$	$\alpha_a = 0,87$	$M_b^- = \frac{q_u l_b^2}{75}$	$M_b^+ = \frac{q_u l_b^2}{100}$	$\alpha_b = 0,13$
1,8	$M_a^- = \frac{q_u l_b^2}{11}$	$M_a^+ = \frac{q_u l_b^2}{15}$	$\alpha_a = 0,92$	$M_b^- = \frac{q_u l_b^2}{120}$	$M_b^+ = \frac{q_u l_b^2}{150}$	$\alpha_b = 0,08$
2,0	$M_a^- = \frac{q_u l_b^2}{10}$	$M_a^+ = \frac{q_u l_b^2}{14}$	$\alpha_a = 0,96$	$M_b^- = \frac{q_u l_b^2}{165}$	$M_b^+ = \frac{q_u l_b^2}{200}$	$\alpha_b = 0,04$
>2,0	$M_a^- = \frac{q_u l_b^2}{9}$	$M_a^+ = \frac{q_u l_b^2}{11}$	$\alpha_a = 1,00$	Temperature and shrinkage reinforcement		$\alpha_b = 0,00$

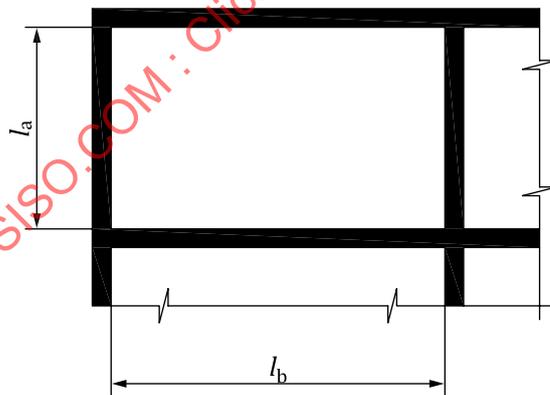


Figure 67 — Corner panel of two-way slabs supported on girders, beams or structural concrete walls

### 11.8.3 Longitudinal flexural reinforcement

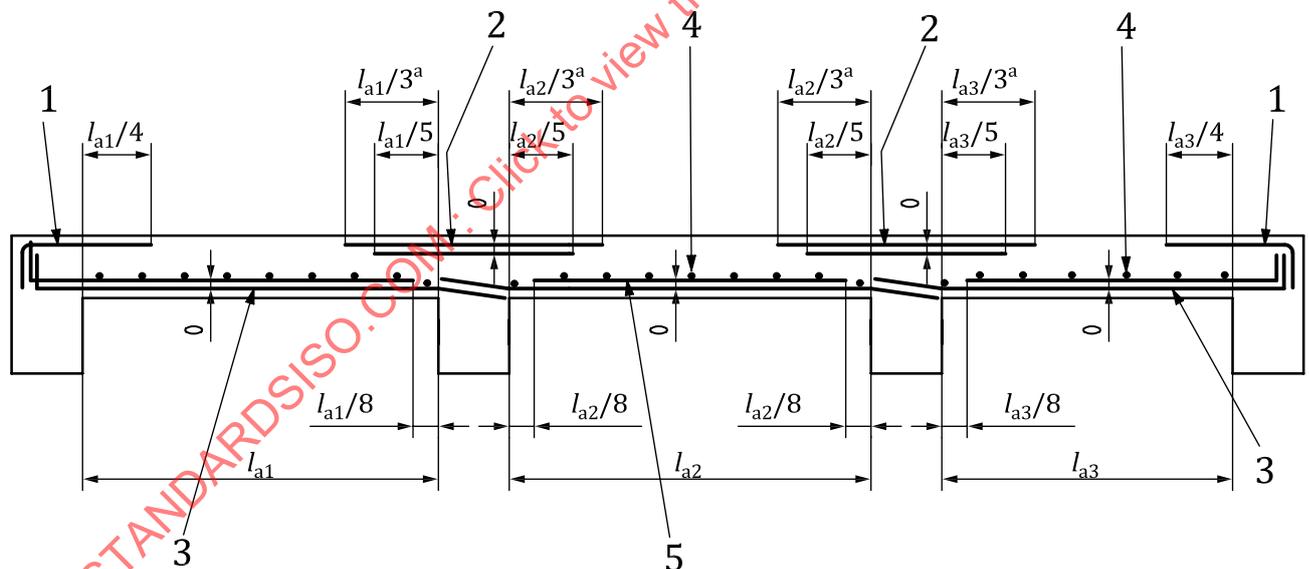
#### 11.8.3.1 Positive flexural reinforcement

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 obtained from the appropriate formula of Table 14 to Table 17 (see also Figure 64 to Figure 67). The positive flexural steel ratio,  $\rho$ , for reinforcement parallel to the short span  $l_a$ , or the long span  $l_b$ , should be determined using Formula (35) or Formula (36) for the corresponding value of  $M_a^+$  or  $M_b^+$  in N · m/m.  $M_a^+$  and  $M_b^+$  should be converted to N · mm/m

( $1 \text{ N} \cdot \text{m}/\text{m} = 10^3 \cdot \text{N} \cdot \text{mm}/\text{m}$ ), using  $d$  in mm and  $b = 1\,000 \text{ mm}$ . All guides for positive flexural reinforcement of 11.3.3 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 gradually decrease 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 68.

### 11.8.3.2 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 obtained from the appropriate formulae of Table 14 to Table 17 (see also Figure 64 to Figure 67). The negative flexural steel ratio,  $\rho$ , for reinforcement parallel to the short span  $l_a$ , or the long span  $l_b$ , should be determined using Formula (35) or Formula (36) for the corresponding value of  $M_a^-$  or  $M_b^-$  in  $\text{N} \cdot \text{m}/\text{m}$ .  $M_a^-$  and  $M_b^-$  should be converted to  $\text{N} \cdot \text{mm}/\text{m}$  ( $1 \text{ N} \cdot \text{m}/\text{m} = 10^3 \cdot \text{N} \cdot \text{mm}/\text{m}$ ), using  $d$  in mm and  $b = 1\,000 \text{ mm}$ . All guides for negative flexural reinforcement of 11.3.4 should be met. At a distance equal to  $l_a/5$  or  $l_b/5$  measured from the face of any interior support, it should be permitted to suspend up to one-half of the negative flexural reinforcement required at the support, and a distance equal to  $l_a/3$  or  $l_b/3$  measured from the face of any interior support, it should be permitted to suspend all the negative flexural reinforcement required at the support of the corresponding span. It should be permitted to gradually decrease 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 68.



#### Key

- 1 negative reinforcement at interior face of external support without continuity
- 2 negative reinforcement at faces of internal supports more than two spans
- 3 positive reinforcement end span
- 4 positive reinforcement in the other direction (negative reinforcement not shown)
- 5 positive reinforcement interior span
- a Negative reinforcement cut-off points should be based upon greater of the two neighbouring spans.

**Figure 68 — Reinforcement for two-way slabs supported by girders, beams, or structural concrete walls**

11.8.4 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,  $\alpha_a$  and  $\alpha_b$ , travelling in the short and long directions, respectively, as given in Table 14 to Table 17 (see also Figure 64 to Figure 67) for the corresponding panel edge conditions and panel span ratio,  $\beta$ . See Figure 69. The factored shear should not be less than the factored shear caused by factored design load,  $q_u$  (in N/m<sup>2</sup>) 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 70.

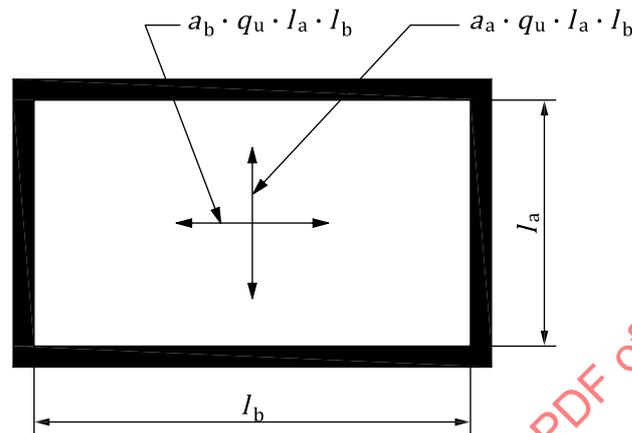


Figure 69 — Fraction of the total load in the panel travelling in each direction in two-way slabs supported on girders, beams or structural concrete walls

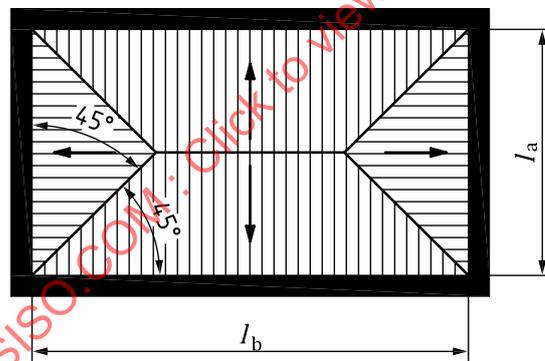


Figure 70 — Tributary areas for minimum shear at the supports of two-way slabs supported on girders, beams or structural concrete walls

The factored shear,  $V_u$ , in N/m, should not be less than the value obtained from Formula (72) at the short span supporting element and from Formula (66) at the long span supporting element.

$$V_u = \frac{\alpha_a \cdot q_u \cdot l_b}{2} \geq \frac{q_u \cdot l_a}{4} \tag{72}$$

$$V_u = \frac{\alpha_b \cdot q_u \cdot l_a}{2} \geq q_u \cdot \left[ \frac{l_a}{2} - \frac{l_a^2}{4 \cdot l_b} \right] \tag{73}$$

The design shear strength,  $\phi \cdot V_n$ , should be calculated employing Formula (45) with  $b_w = b = 1\,000$  mm. Formula (43) should be complied with. This should be accomplished by using an effective depth  $d$ , in

mm, for the slab greater than or equal to the largest value obtained from [Formula \(74\)](#), [Formula \(75\)](#), and [Formula \(76\)](#).

$$d \geq \frac{3 \cdot q_u \cdot \alpha_a \cdot l_a}{\phi \cdot \sqrt{f'_c}} \quad (74)$$

$$d \geq \frac{3 \cdot q_u \cdot \alpha_b \cdot l_b}{\phi \cdot \sqrt{f'_c}} \quad (75)$$

$$d \geq \frac{3 \cdot q_u \cdot l_a}{2 \cdot \phi \cdot \sqrt{f'_c}} \quad (76)$$

In [Formula \(74\)](#) to [Formula \(76\)](#),  $q_u$  should be used in N/m<sup>2</sup>,  $l_a$  and  $l_b$  in m,  $f'_c$  in MPa, and  $\phi = [0,85]$ ; see [6.3.3 d](#)).

### 11.8.5 Calculation of the reactions on the supports

Uniformly distributed factored reaction on the support contributed by any panel of two-way slabs,  $r_u$  in N/m, in the short direction should be the value obtained from [Formula \(77\)](#) and in the long direction the value obtained from [Formula \(78\)](#).

$$r_u = \frac{V_u \cdot l}{l_a} \quad (77)$$

$$r_u = \frac{V_u \cdot l}{l_b} \quad (78)$$

where

$V_u$  is the corresponding factored shear from [Formula \(72\)](#) and [Formula \(73\)](#), in N/m;

$l$  is the centre-to-centre span in that direction, in m;

$l_a$  and  $l_b$  are the corresponding clear spans, in m.

Total factored uniformly distributed reaction on the external supports of edge panels should be equal to the value of the factored uniformly distributed reaction from the panel,  $r_u$ , at the edge support, obtained from [Formula \(77\)](#) or [Formula \(78\)](#) plus the uniformly distributed reaction of any cantilever spanning from that support. Total factored uniformly distributed reactions on internal supports should be the sum of the factored uniformly distributed reactions,  $r_u$ , obtained using either [Formula \(77\)](#) or [Formula \(78\)](#), as appropriate, for both neighbouring spans at that support.

## 12 Girders, beams and joists

### 12.1 General

The design of girders, beams and joists should be performed employing the requirements of [Clause 12](#). The guides 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.

## 12.2 Design load definition

### 12.2.1 Loads to be included

The design load for girders, beams, and joists should be established from the guides of [Clause 8](#). The gravity loads that should be included in the design of the element should be divided in tributary loads from other structural elements supported by the element being designed, and loads applied directly on the element being designed. Adjustments for the effects of lateral loads should be performed employing the guides of [Clause 16](#).

#### 12.2.1.1 Tributary loads

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

- 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;
- b) live loads applied on the supported elements.

#### 12.2.1.2 Loads carried directly by the beam, girder or joist

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

- a) dead loads: including selfweight of the structural element, and the flat and standing non-structural elements, and fixed equipment loads, applied directly on the element;
- b) live loads applied directly to the element being designed.

### 12.2.2 Factored design loads

#### 12.2.2.1 Factored design loads for loads carried directly by the element

- a) For uniformly distributed loads carried directly by the girder, beam, or joist, the value of the uniformly distributed factored design loads,  $w_u$  in N/m, should be the greater value obtained combining  $w_d$  and  $w_l$  using [Formula \(5\)](#) and [Formula \(6\)](#). If the girder, beam, or joist, is part of a roof system, [Formula \(7\)](#) and [Formula \(8\)](#) should also be investigated, choosing the greatest value of all four formulae.
- b) For all concentrated loads carried directly by the girder, beam, or joist, the value of any concentrated factored design load,  $p_u$ , in N, should be the greater value obtained combining  $p_d$  and  $p_l$  [Formula \(5\)](#) and [Formula \(6\)](#), for each concentrated load locations in the girder, beam, or joist span.

#### 12.2.2.2 Factored reactions from supported structural elements

- a) The largest factored uniformly distributed reaction from all tributary structural elements,  $r_u$ , in N/m, should be obtained.
- b) For concentrated loads, the largest factored concentrated reactions from all the supported structural elements,  $R_u$ , in N, should be obtained for all concentrated load locations in the girder, beam, or joist span.

#### 12.2.2.3 Total factored design load

- a) The total factored uniformly distributed load  $W_u$ , in N/m, should be the sum of the values obtained for factored uniformly distributed loads,  $w_u$ , from [12.2.2.1](#) and reactions,  $r_u$ , from [12.2.2.2](#).

- b) For all concentrated load locations in the girder, beam, or joist span the total factored concentrated load  $P_u$ , in N, should be the sum of the values obtained for factored concentrated loads,  $p_u$ , from [12.2.2.1](#) and reactions,  $R_u$ , from [12.2.2.2](#).

## 12.3 Details of reinforcement

### 12.3.1 General

For the purposes of the present guidelines, the reinforcement of girders, beams, and joists, should be of the types described and should comply with the guides of [12.3.2](#) to [12.3.4.22](#).

### 12.3.2 Transverse reinforcement

#### 12.3.2.1 Description

Transverse reinforcement for girders, beams, and joists, should consist of stirrups that surround the longitudinal reinforcement and are placed perpendicular to the longitudinal axis of the element at varying intervals along the axis. The stirrup should consist of single or multiple vertical legs. Each vertical leg should engage a longitudinal bar either by bending around it when the stirrup continues or by the use of a standard stirrup hook (see [9.3.12](#)) surrounding the longitudinal bar at the end of the stirrup. See [Figure 71](#). Under the present guidelines, all stirrups in girders and beams should be closed stirrups with 135° hooks, as shown in [Figure 71 a\)](#). In joists, it should be permitted to employ all the stirrup types shown in [Figure 71](#).

#### 12.3.2.2 Location

#### 12.3.2.3 Minimum transverse reinforcement area

The minimum area of shear reinforcement,  $A_v$ , within a distance  $s$ , should comply with the guides of [9.8.4.4](#).  $A_v$  corresponds to the product of the area of the bar of the stirrup,  $A_b$ , multiplied by the number of vertical legs of the stirrup.

#### 12.3.2.4 Maximum and minimum spacing of stirrups

Stirrups should not be spaced further apart as indicated in [9.8.4.4](#), nor should it be placed closer than as indicated in [9.3.15](#).

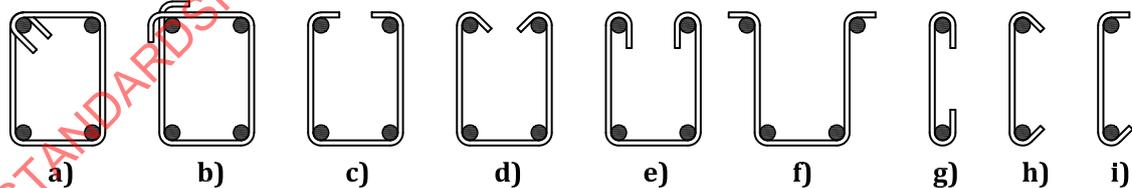


Figure 71 — Typical stirrup shapes

#### 12.3.2.5 Stirrup leg splicing

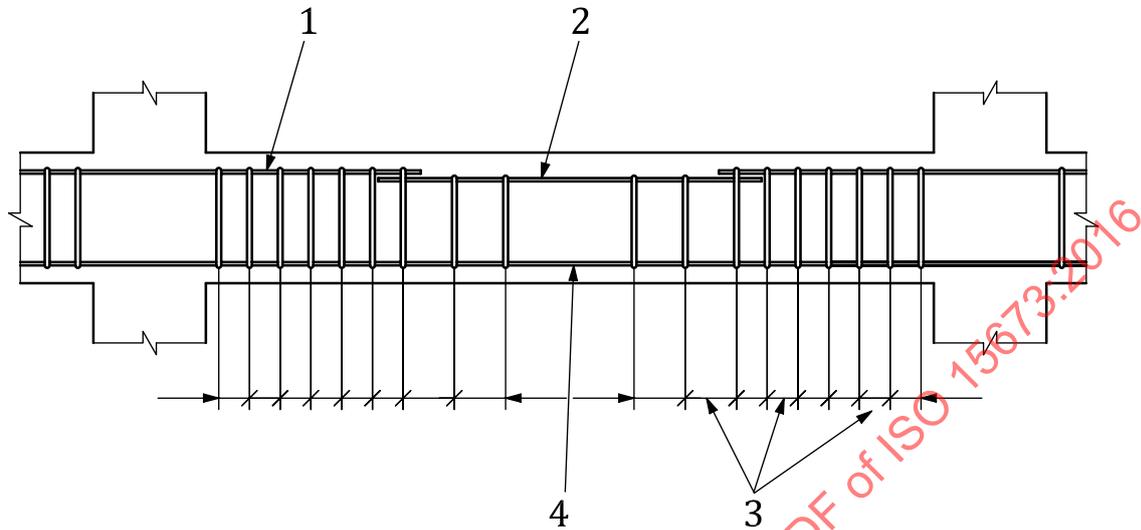
It should not be permitted to lap-splice bars that are part of stirrups.

#### 12.3.2.6 Hanger reinforcement

Where beams are supported by other girders or beams of similar height, special hanger reinforcement stirrups should be provided as indicated in [12.3.5.5.4](#).

**12.3.2.7 Support of stirrups**

Stirrups should be attached and anchored in the upper part of the section to longitudinal negative supporting bars in order to avoid that the stirrups fall during casting of the concrete (see [Figure 72](#)). See [12.3.4.10](#).



**Key**

- 1 negative flexural reinforcement
- 2 stirrup supporting bars
- 3 stirrup separations
- 4 positive flexural reinforcement

**Figure 72 — Typical stirrups spacing along the girder, beam or joist**

**12.3.3 Positive flexural reinforcement**

**12.3.3.1 Description**

Positive flexural reinforcement should be provided in the lower part of the girder, beam or joist section, as required in [Clause 12](#) and should comply with the general guides of [12.3.3](#) and the particular guides for each element type as set forth in [12.3.5](#).

**12.3.3.2 Location**

Positive flexural reinforcement should be provided longitudinally in the girder, beam or joist. Positive flexural reinforcement should be located as close as concrete cover guides permit (see [9.3.10.1](#)) 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.

**12.3.3.3 Minimum reinforcement area**

Positive flexural reinforcement should have an area at least equal to the area guide by [9.5.3.1](#). The minimum number of bars guide by [12.3.4.15](#) should be complied with.

**12.3.3.4 Maximum reinforcement area**

Positive flexural reinforcement area should not exceed the values set forth in [9.5.3.2](#).

### 12.3.3.5 Minimum and maximum reinforcement separation

Positive flexural reinforcement should not be spaced closer than guided by [9.3.15](#) and [12.3.4.11](#). The maximum reinforcement separation should comply with [12.3.4.15](#). When two or more layers of positive reinforcement are employed, the layers should not be placed closer than permitted by [9.3.16](#).

### 12.3.3.6 Cut off points

It should be permitted to suspend, at the locations indicated in [12.3.5.5](#), no more than one-half of the positive flexural reinforcement required to resist the corresponding factored design positive moment at mid-span.

### 12.3.3.7 Reinforcement splicing

It should be permitted to lap-splice the remaining positive flexural reinforcement from [12.3.3.6](#) between the cut-off point and the opposite face of the support.

### 12.3.3.8 Embedment at interior supports

Positive flexural reinforcement suspended at an interior support should be embedded by continuing it to the opposite face of the support, plus the distance required to comply with the lap splice guide of [9.4.2](#).

### 12.3.3.9 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.

### 12.3.3.10 Positive flexural reinforcement acting in compression

Positive flexural reinforcement acting in compression should be surrounded with stirrups or ties that comply with [9.6.3.2](#).

### 12.3.3.11 Minimum diameter of longitudinal reinforcement

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

## 12.3.4 Negative flexural reinforcement

### 12.3.4.1 Description

Negative flexural reinforcement should be provided in the upper part of the girder, beam or joist section, at edges and supports, in the amounts and lengths guide in [Clause 12](#), and should comply with the general guides of [12.3.4](#), and the particular guides of [12.3.5](#).

### 12.3.4.2 Location

Negative flexural reinforcement should be provided at edge and intermediate supports. Negative flexural reinforcement should be located as close as concrete cover guides permit (see [9.3.10.1](#)) to the upper surface of the 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.

#### 12.3.4.3 Minimum reinforcement area

Negative flexural reinforcement should have an area at least equal to the area guide by [9.5.3.1](#). The minimum number of bars required by [12.3.4.15](#) should be complied with.

#### 12.3.4.4 Maximum reinforcement area

Negative flexural reinforcement area should not exceed the values set forth in [9.5.3.2](#).

#### 12.3.4.5 Minimum and maximum reinforcement separation

Negative flexural reinforcement should not be spaced closer than guided by [9.3.15](#) and [12.3.4.11](#). The maximum reinforcement separation should comply with [12.3.4.15](#). When two or more layers of negative reinforcement are employed, the layers should not be placed closer than permitted by [9.3.16](#). Negative reinforcement of T-beam construction should comply with [12.3.4.20](#).

#### 12.3.4.6 Cut off points

It should be permitted to suspend the negative flexural reinforcement, except for cantilevers, at the locations indicated in [12.3.5.5](#). Where adjacent spans are unequal, cut-off points of negative flexural reinforcement should be based on the guides for the longer span.

#### 12.3.4.7 Reinforcement splicing

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

#### 12.3.4.8 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, complying with the anchorage distance required by [9.4.3](#). At the external edge of cantilevers, negative flexural reinforcement perpendicular to the edge should end in a standard hook.

#### 12.3.4.9 Negative flexural reinforcement acting in compression

Negative flexural reinforcement acting in compression should be surrounded with stirrups or ties that comply with [9.6.3.2](#).

#### 12.3.4.10 Negative reinforcement for support of stirrups

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

#### 12.3.4.11 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 [9.3.10](#)), the maximum nominal coarse aggregate size, and the minimum clear spacing between bars (see [9.3.13](#)). When these computations are not performed, it should be permitted to employ the guides of [12.3.4.12](#) to [12.3.4.14](#).

#### 12.3.4.12 Girders and beams with $b_w \geq 300$ mm

For girders and beams whose width,  $b_w$ , is greater than or equal to 300 mm, it should be permitted to determine the maximum number of bars in a layer employing [Formula \(79\)](#) where  $b_w$  is the girder or beam width, in mm. See [Table 18](#).

$$\text{No. of bars in a layer} \leq \frac{b_w}{50} - 3 \quad (79)$$

#### 12.3.4.13 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 greater than or equal to 250 mm. Two longitudinal bars should be employed for girders and beams whose width,  $b_w$ , is less than 250 mm. See [Table 18](#).

**Table 18 — Maximum number of longitudinal bars in a layer for girders and beams**

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

#### 12.3.4.14 Joists

The maximum number of longitudinal bars in joists should be one for web widths,  $b_w$ , (see [Figure 48](#)), less than or equal to 150 mm, but it should be permitted to bundle in contact up to two bars locating one in top of the other. For web widths greater than 150 mm and less than 200 mm, the maximum number of bars in a single layer should be two, and it should not be permitted to bundle them. For web widths greater than and equal to 200 mm, the maximum number of bars in a single layer should be one more than those allowed for girders and beams in [12.3.4.12](#) and [12.3.4.13](#).

#### 12.3.4.15 Minimum number of longitudinal bars in a layer

To minimize flexural cracking width at points of maximum moment, a larger number of smaller diameter bars should be employed as opposite to a small number of large diameter bars. For joists, the minimum number of longitudinal bars should be one. [12.3.4.16](#) and [12.3.4.17](#) should be met at sections of maximum positive and negative moment for girders and beams whose width,  $b_w$ , is greater than or equal to 300 mm. For girders and beams with  $b_w$  less than 300 mm, the minimum number of longitudinal bars should be two.

#### 12.3.4.16 Exterior exposure

The minimum number of longitudinal bars in a layer that should be employed for girders and beams that are exposed to earth or weather should be greater than or equal to the value given by [Formula \(80\)](#), where  $b_w$  is the girder or beam width in mm.

$$\text{No. of bars in a layer} \geq \frac{b_w}{100} \quad (80)$$

12.3.4.17 Interior exposure

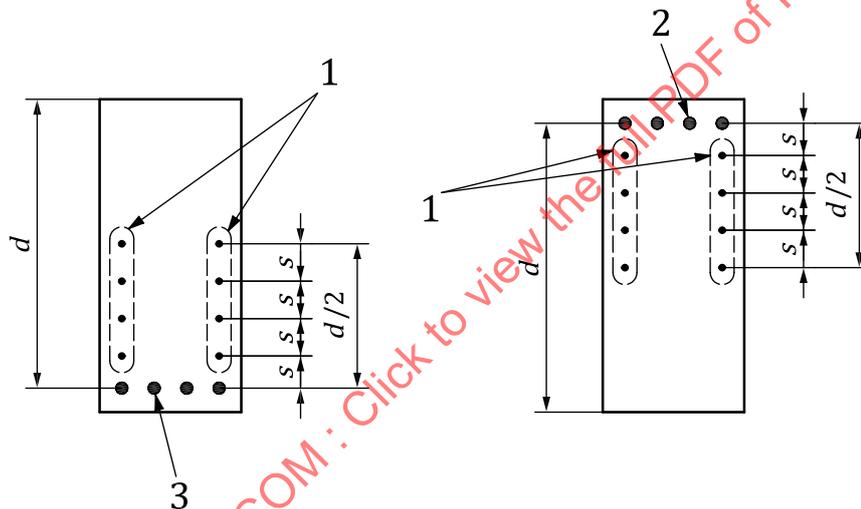
The minimum number of longitudinal bars in a layer that should be employed for girders, and beams, that are not exposed to earth or weather, should be greater than or equal to the value given by [Formula \(81\)](#), where  $b_w$  is the girder or beam width in mm.

$$\text{No. of bars in a layer} \geq \frac{b_w}{200} \tag{81}$$

12.3.4.18 Skin reinforcement

If the effective depth  $d$  of a girder, beam, or joist exceeds 800 mm, longitudinal skin reinforcement should be uniformly distributed along both side faces of the member for a vertical distance equal to  $d/2$  nearest the flexural tension reinforcement. The vertical spacing,  $s$ , in mm, between bars should be obtained using [Formula \(82\)](#), but it should not exceed  $d/6$ , nor 300 mm. See [Figure 73](#)

$$s = \frac{1000 \cdot A_b}{d - 750} \tag{82}$$



Key

- 1 skin reinforcement
- 2 negative reinforcement in tension
- 3 positive reinforcement in tension

Figure 73 — Skin reinforcement for girders, beams and joists with  $d > 800$  mm

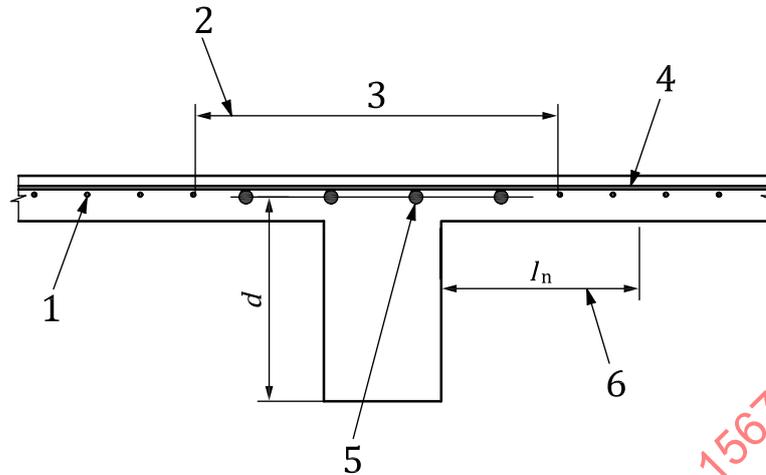
12.3.4.19 Reinforcement in flanges of T-beams

For girders and beams that are shaped as T-beams, except joists, with a flange or slab in the upper part, the guides of [12.3.4.20](#) and [12.3.4.21](#) for the reinforcement located in the flange should be employed. When the girder or beam is part of a slab-on-girder system, the reinforcement in the flange should not be less than the required for the slab.

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

Where flanges of T-beam construction are in tension, negative flexural reinforcement in the direction of the beam should be distributed over width equal to the smaller of the effective flange width defined in [9.7.6](#) or one-tenth of the span of the beam. If the effective flange width of [9.7.6](#) exceeds one-tenth of the span, the rest of the effective flange width should have reinforcement in the direction of the

beam greater than or equal to the shrinkage and temperature reinforcement for slabs of 9.5.2.1. See Figure 74. For this case, the guides of 12.3.4.11 should not apply.



#### Key

- 1 shrinkage and temperature reinforcement outside of with for distribution of negative reinforcement
- 2 whith for distribution of negative flexural reinforcement in tension
- 3 minimum of the effective flange whith or on-tenth of the beam span
- 4 transverse reinforcement calculated supposing the flange acts as a cantilever
- 5 beam negative flexural reinforcement in tension
- 6 clear cantilever span for obtaining transverse reinforcement, equal to overhanging portion of effective flange width or full overhang for isolated T-beams

Figure 74 — Reinforcement in flanges of T-beams

#### 12.3.4.21 Transverse flange reinforcement

In the top of the flange, reinforcement perpendicular to the beam should be provided to resist the factored negative moment obtained from Formula (56), using a value of  $l_m$  equal to the overhanging portion of the flange width defined in 9.7.6, and for isolated T-beams the full width of overhanging flange or the flange width defined in 9.7.6. This reinforcement should comply with the guides for negative flexural reinforcement in slabs as set forth in 11.3.4. See Figure 74.

#### 12.3.4.22 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 Clause 16. Joists and beams that are not part of a frame are exempt from the additional seismic guides.

### 12.3.5 Joists and beams supported on girders

#### 12.3.5.1 General

12.3.5 cover joists and beams that are supported on girders and are cast monolithically with them. Two-way joist systems or waffle-on-beams systems, as described in 10.1.4.4, are covered and should comply with the general guides of Clause 10.

### 12.3.5.2 Dimensional guides

#### 12.3.5.2.1 Joists

In addition to the appropriate guides of [Clause 12](#), joists should comply with the general dimensional guides of [6.1](#), and the particular guides of [10.1.4.1](#). The minimum allowable depth should comply with [10.5.3](#) for one-way joist and of [10.5.4](#) for two-way joists.

#### 12.3.5.2.2 Beams

In addition to the appropriate guides of [Clause 12](#), beams supported on girders should comply with the general dimensional guides of [6.1](#) and the particular guides of [10.1.2](#). The minimum allowable depth should comply with [10.5.3](#). The width of the web of beams,  $b_w$ , should not be less than 200 mm. The spacing between lateral supports of isolated beams should not exceed 50 times the least width  $b$  of compression flange or face.

#### 12.3.5.2.3 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.

### 12.3.5.3 Factored flexural moment

#### 12.3.5.3.1 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 out of the edge supporting girders, beams or structural concrete walls, should be calculated supposing that one-half of the distributed factored load,  $W_u$ , acts as a concentrated load at the tip of the cantilever along with all concentrated loads that act on the span of the cantilever,  $\Sigma P_u$ , and the other one-half acts as uniformly distributed load over the full span, using [Formula \(83\)](#), 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.

$$M_u^- = \frac{3 \cdot W_u \cdot l_n^2}{4} + l_n \cdot \sum P_u \quad (83)$$

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

#### 12.3.5.3.2 One-span joists and beams supported on beams, girders or walls

The factored positive and negative flexural moment,  $M_u$ , in N · m, for one-span beams and one-span one-way joists should be calculated using the [Formula \(84\)](#) and [Formula \(85\)](#), where  $l_m$  should be the clear span of the beam or joist in m,  $W_u$  should be employed in N/m, and  $\Sigma P_u$  should be in N.

Positive moment:

$$M_u^+ = \frac{W_u \cdot l_m^2}{8} + \frac{l_m}{4} \cdot \sum P_u \quad (84)$$

Negative moment at supports:

$$M_u^- = \frac{W_u \cdot l_m^2}{24} + \frac{l_m}{16} \cdot \sum P_u \quad (85)$$

### 12.3.5.3.3 Joists and beams supported on beams, girders or walls, with two or more spans

The factored positive and negative flexural moment,  $M_u$ , in  $N \cdot m$ , for beams and one-way joists supported on beams, girders or structural walls should be calculated using the [Formula \(86\)](#) to [Formula \(91\)](#), where  $l_m$  should be the clear span of the beam or joist in m,  $W_u$  should be employed in  $N/m$ , and  $\Sigma P_u$  should be in N.

Positive moment:

— at end spans:

$$M_u^+ = \frac{W_u \cdot l_m^2}{11} + \frac{l_m}{9} \cdot \Sigma P_u \quad (86)$$

— at interior spans:

$$M_u^+ = \frac{W_u \cdot l_m^2}{16} + \frac{l_m}{5} \cdot \Sigma P_u \quad (87)$$

Negative moment at supports:

— at interior face of external support:

$$M_u^- = \frac{W_u \cdot l_m^2}{24} + \frac{l_m}{16} \cdot \Sigma P_u \quad (88)$$

— at exterior face of first internal support, only two spans:

$$M_u^- = \frac{W_u \cdot l_m^2}{9} + \frac{l_m}{6} \cdot \Sigma P_u \quad (89)$$

— at faces of internal supports, more than two spans:

$$M_u^- = \frac{W_u \cdot l_m^2}{10} + \frac{l_m}{7} \cdot \Sigma P_u \quad (90)$$

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

$$M_u^- = \frac{W_u \cdot l_m^2}{12} + \frac{l_m}{8} \cdot \Sigma P_u \quad (91)$$

### 12.3.5.3.4 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 obtained from [12.3.5.3.1](#) to [12.3.5.3.3](#), and [12.3.5.4.1](#) to [12.3.5.4.3](#), if the following guides are met.

- The analysis procedure should be based on established principles of structural mechanics.
- The procedure should take into account equilibrium, compatibility of deformations, general stability, and short-term and long-term material properties.
- The analysis procedure should take into account the flexibility of the supports and the interaction between flexure and torsion of the supported and supporting elements.
- The modulus of elasticity of concrete should be permitted to be taken as  $E_c = 4500\sqrt{f'_c}$ , in MPa.

- e) Use of any set of reasonable assumptions should be permitted for computing relative flexural and torsional stiffness of the structural elements. The assumptions adopted should be consistent throughout the analysis.
- f) Span length should be taken as the distance centre-to-centre of supports, but it should be permitted to obtain the factored moment and shear at faces of supports.
- g) It should be permitted to assume that the arrangement of live load is limited to combinations of factored dead load on all spans with full factored live load on two adjacent spans and factored dead load on all spans with full factored live load on alternate spans.

**12.3.5.3.5 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 employing the guides of [11.8.1](#) and [11.8.2](#), except the minimum depth of the supporting beams or girders guide in [11.8.1 c](#)) (see [10.1.4.4](#)).

**12.3.5.4 Factored shear**

**12.3.5.4.1 Cantilevers of joists and beams supported on beams, girders or walls**

The factored shear,  $V_u$ , at the support of cantilevers should be calculated using [Formula \(92\)](#), where  $l_m$  should be the clear span of the cantilever in m,  $W_u$  should be employed in N/m, and  $\Sigma P_u$  in N.

$$V_u = W_u \cdot l_m + \sum P_u \tag{92}$$

**12.3.5.4.2 One-span joists and beams supported on beams, girders or walls**

The factored shear,  $V_u$ , in N, for one-span beams and one-span one-way joists should be calculated using [Formula 93](#), where  $l_m$  should be the clear span of the beam or joist in m,  $W_u$  should be employed in N/m, and  $\Sigma P_u$  should be in N.

$$V_u = \frac{W_u \cdot l_m}{2} + 0,8 \cdot \sum P_u \tag{93}$$

**12.3.5.4.3 Joists and beams supported on beams, girders or walls, with two or more spans**

The factored positive and negative flexural moment,  $M_u$ , in N · m, for beams and one-way joists supported on beams, girders or structural walls should be calculated using [Formula \(94\)](#) and [Formula \(95\)](#), where  $l_m$  should be the clear span of the beam or joist in m,  $W_u$  should be employed in N/m, and  $\Sigma P_u$  should be in kN.

At exterior face of first interior support:

$$V_u = 1,15 \cdot \frac{W_u \cdot l_m}{2} + 0,80 \cdot \sum P_u \tag{94}$$

At faces of all other supports:

$$V_u = \frac{W_u \cdot l_m}{2} + 0,75 \cdot \sum P_u \tag{95}$$

**12.3.5.4.4 Use of frame analysis**

It should be permitted to use a frame analysis for obtaining the factored shears as a substitute for the values obtained from [12.3.5.4.1](#) to [12.3.5.3.3](#) if the guides of [12.3.5.3.4](#) are met.

#### 12.3.5.4.5 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 employing the guides of [11.8.1](#) and [11.8.4](#), except the minimum depth of the supporting beams or girders guide in [11.8.1 c](#)) (see [10.1.4.4](#)).

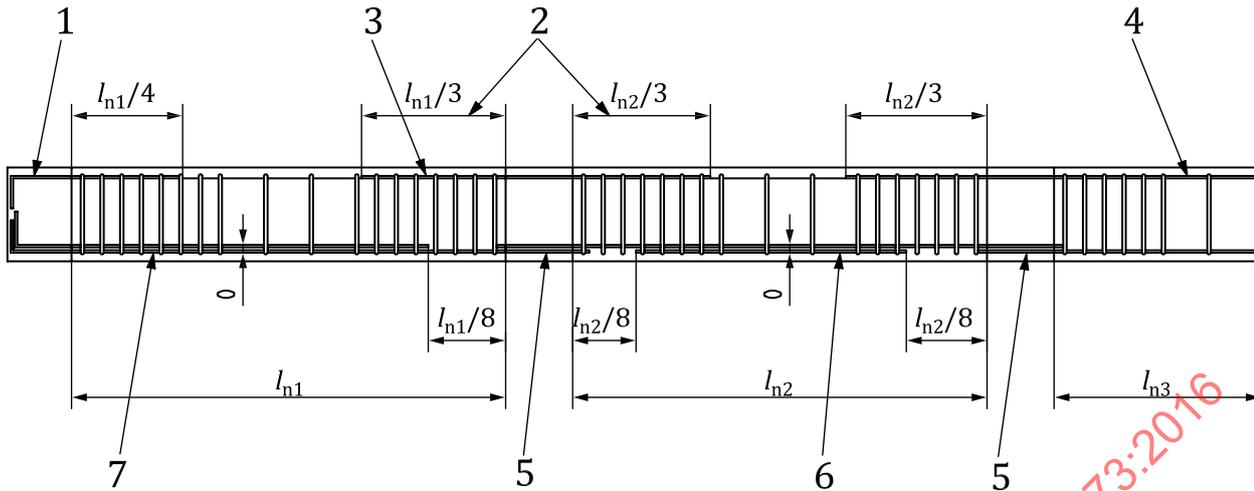
### 12.3.5.5 Reinforcement

#### 12.3.5.5.1 Positive flexural reinforcement

The positive reinforcement area should be determined employing [Formula \(35\)](#) or [Formula \(36\)](#), with the appropriate value of  $M_u^+$  obtained from [12.3.5.3](#) converted to  $N \cdot mm$  ( $1 N \cdot m = 10^3 \cdot N \cdot mm$ ), using  $d$  and  $b$  in mm. When a slab is present in the upper part of the section or when the beam or joist is T-shaped, it should be permitted to employ the T-beam effect as indicated in [9.7.6](#). The positive flexural reinforcement should comply with the guides of [12.3.3](#). At internal supports, at a distance equal to  $l_m/8$  measured from the face of the supports into the span, up to one-half of the positive flexural reinforcement should be permitted to be suspended if there are no concentrated loads within that distance. For one-span beams and joists, no suspension of positive reinforcement should be permitted. See [Figure 75](#).

#### 12.3.5.5.2 Negative flexural reinforcement

The negative flexural reinforcement area should be determined employing [Formula 35](#) or [Formula 36](#), for the greater value of  $M_u^-$  obtained from [12.3.5.3](#) for both sides of the support, converted to  $N \cdot mm$  ( $1 N \cdot m = 10^3 \cdot N \cdot mm$ ), using  $d$  and  $b$  in mm. This reinforcement should comply with the guides of [12.3.4](#). 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 [12.3.4.19](#). At a distance equal to  $l_m/4$  for external supports, and  $l_m/3$  for internal supports, measured from the internal face of the support into the span, all the negative flexural reinforcement should be permitted to be suspended. See [Figure 75](#). No suspension of negative reinforcement should be permitted in cantilevers.



**Key**

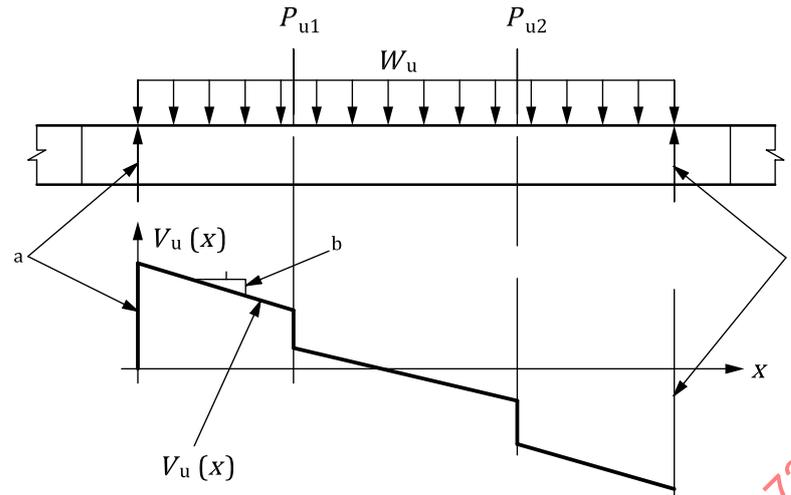
- 1 negative reinforcement at interior face of external support
- 2 negative reinforcement cut-off points based greater of the neighbouring spans
- 3 negative reinforcement interior support
- 4 greater of cantilever negative reinforcement or required for internal support
- 5 splice according to 9.4.2
- 6 positive reinforcement end span
- 7 positive reinforcement interior span

**Figure 75 — Reinforcement for beams and joists supported on beams or girders**

**12.3.5.5.3 Transverse reinforcement**

The values of  $V_u$  at the faces of the right and left supports should be obtained using the appropriate formula from 12.3.5.4. A diagram showing the shear variation within the span should be constructed, starting with the value of  $V_u$  in N, at the face of the left support taken as positive. The shear from this point proceeding to the right should be decreased at a rate equal to  $[(V_u)_{\text{left supp.}} + (V_u)_{\text{right supp.}} - \Sigma P_u] / l_m$ , in kN/m. At any place where a concentrated load is applied, the value of  $P_u$  in N, should be subtracted from the value of shear shown in the diagram at the left of the load. Proceeding as described, at the face of the left support, the negative value of  $V_u$  in N there should be reached in the diagram. See Figure 76. At any place within the span, the value of  $(l \cdot V_n)$  as calculated following the guides of 9.8.4 should be greater than or equal than the absolute value of  $V_u(x)$  as shown in the calculated diagram.

The shear reinforcement should comply with the guides of 12.3.2 and 9.8.4. The limits for  $(\phi \cdot V_n)$  as defined in 9.8.4.4 should be marked in the shear diagram, and a minimum amount of shear reinforcement as defined by Formula (48) should be established. Appropriate values of the spacing of stirrups  $s$  should be defined for the different region within the shear diagram. A minimum practicable spacing of stirrups as guided by 9.3.15 should be observed. The first stirrup should not be placed further than  $s/2$  from the face of the supporting element, being  $s$  the required spacing of stirrups at the support.



- a  $(V_u)_{\text{left supp.}}$   
 b  $[(V_u)_{\text{left supp.}} + (V_u)_{\text{right supp.}} - \Sigma P_u] 1/l_n$   
 c  $(V_u)_{\text{right supp.}}$

**Figure 76 — Calculation of the shear diagram of the beam or joist supported on beams or girders**

#### 12.3.5.5.4 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 77](#). The determination of the hanger reinforcement should be made complying with the following.

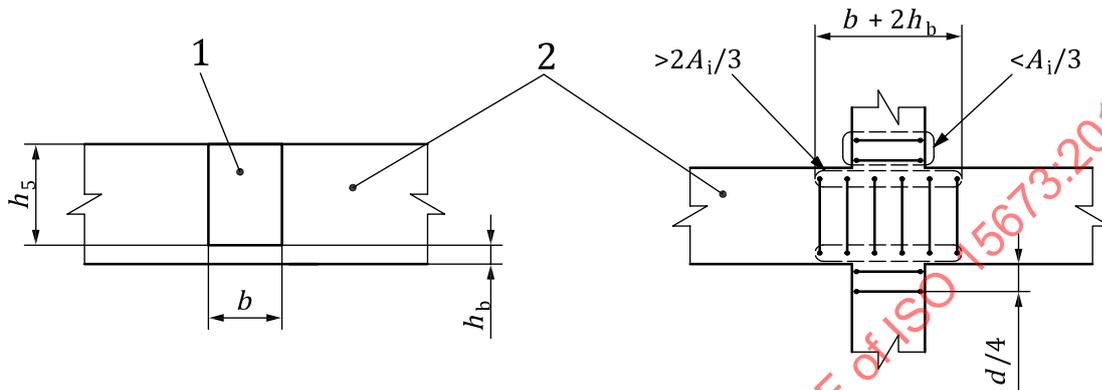
- a) If  $V_u$  from the supported beam at the interface is less than  $\left[ \phi \cdot \frac{\sqrt{f'_c}}{4} \cdot b_w \cdot d \right]$ , it should be permitted to waive the hanger reinforcement, where  $\phi = [0,85]$ . See [6.3.3 d](#).
- b) If  $h_b$  is greater than one-half the total depth of the supporting girder, where  $h_b$  is the vertical distance measured from the bottom of the supporting girder to the bottom of the supported beam, it should be permitted to waive the hanger reinforcement.
- c) The hanger reinforcement should be full depth closed stirrups with a total area  $A_i$  as determined from [Formula \(96\)](#).

$$A_i \geq \frac{\left( 1 - \frac{h_b}{h_s} \right) V_u}{\phi \cdot f_y} \quad (96)$$

- d) In [Formula \(96\)](#),  $V_u$  is the shear from the supported beam at the interface,  $A_i$  is the area of the additional stirrups,  $h_s$  is the total height of the supporting girder,  $f_y$  is the specified yield strength of the steel of the stirrups, and  $\phi = [0,85]$ . See [6.3.3 d](#).
- e) Additional stirrups with an area greater than or equal to  $2/3$  of  $A_i$  should be placed in the supporting girder for a distance along the longitudinal axis of the supporting girder equal to or less than the width of the supported beam,  $b_w$ , plus  $h_b$  at each side. In the computation of the  $2/3$  of  $A_i$ , only the

area of legs of the additional stirrups that are located at the side of the supported beam should be taken into account.

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



- Key**
- 1 supported beam
  - 2 supported girder

Figure 77 — Hanger reinforcement

**12.3.5.6 Calculation of the reactions on beams and girders**

**12.3.5.6.1 One-way joists**

The factored reaction on the supports of joist should be permitted to be considered uniformly distributed. The factored reaction on the supports  $r_u$ , in N/m, should be the value obtained from [Formula \(97\)](#) plus the uniformly distributed reaction from any cantilever spanning from that support (see [Figure 48](#)):

$$r_u = \frac{V_u \cdot l}{s \cdot l_n} \tag{97}$$

where

- $V_u$  is the factored shear from [12.3.5.4](#), in N;
- $l$  is the centre-to-centre span of the beam, in m;
- $l_n$  is the clear span of the joint, in m;
- $s$  is the centre-to-centre spacing between joists, in m.

**12.3.5.6.2 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 employing the guides of [11.8.1](#) and [11.8.5](#), except the minimum depth of the supporting beams or girders required in [11.8.1 c](#)) (see [10.1.4.4](#)).

### 12.3.5.6.3 Beams

The factored reaction on the supports  $R_u$ , in N, should be the value obtained from [Formula \(98\)](#) plus the factored reaction from any cantilever spanning from that support:

$$R_u = \frac{V_u \cdot l}{l_m} \quad (98)$$

where

$V_u$  is the factored shear from [12.3.5.4](#), in N;

$l$  is the centre-to-centre span of the beam, in m;

$l_m$  is the clear span of the beam, in m.

## 13 Columns

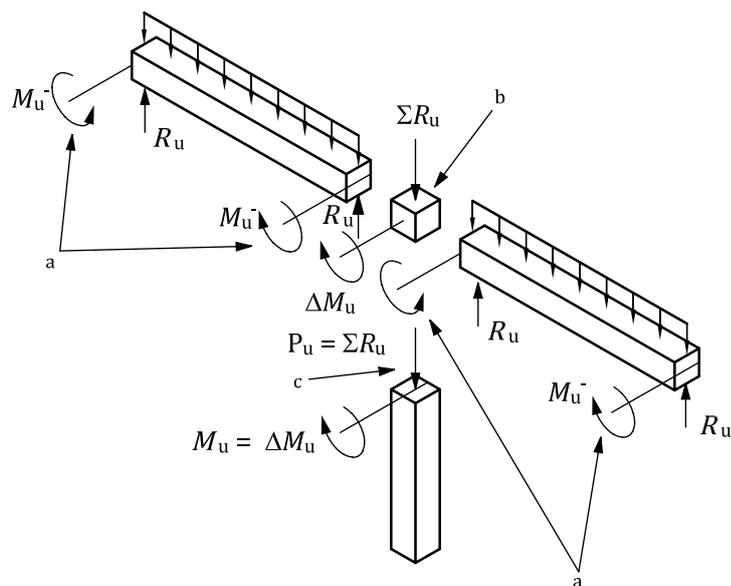
### 13.1 General

The design of columns should be performed using the guides of [Clause 13](#). The members covered by this subclause are members reinforced with longitudinal bars and lateral ties, and members reinforced with longitudinal bars and continuous spiral. Both rectangular and circular sections are covered.

### 13.2 Design load definition

#### 13.2.1 Loads to be included

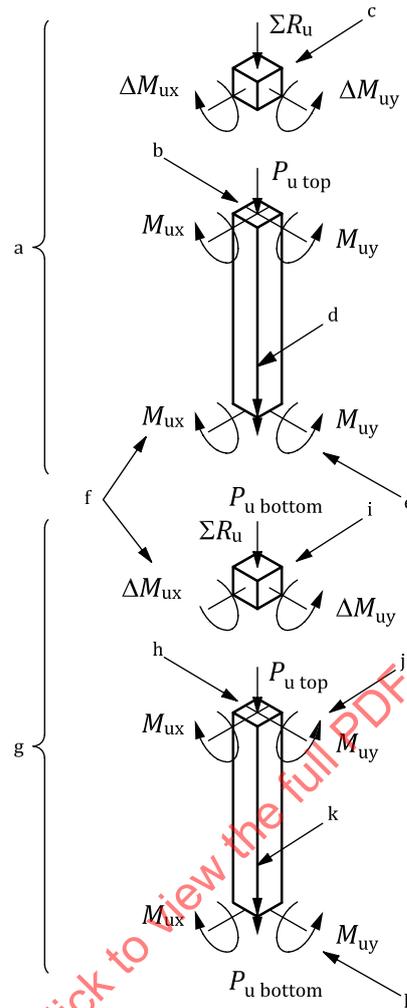
The design load for columns belonging to frames or slab-column systems should be established from the tributary loads from each floor located above the column, plus the selfweight of the column. Tributary loads should be established from the guides of [8.1](#) and the particular guides of each tributary element type. See [Figure 78](#).



- a Reactions at the ends of the element.
- b Actions at the joint.
- c Loads applied to the top of the column.

Figure 78 — Column factored design forces from one story and in one direction

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- a Story  $n$ .
- b Loads applied to the top of the column.
- c Actions at the joint of story  $n$  from tributary elements.
- d Selfweight of the column.
- e  $P_{u \text{ bottom}}$  equal to  $P_{u \text{ top}}$  plus the column selfweight.
- f Column moment obtained from distribution of unbalanced moment at joint of same level.
- g Story  $n-1$ .
- h Loads applied to the top of the column.
- i Actions at the joint of story  $n-1$  from tributary elements.
- j  $P_{u \text{ top}}$  of story  $n-1$  equal to  $P_{u \text{ bottom}}$  from story  $n$  plus  $\Sigma R_u$  from story  $n-1$ .
- j Selfweight of the column.
- l  $P_{u \text{ bottom}}$  equal to  $P_{u \text{ top}}$  plus the column selfweight.

Figure 79 — Column forces in different floors

### 13.2.2 Dead load and live load

The values of  $P_d$  for dead load and  $P_l$  for live load should be in N.  $P_d$  should include the selfweight of the column, assuming concrete unit weight as  $24 \times 10^3 \text{ N/m}^3$ . The selfweight should be factored employing the load factors for dead load of the corresponding combination formula from 8.1.1.1. It should be

permitted to apply the selfweight of the column corresponding to each floor at the lower part of the column in that floor. See [Figure 79](#).

**13.2.3 Factored design loads**

The value of the factored design loads  $P_u$  and  $M_u$  should be established for the column at the upper and lower part of the column in each story. A distinction should be made about the direction of the axis in plan along which the moments  $M_{ux}$  and  $M_{uy}$  act. See [Figure 79](#).

**13.3 Dimensional guidelines**

**13.3.1 General**

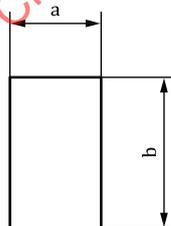
In addition to the appropriate guidelines of the present subclause, columns should comply with the general dimensional guidelines set forth in [6.1](#). Columns should be aligned vertically, without eccentricity between upper and lower columns and should be continuous all the way down to the foundation. Column section shape should be either rectangular or circular. All other cross-section shapes are beyond the scope of these guidelines.

**13.3.2 Limiting section dimensions**

**13.3.2.1 Minimum section dimensions for rectangular columns**

Under the present guidelines, section dimension for rectangular columns should comply with the following limits (see [Figure 80](#)).

- a) The shortest cross-sectional dimension should not be less than 300 mm. For columns in buildings located in seismic risk zones, see [16.5.3.1](#).
- b) The ratio of the largest cross-sectional dimension to the perpendicular shortest dimension should not exceed 3.



a  $b_c \geq 300 \text{ mm}$

b  $h \left( \frac{h_c}{b_c} \right) \leq 3$

**Figure 80 — Minimum cross-section dimensions for rectangular columns**

**13.3.2.2 Minimum section dimensions for circular columns**

Columns with circular cross-section should have a diameter of at least 300 mm, as shown in [Figure 81](#).

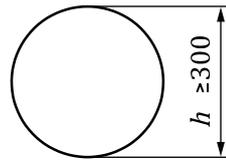
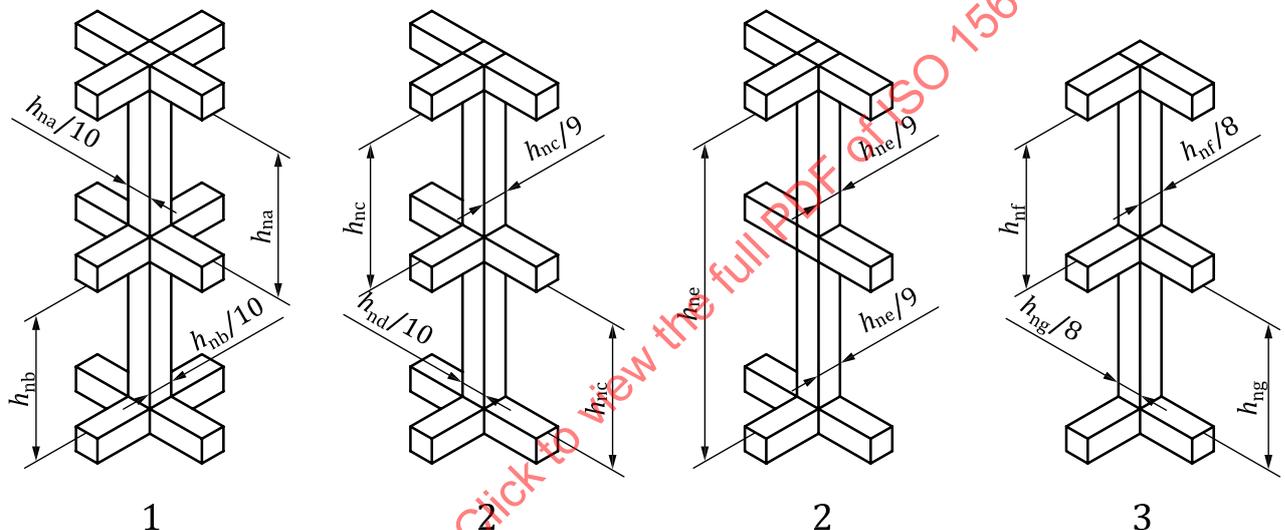


Figure 81 — Minimum cross-section dimension for circular columns

13.3.3 Distance between lateral supports

13.3.3.1 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 82.



- Key**
- 1 central
  - 2 edge
  - 3 corner

Figure 82 — Lateral restraint for columns

13.3.3.2 Central columns

The clear distance between lateral supports,  $h_n$ , for central columns should not exceed ten times the dimension of the column cross-section parallel to the direction of the support. See Figure 82.

13.3.3.3 Edge columns

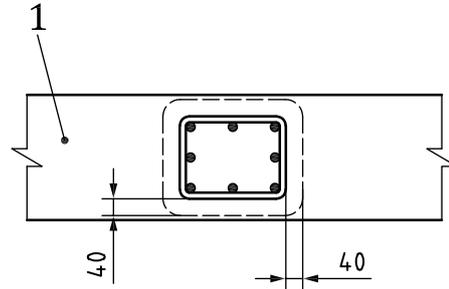
The clear distance between lateral supports,  $h_n$ , perpendicular to an edge for edge columns should not exceed nine times the dimension of the column cross-section perpendicular to the edge. See Figure 82.

13.3.3.4 Corner columns

The clear distance between lateral supports,  $h_n$ , for corner columns should not exceed eight times the minimum dimension of the column cross-section. See Figure 82.

**13.3.4 Column built monolithically with wall**

Outer limits of the effective cross-section of a tied or spirally reinforced column built monolithically with a concrete wall should be taken not greater than 40 mm outside the tie or spiral reinforcement or the lateral wall faces. See [Figure 83](#).



**Key**  
1 wall

**Figure 83 — Effective cross-section of columns built monolithically with a wall**

**13.4 Details of reinforcement**

**13.4.1 General**

For the purposes of these guidelines, the reinforcement of columns should be of the types described in this subclause and should comply with the guides of [13.4.2](#) to [13.4.3.4](#).

**13.4.2 Longitudinal reinforcement**

**13.4.2.1 Description and location**

Longitudinal reinforcement should be provided in the periphery of the column section, as guided in [9.5.4.4](#). Longitudinal reinforcement should be located as close as concrete cover guides permit (see [9.3.10.1](#) and [13.4.2.9](#)) to the lateral surfaces of the column. The amount of longitudinal reinforcement should be that guide to resist the simultaneous action of a combination of factored axial load and factored moments at the section acting about the two main axis of the section of the column. See [Figure 84](#).

**13.4.2.2 Minimum and maximum longitudinal reinforcement area**

The maximum and minimum longitudinal reinforcement area should comply with the guides of [9.5.4.1](#) ( $0,01 \leq \rho_t \leq 0,06$ ). The maximum longitudinal reinforcement area is also limited by the beam reinforcement in the beam – column joint.

**13.4.2.3 Minimum diameter of longitudinal bars**

Longitudinal bars of columns should comply with the minimum guide nominal diameter,  $d_b$ , as set forth in [9.5.4.2](#) (16 mm).

**13.4.2.4 Minimum number of longitudinal bars**

The minimum number of longitudinal bars in rectangular and round columns should be as set forth in [9.5.4.3](#) (4 bars in rectangular columns or 6 in circular columns).

### 13.4.2.5 Minimum and maximum reinforcement separation

Longitudinal reinforcement should not be spaced closer than guide by [9.3.17](#) ( $1,5 d_b$  or 40 mm).

### 13.4.2.6 Reinforcement splicing

It should be permitted to lap-splice up to one-half the longitudinal reinforcement at any given section, as long as only alternate bars are lap-spliced. See [Figure 85](#).

All lap splices of longitudinal reinforcement should comply with [9.4.2.1](#) ["alternative methods like gas pressure welding or mechanical connectors could be used taking account of the practical situation of each country"].

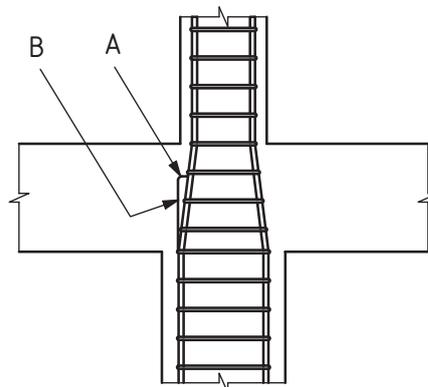
### 13.4.2.7 End anchorage of reinforcement

Longitudinal reinforcement at the upper end of the columns and at the foundation elements that transmits the loads to the underlying soil should extend to the extreme and end with a standard hook.

### 13.4.2.8 Longitudinal bar offset

Offset bent longitudinal bars should conform to the following.

- Slope of inclined portion of an offset bar with axis of column should not exceed 1 in 6.
- Portions of bar above and below an offset should be parallel to axis of column.
- Horizontal support at offset bends should be provided by lateral ties or spirals.
- 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.
- Lateral ties or spirals should be placed not more than 150 mm from points of bend.
- Offset bars should be bent before placement in the forms.
- Where a column face is offset from the face of the column below more than  $1/6$  of the depth of the girder or slab, longitudinal bars should not be offset bent. Separate dowels, lap spliced with the longitudinal bars adjacent to the offset column faces should be provided. Lap splices should conform to [9.4.2.1](#).



The ratio of  $A/B$  should not exceed  $1/6$ .

**Figure 84 — Longitudinal bar offset**

**13.4.2.9 Maximum number of longitudinal bars per face of rectangular column**

The maximum number of longitudinal bars in a layer should be determined for the longitudinal and transverse reinforcement bar diameters, the appropriate concrete cover (see 9.3.10), the maximum nominal coarse aggregate size, and the minimum clear spacing between bars (see 9.3.13). When these computations are not performed, it should be permitted to employ the following guides.

- a) For columns section dimension under study,  $b_c$ , greater than or equal to 400 mm, it should be permitted to determine the maximum number of bars in a layer employing [Formula \(99\)](#), where  $b_c$  is the column dimension under study in mm. See [Table 19](#).

$$\text{No. of bars per face} \leq \frac{b_c}{68} - 1 \tag{99}$$

- b) Three longitudinal bars should be permitted in the face of columns whose dimension under study,  $b_c$ , is less than 400 mm and greater than or equal to 300 mm. See [Table 19](#).

**Table 19 — Maximum number of longitudinal bars per face of rectangular column**

Column dimension $b_c$ (mm)	Maximum number of longitudinal bars
$b_c < 300$ mm	Section not permitted
$300 \text{ mm} \leq b_c < 400$ mm	3 bars
$400 \text{ mm} \leq b_c$	$\leq \left( \frac{b_c}{68} - 1 \right)$ bars

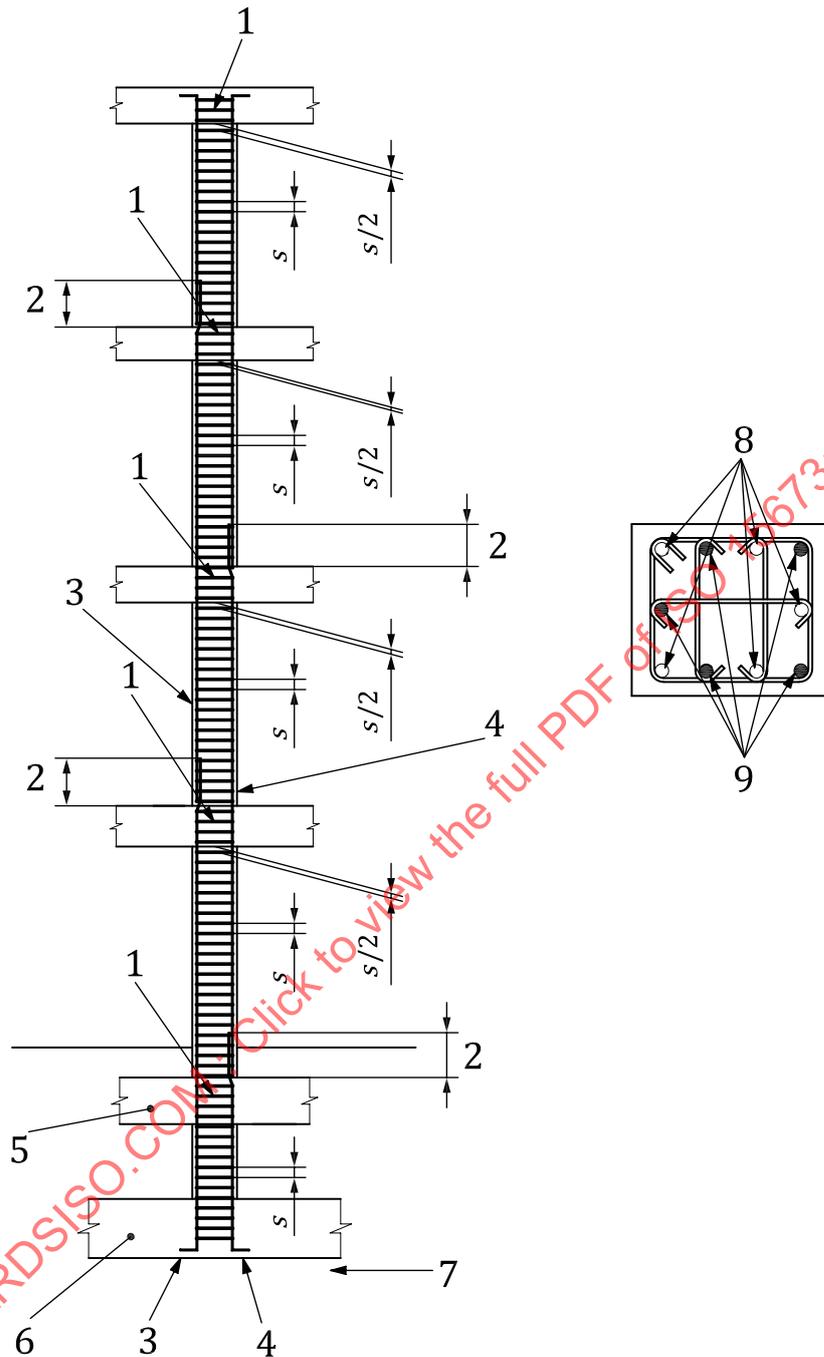
**13.4.2.10 Maximum number of longitudinal bars in circular columns**

The maximum number of longitudinal bars in circular columns should be determined for the longitudinal and transverse reinforcement bar diameters, the appropriate concrete cover (see 9.3.10), the maximum nominal coarse aggregate size, and the minimum clear spacing between bars (see 9.3.13). When these computations are not performed, it should be permitted to determine the maximum number of bars employing [Formula \(100\)](#), where  $h$  is the column diameter in mm. See [Table 20](#).

$$\text{No. of bars} \leq \frac{h}{22} - 6 \tag{100}$$

**Table 20 — Maximum number of longitudinal bars in circular columns**

Column diameter $h$ (mm)	Maximum number of longitudinal bars
$h < 300$ mm	Section not permitted
$300 \text{ mm} \leq h$	$\leq \left( \frac{h}{22} - 6 \right)$ bars



**Key**

- 1 joint ties
- 2 lap splice as guide in [9.4.2](#)
- 3 type A long bars
- 4 type B long bars
- 5 grade beam
- 6 foundation element
- 7 bearing soil
- 8 type A
- 9 type B

**Figure 85 — Typical column reinforcement layout**

### 13.4.3 Transverse reinforcement

#### 13.4.3.1 General

Transverse reinforcement in the form either of tie reinforcement, complying with [9.6.4.1](#), or spiral reinforcement, complying with [9.6.4.2](#), should be provided in all columns. At column-beam joints, a minimum amount of ties, as guided by [9.6.4.3](#) should be placed. It should not be permitted to lap-splice bars that are part of column ties.

#### 13.4.3.2 Maximum and minimum tie and spiral spacing

The maximum spacing along the axis of the column of the ties or spiral should be as indicated in [9.6.4.1](#) or [9.6.4.2](#), respectively. Tie and spirals should not be spaced closer than guided by [9.3.15](#).

#### 13.4.3.3 Tie hooks

Under the present guidelines, all ties of columns should have 135° hooks (see [9.3.12](#)). It should be permitted to employ crossties with a 135° hook in one end and a 90° in the other end, and consecutive crossties engaging the same longitudinal bar should have their 90° hooks at opposite sides of the flexural member (referring flexural members to the columns).

#### 13.4.3.4 Column reinforcement in seismic zones

In columns that are part of a moment resisting frame located in seismic zones, reinforcement should comply with the additional guides of [Clause 16](#). Columns that are part of slab-column frames in seismic zones should comply with the additional guides of [Clause 16](#).

## 13.5 Flexural design guidelines

### 13.5.1 Factored loads

The factored axial load,  $P_u$ , and moment,  $M_u$ , at section under study should be obtained following the requirements of [13.2](#).

### 13.5.2 Initial trial cross-section dimensions and longitudinal reinforcement

#### 13.5.2.1 Trial cross-section dimensions

It should be permitted to establish initial trial cross-section dimensions in the following manner.

- a) First trial gross cross-sectional area,  $A_g$ , should be obtained from [Formula \(101\)](#):

$$A_g \approx \frac{2 \cdot (P_u)_{\max.}}{f'_c} \quad (101)$$

- b) For rectangular cross-sections least dimension,  $b$ , should comply with [Formula \(102\)](#):

$$b \begin{cases} \geq 300 \text{ mm} \\ \geq h/3 \\ \geq h_n/10 \text{ for central columns} \\ \geq h_n/9 \text{ for edge columns} \\ \geq h_n/8 \text{ for corner columns} \end{cases} \quad (102)$$

- c) For rectangular cross-sections larger dimension,  $h$ , should comply with [Formula \(103\)](#):

$$h \begin{cases} \geq 300 \text{ mm} \\ \leq 3 \cdot b \\ \geq h_n/10 \text{ for central columns} \\ \geq h_n/9 \text{ for edge columns} \\ \geq h_n/8 \text{ for corner columns} \end{cases} \quad (103)$$

- d) For circular columns diameter,  $h$ , should comply with [Formula \(104\)](#):

$$h \begin{cases} \geq 300 \text{ mm} \\ \geq h_n/10 \text{ for central columns} \\ \geq h_n/9 \text{ for edge columns} \\ \geq h_n/8 \text{ for corner columns} \end{cases} \quad (104)$$

### 13.5.2.2 Trial longitudinal reinforcement

It should be permitted to establish initial trial area of longitudinal reinforcement,  $A_{st}$ , in the following manner.

- a) For rectangular cross-sections the first trial area of longitudinal reinforcement,  $A_{st}$ , should comply with [Formula \(105\)](#) (for minimum bar diameter of [9.5.4.2](#)):

$$A_{st} \geq \begin{cases} 0,01 \cdot A_g \\ 4 \cdot (A_b)_{\min.} \end{cases} \quad (105)$$

- b) For circular cross-sections the first trial area of longitudinal reinforcement,  $A_{st}$ , should comply with [Formula \(106\)](#) (for minimum bar diameter of [9.5.4.2](#)):

$$A_{st} \geq \begin{cases} 0,01 \cdot A_g \\ 6 \cdot (A_b)_{\min.} \end{cases} \quad (106)$$

### 13.5.2.3 Factored flexural moment verification

Interaction diagrams for the column dimensions and reinforcement should be calculated in both directions using the guides of [13.7.2](#) to [13.7.4.4](#). The design moment in both directions should be verified employing the guides of [13.7.4.5](#). If the factored flexural moment,  $M_u$ , at required factored axial load,  $P_u$ , exceeds the design moment strength at axial load level  $P_u$ , the area of longitudinal reinforcement should be increased, without exceeding the maximum reinforcement area permitted by [9.5.4.1](#) or the maximum number of bars in the face of column of [13.4.2.10](#). If an increase of the column dimensions is required because these limits are exceeded, the columns selfweight should be corrected and the column should be verified for the new dimensions. These verifications should be performed at the upper and lower sections of the column of the same story.

### 13.5.2.4 Biaxial moment strength verification

Once the column is verified for both directions independently, the biaxial design moment should be verified employing the guides of [13.7.4.7](#) at the upper and lower sections of the column of the same story.

## 13.6 Shear design guidelines

### 13.6.1 Factored shear

The factored shear,  $V_u$ , should be determined from the vertical loads and from the horizontal loads. The value should be that obtained from the appropriate load combinations from [8.1.1](#).

**13.6.1.1 Factored shear from vertical loads**

The factored shear caused by the vertical loads should be determined from [Formula \(107\)](#) for each direction:

$$V_u = \frac{(M_u)_{top} + (M_u)_{bottom}}{h_n} \tag{107}$$

where

- $(M_u)_{top}$  is the factored moment at the upper end of the column;
- $(M_u)_{bottom}$  is the factored moment at the lower end of the columns;
- $h_n$  is the clear distance between lateral supports of the column.

**13.6.1.2 Factored shear from horizontal loads**

The factored shear,  $V_u$ , caused by horizontal loads should be determined from the horizontal loads prescribed in [Clause 8](#), employing the appropriate load combinations from [8.1.1](#).

**13.6.2 Shear strength verification**

The shear strength verification should be performed for beam-action shear employing the guides of [9.8.3](#) and [9.8.4](#). The contribution of concrete to shear strength should be evaluated using [Formula \(45\)](#). The contribution of the transverse reinforcement of the column should be determined in the direction under study using [Formula \(46\)](#) where  $A_v$  corresponds to the area of the tie legs parallel to the shear, and  $s$  to the larger tie spacing within the clear height of the column. The verification in the direction under study should be performed employing [Formula \(43\)](#) and [Formula \(44\)](#). If [Formula \(43\)](#) is not met, the tie spacing  $s$  should be reduced.

**13.6.3 Biaxial shear strength verification**

When the column is subjected to shear in the direction of each axis simultaneously, it should comply with [Formula \(108\)](#):

$$\sqrt{\left[ \frac{(V_u)_x}{(\phi \cdot V_n)_x} \right]^2 + \left[ \frac{(V_u)_y}{(\phi \cdot V_n)_y} \right]^2} \leq 1,0 \tag{108}$$

where  $(V_u)_x$  and  $(V_u)_y$  correspond to the factored shear that act in the direction of axis  $x$  and  $y$ , and  $(\phi \cdot V_n)_x$  and  $(\phi \cdot V_n)_y$  correspond to the values of the design shear strength obtained from [Formula \(107\)](#) for the appropriate direction  $x$  or  $y$ .

**13.7 Strength of members subjected to axial loads with or without flexure**

**13.7.1 General**

Calculation of the design strength of member sections of columns and structural concrete walls subjected to axial loads or axial loads accompanied by flexural moments should be performed employing the requirements of [9.6.4](#).

**13.7.2 Combined factored axial load and factored bending moment**

The factored axial load,  $P_u$ , and the factored bending 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 [Clause 11](#) and [Clause 12](#).

### 13.7.3 Design strength for axial compression

#### 13.7.3.1 Design strength for axial compression without flexure

[Formula \(109\)](#) should be used to determine the design axial strength,  $\phi \cdot P_{0n}$ , for axial compression without flexure:

$$\phi \cdot P_{0n} = \phi \cdot \left[ 0,85 \cdot f'_c (A_g - A_{st}) + A_{st} \cdot f_y \right] \quad (109)$$

In [Formula \(109\)](#),  $\phi = [0,70]$  for columns with ties and structural concrete walls and  $\phi = [0,75]$  for columns with spiral reinforcement; see [6.3.3 c](#)).

#### 13.7.3.2 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 [Formula \(110\)](#) and [Formula \(111\)](#).

a) Columns with ties and structural concrete walls:

$$\phi \cdot P_{n(\max)} \leq 0,80 \cdot \phi \cdot P_{0n} \quad (110)$$

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

b) Columns with spiral reinforcement:

$$\phi \cdot P_{n(\max)} \leq 0,85 \cdot \phi \cdot P_{0n} \quad (111)$$

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

### 13.7.4 Balanced strength for axial compression with flexure

#### 13.7.4.1 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 [Formula \(112\)](#) and [Formula \(113\)](#), respectively. However, these formulae only apply to rectangular columns with symmetrical reinforcement.

$$\phi \cdot P_{bn} = \phi \cdot 0,40 \cdot f'_c \cdot h \cdot b \quad (112)$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0,25 \cdot h + \phi \cdot (0,95A_{se} + 0,16A_{si}) \cdot f_y \cdot \left( \frac{h}{2} - d' \right) \quad (113)$$

For [Formula \(113\)](#), the total longitudinal reinforcement area,  $A_{st}$ , should be divided into edge steel,  $A_{se}$ , and intermediate steel,  $A_{si}$ , in such a manner that  $A_{se} + A_{si} = A_{st}$ ; see [Figure 86](#). In [Formula \(112\)](#),  $\phi = [0,70]$  while in [Formula \(113\)](#),  $\phi = [0,90]$ ; see [6.3.3 c](#)).

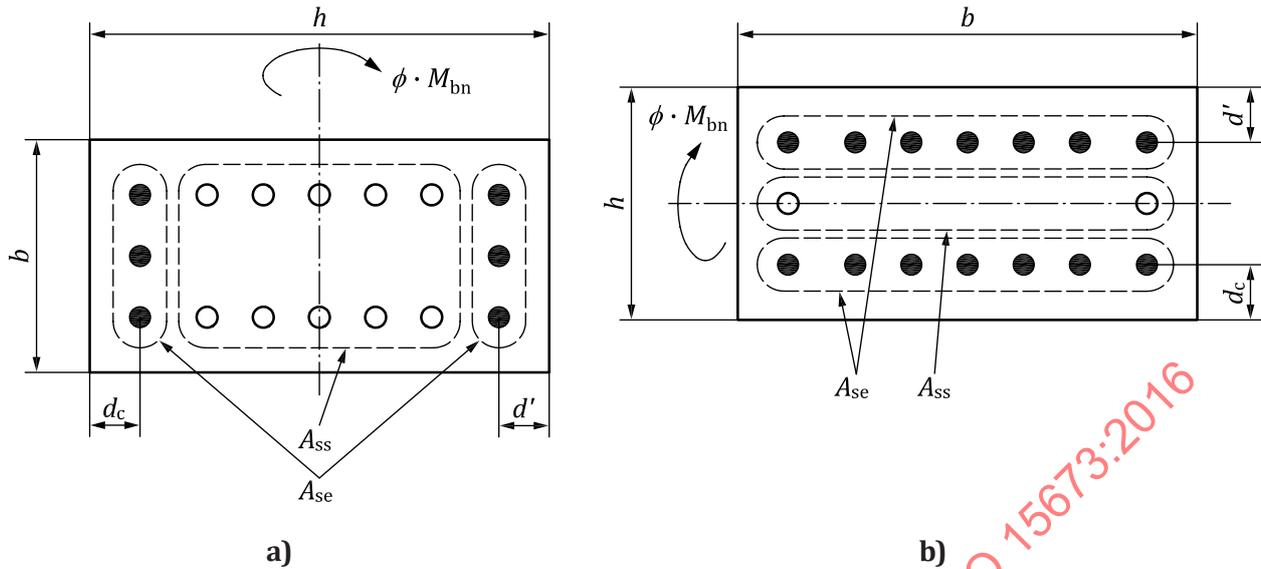


Figure 86 — Dimensions for calculation of balanced moment design strength

### 13.7.4.2 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 [Formula \(114\)](#) and [Formula \(115\)](#), respectively:

$$\phi \cdot P_{bn} = \phi \cdot 0.4 \cdot f'_c \cdot A_c \quad (114)$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0.2 \cdot h + \phi \cdot 0.5 \cdot A_{st} \cdot f_y \cdot \left( \frac{h}{2} - d' \right) \quad (115)$$

For [Formula \(115\)](#),  $h$  should be taken as the diameter of the section of the column. In [Formula \(114\)](#),  $\phi = [0,75]$  while in [Formula \(115\)](#),  $\phi = [0,90]$ ; see [6.3.3 c](#)).

### 13.7.4.3 Tension-controlled strength for axial compression with flexure

#### 13.7.4.3.1 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 tension-controlled design strength point should be determined in accordance with [Formula \(116\)](#) and [Formula \(117\)](#), respectively. However, these formulae only apply to rectangular columns with symmetrical reinforcement.

$$\phi \cdot P_{tcn} = \phi \cdot 0.18 \cdot f'_c \cdot h \cdot b \quad (116)$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0.45 \cdot h + \phi \cdot (0.95A_{se} + 0.16A_{si}) \cdot f_y \cdot \left( \frac{h}{2} - d' \right) \quad (117)$$

For [Formula \(117\)](#), the total longitudinal reinforcement area,  $A_{st}$ , should be divided into extreme steel,  $A_{se}$ , and side steel,  $A_{si}$ , in such a manner that  $A_{se} + A_{si} = A_{st}$ ; see [Figure 86](#). In [Formula \(116\)](#),  $\phi = [0,70]$  and in [Formula \(117\)](#),  $\phi = [0,90]$ ; see [6.3.3 c](#)).

### 13.7.4.3.2 Circular-section columns with spiral reinforcement

The values for axial force,  $\phi \cdot P_{bn}$ , and moment,  $\phi \cdot M_{bn}$ , at the tension-controlled design strength point should be calculated in accordance with [Formula \(118\)](#) and [Formula \(119\)](#), respectively:

$$\phi \cdot P_{tcn} = \phi \cdot 0,1 \cdot f'_c \cdot A_c \quad (118)$$

$$\phi \cdot M_{tcn} = \phi \cdot P_{tcn} \cdot 0,5 \cdot h + \phi \cdot 0,6 \cdot A_{st} \cdot f_y \cdot \left( \frac{h}{2} - d' \right) \quad (119)$$

For [Formula \(119\)](#),  $h$  should be taken as the diameter of the section of the column. In [Formula \(118\)](#),  $\phi = [0,75]$  while in [Formula \(119\)](#),  $\phi = [0,90]$ ; see [6.3.3 c](#)).

### 13.7.4.4 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 [Formula \(120\)](#):

$$\phi \cdot P_{tn} = \phi \cdot A_{st} \cdot f_y \quad (120)$$

In [Formula \(120\)](#),  $\phi = [0,90]$ ; see [6.3.3 b](#)).

### 13.7.4.5 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 [Formula \(121\)](#):

$$\phi \cdot M_n \geq M_u \quad (121)$$

Compliance with [Formula \(121\)](#) should be established by providing that the coordinates of  $(M_u, P_u)$  are inside the interaction design-strength surface of a moment against axial load interaction diagram relating  $\phi \cdot M_n$  and  $\phi \cdot P_n$ , as indicated by the hatched area in [Figure 87](#).



$$P_u \geq -(\phi \cdot P_{tn}) \quad (123)$$

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

$$M_u \leq \phi \cdot M_n = \frac{(\phi \cdot P_{0n}) - P_u}{(\phi \cdot P_{0n}) - (\phi \cdot P_{bn})} \cdot (\phi \cdot M_{bn}) \quad (124)$$

For values of  $\phi \cdot P_{tcn} \leq P_u < \phi \cdot P_{bn}$ :

$$M_u \leq \phi \cdot M_n = \frac{(\phi \cdot M_{bn}) - (\phi \cdot M_{tcn})}{(\phi \cdot P_{bn}) - (\phi \cdot P_{tcn})} \cdot (P_u - \phi \cdot P_{bn}) + (\phi \cdot M_{bn}) \quad (125)$$

For values of  $P_u < \phi \cdot P_{tcn}$ :

$$M_u \leq \phi \cdot M_n = \frac{P_u + (\phi \cdot P_{tn})}{(\phi \cdot P_{tcn}) + (\phi \cdot P_{tn})} \cdot (\phi \cdot M_{tcn}) \quad (126)$$

#### 13.7.4.6 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,  $\phi$ , as set forth in this document is warranted.

#### 13.7.4.7 Biaxial moment strength

Corner columns and other columns subjected to moments about each axis simultaneously should be in accordance with [Formula \(127\)](#):

$$\frac{(M_u)_x}{(\phi \cdot M_n)_x} + \frac{(M_u)_y}{(\phi \cdot M_n)_y} \leq 1,0 \quad (127)$$

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  $P_u$ .  $(\phi \cdot M_n)_x$  and  $(\phi \cdot M_n)_y$  correspond to the values of the values of the design moment strength derived from [Formula \(124\)](#), [Formula \(125\)](#) or [Formula \(126\)](#) for the factored axial load value,  $P_u$ , and for the appropriate direction x or y.

## 14 Structural concrete walls

### 14.1 General

The design of structural concrete walls should be performed using the guides of [Clause 14](#). Both in-plane and out-of-plane effects on reinforced concrete structural walls are covered.

### 14.2 Design load definition

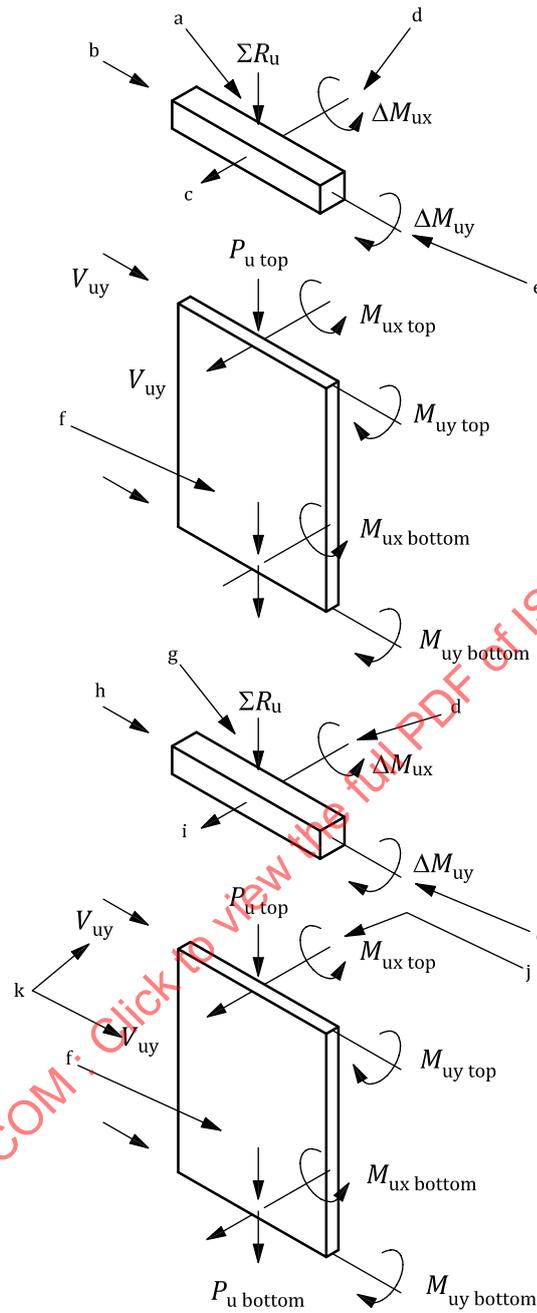
#### 14.2.1 Loads to be included

The design load for structural concrete walls should be established from the guides of [Clause 8](#). The loads that should be included in the design are as follows(see [Figure 88](#)).

- a) Tributary live and dead loads from the tributary structural elements from each floor located above. Tributary loads should be established from the guides of [Clause 8](#) and the particular guides of each tributary element type.

- b) Selfweight of the structural concrete wall.
- c) Lateral forces from wind, earthquake or soil lateral pressures.

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- a Actions at the joint of story  $n$  from tributary elements.
- b Lateral force applied to the wall at story  $n$  in direction  $x$ .
- c Lateral force applied to the wall at story  $n$  in direction  $y$ .
- d Unbalanced moment from tributary story elements in direction  $x$ .
- e Unbalanced moment from tributary story elements in direction  $y$ .
- f Selfweight.
- g Actions at the joint of story  $n-1$  from tributary.
- h Lateral force applied to the wall at story  $n-1$  in direction  $x$ .
- i Lateral force applied to the wall at story  $n-1$  in direction  $y$ .
- j  $P_u$  top of story  $n-1$  wall equal to  $P_u$  bottom from story  $n$  plus  $\Sigma R_u$  from story  $n-1$ .
- k  $V_u$  of story  $n-1$  wall equal to  $V_u$  from story  $n$  plus lateral force from story  $n$ .

**Figure 88 — Structural concrete wall applied factored design forces**

14.2.2 Dead load and live load

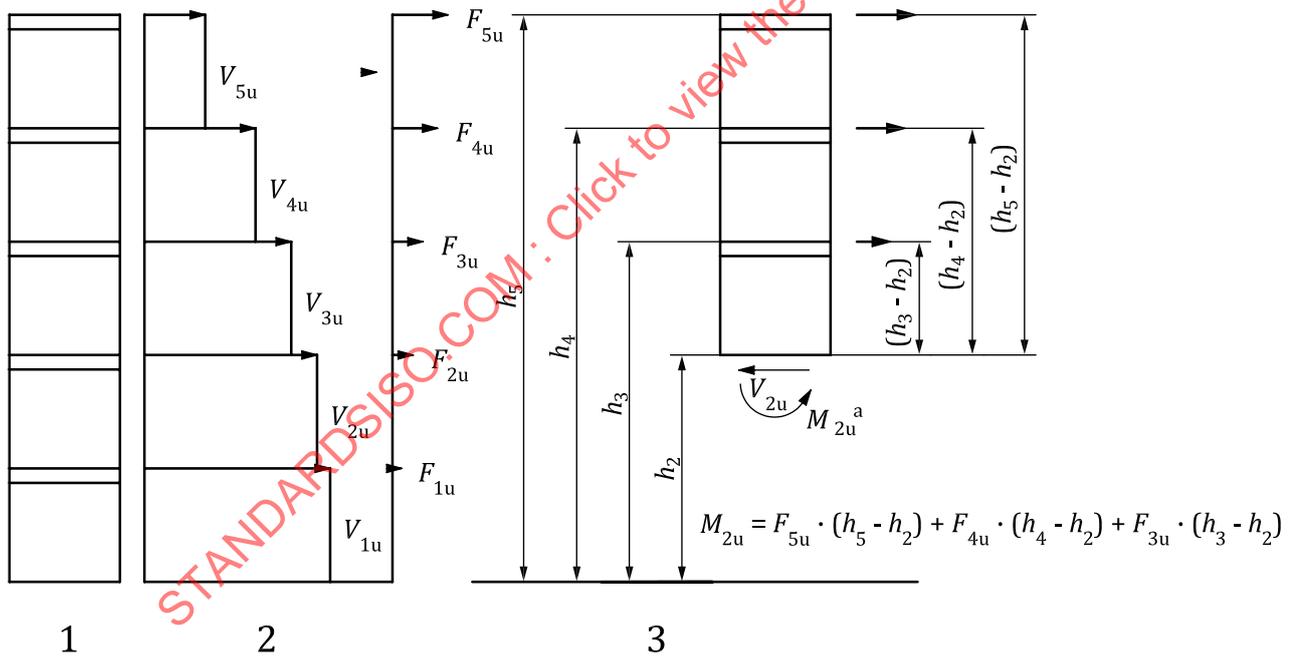
The values of  $P_d$  for dead load and  $P_l$  for live load should be in N.  $P_d$  should include the selfweight of the structural concrete wall, at  $24 \times 10^3$  N/m<sup>3</sup>. The selfweight should be factored employing the load factors for dead load of the corresponding combination formula from 8.1.1.1. It should be permitted to apply the selfweight of the wall corresponding to each floor at the lower part of the structural concrete wall in that floor. The value of the unbalanced moment caused by vertical loads should be obtained from the guides of the supported element.

14.2.3 Lateral design load

The value of the factored lateral load moment,  $M_u$ , (see Figure 89) should be established at the upper and lower part of the structural concrete wall in both principal plan direction at each story in the following manner.

- a) The lateral load factored shear at each story  $x$ ,  $V_{xu}$ , should be obtained for the wall from the guides of Clause 16.
- b) The factored lateral force applied at each story  $x$ ,  $F_{xu}$ , should be obtained as the difference in factored shear between the two neighbouring stories,  $V_{xu}$  and  $V_{(x+1)u}$ .
- c) Factored lateral load moment,  $M_{xu}$ , at each story  $x$  should be obtained employing Formula (128).

$$M_{xu} = \sum_{i=x}^n [F_{iu} \cdot (h_i - h_x)] \tag{128}$$



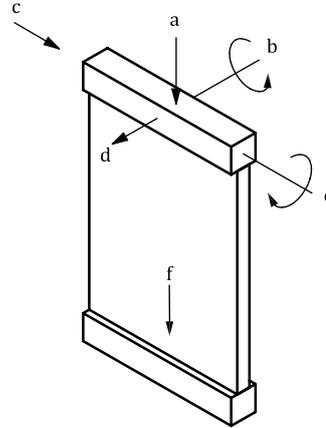
Key

- 1 wall
- 2 wall shears
- 3 wall lateral forces
- a Lateral load factored design forces at level 2.

Figure 89 — Calculation of the lateral load factored moment

#### 14.2.4 Factored design load

The value of the factored design forces  $P_u$ ,  $V_u$ , and  $M_u$  should be established at the upper and lower part of the structural concrete wall in each story. A distinction should be made about the direction of the horizontal forces, dividing them into in-plane and out-of-plane forces. See [Figure 90](#).



- a Axial force.
- b In-plane moment.
- c In-plane shear.
- d Out-of-plane shear.
- e Out-of-plane moment.
- f Selfweight.

Figure 90 — In-plane and out-of-plane forces

### 14.3 Dimensional guidelines

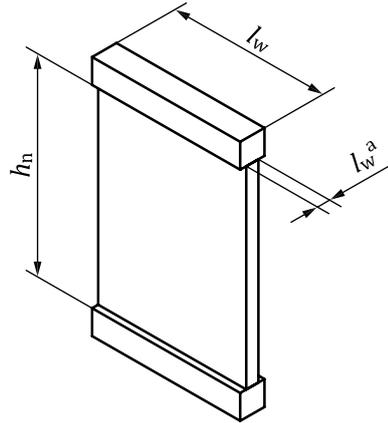
#### 14.3.1 General

In addition to the appropriate guides of [14.3](#), structural concrete walls should comply with the general dimensional guides set forth in [6.1](#) and [16.4.1](#). Structural concrete wall section shape should be rectangular. All other cross-section shapes are beyond the scope of this document, with the exception permitted by [14.3.2.2](#). Structural concrete walls should be aligned vertically and should be continuous all the way down to the foundation.

#### 14.3.2 Limiting dimensions

##### 14.3.2.1 Minimum thickness structural concrete walls

Under the present guidelines, the thickness of structural concrete walls should not be less than 150 mm (see [Figure 91](#)) nor  $1/25$  of the length of the wall  $l_w$ , and at changes of thickness in contiguous stories, the guides of [16.4.1 c](#)) should be met.



$$a \quad b_w \geq \begin{cases} 150 \text{ mm} \\ h_n / 20 \\ l_w / 25 \end{cases}$$

Figure 91 — Minimum cross-section dimensions for rectangular structural concrete walls

#### 14.3.2.2 Columns embedded in walls

When columns are built monolithically embedded in walls, it should be permitted to make the wall thicker at the location of the column, without having to increase the thickness of the whole wall cross-section. The transverse dimension of the column should meet the guides of [13.3.3](#). It should be permitted to place the increase of thickness at one side of the cross-section.

#### 14.3.2.3 Distance between lateral supports

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 wall. See [Figure 91](#). The clear distance between vertical lateral supports,  $h_n$ , for structural concrete walls should not exceed 20 times the thickness of the structural concrete wall.

#### 14.3.2.4 Beams on top of walls

Beams or girders should be provided for the full horizontal length of the wall at every floor and roof supported by the structural wall. These beams or girders should comply with the guides of [12.3.5.2.2](#) and should be reinforced as collector elements following the guides of [12.3.5.3](#).

### 14.4 Details of reinforcement

#### 14.4.1 General

For the purposes of these guidelines, the reinforcement of structural concrete walls should be of the types described in this subclause and should comply with the guides of [14.4.2](#) to [14.4.4](#).

## 14.4.2 Number of curtains of reinforcement

### 14.4.2.1 Two curtains of reinforcement

Two curtains of reinforcement parallel with the faces of the wall should be employed in the following cases:

- a) when the wall is more than 250 mm thick;
- b) in walls where the vertical reinforcement ratio,  $\rho_v$ , exceeds 0,01 (see [9.5.5.2](#) and [14.4.3.2](#));
- c) in walls where the in-plane factored shear force,  $V_u$ , in the wall exceeds  $(\phi \cdot V_c)$ .

The division of reinforcement into layers and their location within the wall section should comply with [9.3.21.2](#).

### 14.4.2.2 One curtain of reinforcement

In all the other cases not covered by [14.4.2.1](#), it should be permitted to employ only one curtain of reinforcement located in the centre of the thickness of the wall.

## 14.4.3 Vertical reinforcement

### 14.4.3.1 Description

Vertical reinforcement should consist in one or two layers of bars or welded-wire reinforcement placed parallel with the faces of the walls. The amount of vertical reinforcement should be that required to resist the simultaneous action of a combination of factored axial load and factored moments at the section acting about the two main axis of the section of the structural concrete wall.

### 14.4.3.2 Minimum and maximum vertical reinforcement area

The maximum and minimum vertical reinforcement area should comply with the guides of [9.5.5](#). When the amount and separation of vertical reinforcement vary within the wall cross-section or columns are built monolithically embedded within the wall cross-section, the following guides should be met.

- a) Where the vertical reinforcement is concentrated either by increasing the vertical bar diameter or reducing the spacing between bars, the vertical reinforcement ratio,  $\rho_v$ , in that portion of the wall should not exceed maximum vertical reinforcement ratio set forth by [9.5.5.2](#). The vertical reinforcement ratio should be evaluated over an area bounded by the faces of the wall and 50 mm measured along the length of wall from the last bars with a closer spacing or larger diameter. See [Figure 92 a](#)).
- b) Where the vertical reinforcement is reduced either by separating it further apart or by decreasing the vertical bar diameter, the vertical reinforcement ratio,  $\rho_v$ , should not be less than the minimum vertical reinforcement ratio set forth by [9.5.5.1](#) at any place within the wall cross-section. See [Figure 92 b](#)).