INTERNATIONAL STANDARD

ISO 15673

Second edition
2016-12-01

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

Reference number
ISO 15673:2016(E)

International Organization for Standardization

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ISO copyright office
Ch. de Blandonnet 8 • CP 401
CH-1214 Vernier, Geneva, Switzerland
Tel. +41 22 749 01 11
Fax +41 22 749 09 47
copyright@iso.org
www.iso.org

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Contents

Page

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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).

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

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

For an explanation 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.

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.

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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 regulartory 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.

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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*

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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^2.

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

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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)

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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 *crosstie* (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
crosstie
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 crossties 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

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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, ϕ* (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

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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)

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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 continuosly 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

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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

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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)

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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

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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
ϕ
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

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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.

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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^2
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^2
A_g	gross area of section of element	mm^2
A_j	effective cross-sectional area within a joint for shear evaluation or area of additional hanger reinforcement, where beams are supported by girders or other beams	mm^2
A_s	area of longitudinal tension reinforcement	mm^2
$A_S^{'}$	area of longitudinal compression reinforcement	mm^2
$A_{s,min}$	minimum area of longitudinal tension reinforcement	mm^2
A_{se}	total extreme steel area in a column or structural concrete wall for computation of the balanced moment strength	mm^2
A_{si}	total side steel area in a column or structural concrete wall for computation of the balanced moment strength	mm^2
A_{st}	total area of longitudinal reinforcement	mm^2
A_{su}	wind exposed surface area	m^2
A_v	area of shear reinforcement within a distance s	mm^2
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

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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 · f_y)
f_{ys}	specified yield strength of transverse or spiral reinforcement	MPa
f_{ysd}	yield strength of transverse or spiral reinforcement reinforcement reduced by the material factor	MPa
F	loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces	
F_a	site soil coefficient	
F_i, F_x	design wind or seismic force applied at level i or x, respectively	kN
F_{iu}, F_{xu}	factored design lateral force applied to the wall at level i or x, respectively	N
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^4
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

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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
ΔM_u	factored unbalanced moment at a column-girder joint or factored unbalanced moment at a wall-girder joint	N · m
P_d	non factored dead load axial force at section or non factored concentrated dead load applied directly to the element	N
P_{bn}	nominal compression axial load strength at section at balanced conditions	N
P_{br}	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^2
q_l	non-factored live load per unit area	N/m^2
q_u	factored load per unit area	N/m^2
r_u	factored uniformly distributed reaction from the slab on the supporting girder, beam or structural concrete wall	N/m
R_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

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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
ΔV_e	factored design shear force from the development of the probable flexural capacity of the element at the faces of the joints	N
V_{iu}, V_{xu}	factored story shear caused by lateral loads at story i or x, respectively	N
V_n	nominal shear strength at section	N
V_s	contribution of the 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
α_a	fraction of the load that travels in the short direction in two-way slabs-on-girders	
α_b	fraction of the load that travels in the long direction in two-way slabs-on-girders	
α_s	constant used to compute nominal punching shear strength in slabs	
β	ratio of clear spans in long to short direction of two-way slabs	
ϕ	strength reduction factor	
ρ	ratio of longitudinal tension reinforcement	$\frac{A_s}{b \cdot d}$
ρ'	ratio of longitudinal compression reinforcement	
ρ_h	ratio of horizontal reinforcement in structural concrete walls	
ρ_{max}	maximum permissible ratio of longitudinal flexural tension reinforcement	
ρ_{min}	minimum permissible ratio of longitudinal flexural tension reinforcement	
ρ_s	ratio of spiral reinforcement	
ρ_t	ratio of total longitudinal reinforcement area to gross concrete section area	$\frac{A_{st}}{b \cdot d}$
ρ_v	ratio of vertical reinforcement in structural concrete walls	

4.2 Abbreviated terms

col. column

deg. degrees

max. maximum

min. minimum

No. number

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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 than 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.

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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.

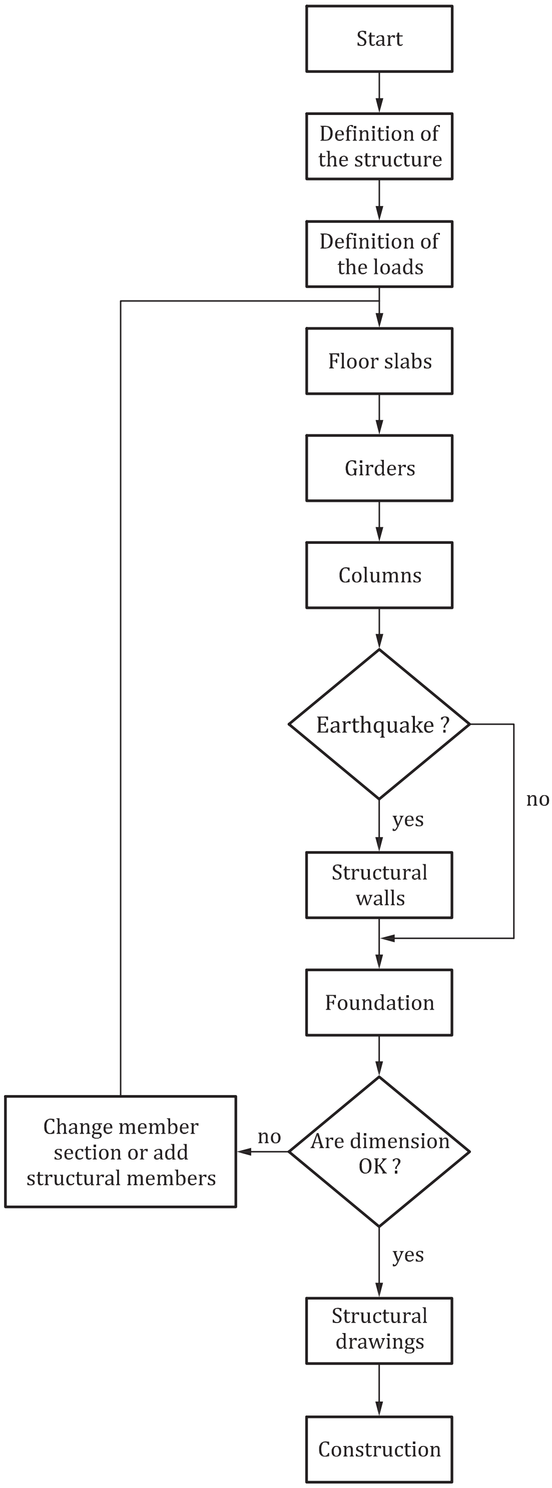

Figure 1 — Design procedure under the standard

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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.

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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^2.

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.

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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.

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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, ϕ, less than 1,0, and load factors, γ, generally greater than 1,0, should be employed, as given in Formula (2):

$$\phi \cdot R_\text{n} \geq \gamma_1 \cdot S_1 + \gamma_2 \cdot S_2 + \ldots \tag{2}$$

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

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

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

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 ϕ.

a) Flexure, without axial load ϕ = [0,90]

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

c) Axial compression and axial compression with flexure:

  1) columns with ties and structural concrete walls ϕ = [0,65]

  2) columns with spiral reinforcement ϕ = [0,70]

d) Shear and torsion ϕ = [0,75]

e) Bearing of concrete ϕ = [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

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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.

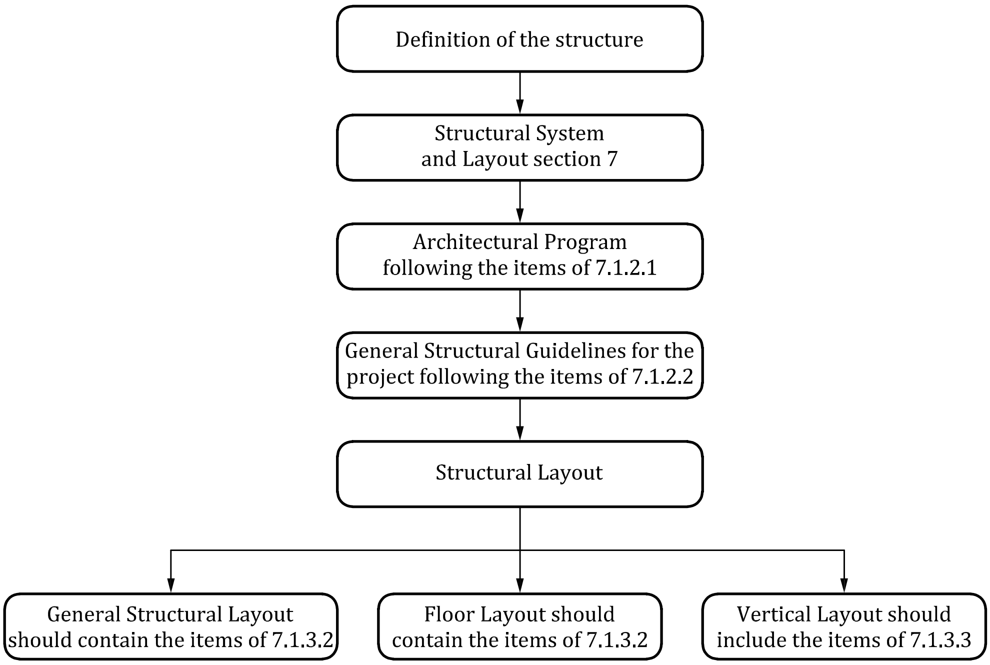

Figure 2 — Definition of the structure

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

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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.

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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.

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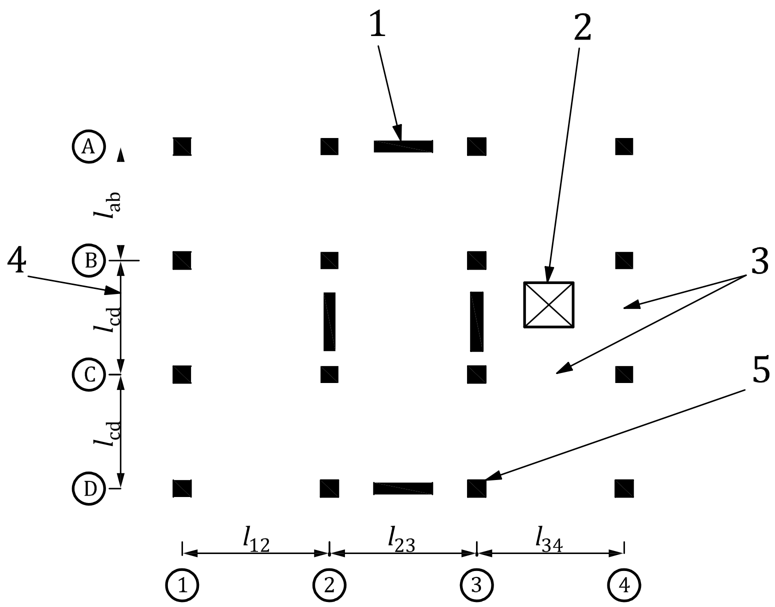
f) In seismic zones, the location and distribution of all structural concrete walls.



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;

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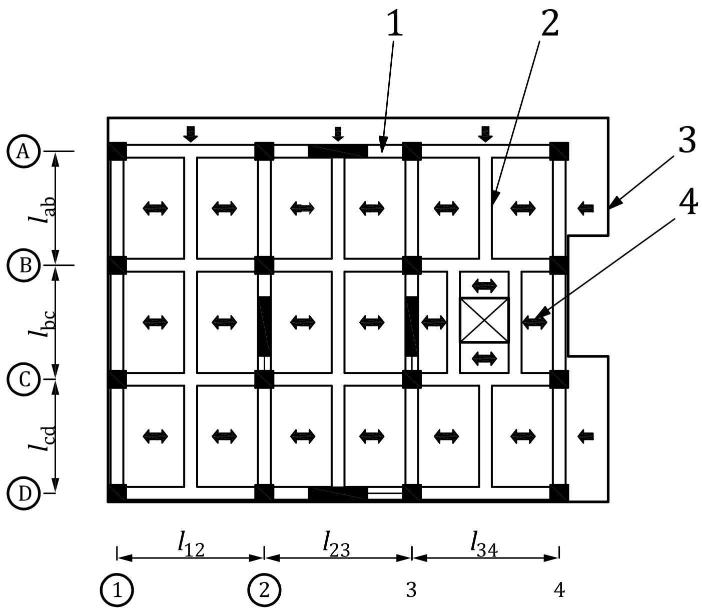
e) vertical space necessary to accommodate horizontal/vertical service elements for power, water supply and drainage, heating, ventilation, and air conditioning;

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 gider
2 beam
3 structural perimeter
4 load path

Figure 4 — Typical floor structural layout

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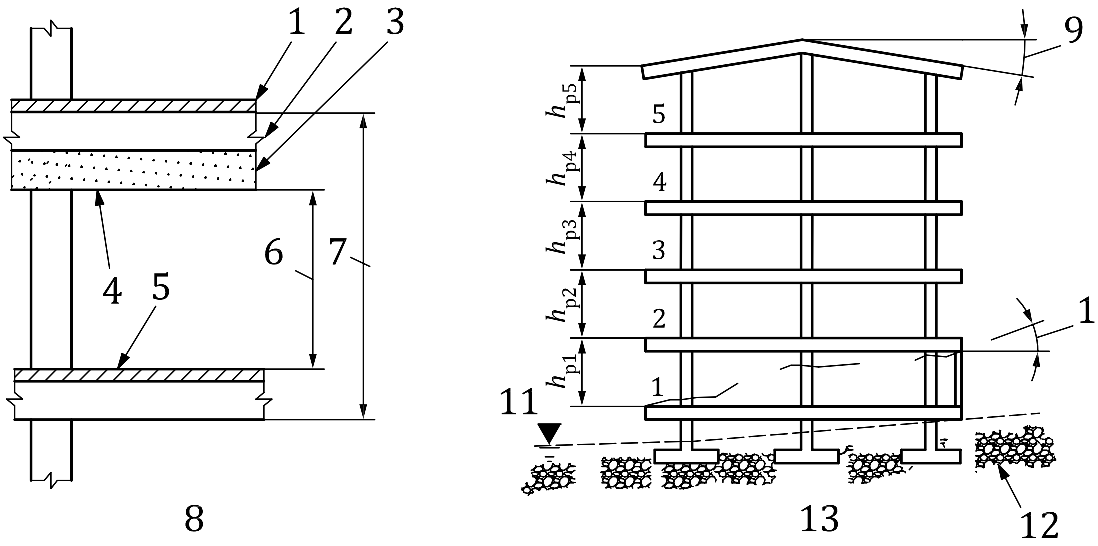

Key

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

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.

a) 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.

b) The number of stories should not exceed the maximum permitted, given in 6.1.2.

c) The area of the largest floor should not exceed the maximum permitted area of 6.1.3.

d) 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.

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

f) The difference between adjacent spans should not exceed the limits of 6.1.6.

g) 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.

h) No girder, beam or slab cantilever, should exceed the limits of 6.1.8.

i) No sloping girder, beam, joist, or slab should have a slope greater than the maximum permitted value of 6.1.9.

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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 ± 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

$$U = \left[1{,}4\right] \cdot D \tag{5}$$

$$U = \left[1{,}2\right] \cdot D + \left[1{,}6\right] \cdot L \tag{6}$$

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):

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

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

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):

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

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

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

$$U = [0,9] \cdot D \pm [1,6] \cdot W \tag{10}$$

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):

$$U = [1,2] \cdot D + [1,0] \cdot L \pm [1,0] \cdot E + [0,2] \cdot S \tag{11}$$

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

$$U = [0,9] \cdot D \pm [1,0] \cdot E \tag{12}$$

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

$$U = [1,2] \cdot D + [1,6] \cdot L + [1,6] \cdot H \tag{13}$$

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

$$U = [0,9] \cdot D + [1,6] \cdot H \tag{14}$$

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] · 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] · 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 preformed using this document and the appropriate standard of each country should be employed.

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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, *ρ*, for evaluation of dead loads from materials

Material	Density, ρ (kg/m^3)	Material	Density, *ρ* (kg/m^3)
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 unfilles	1 144
Sand and gravel, wet	1 907	Tin	7 294
Glass	2 543	Water	
Gravel, dry	1 653	Fresh	985

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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^2).

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.

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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

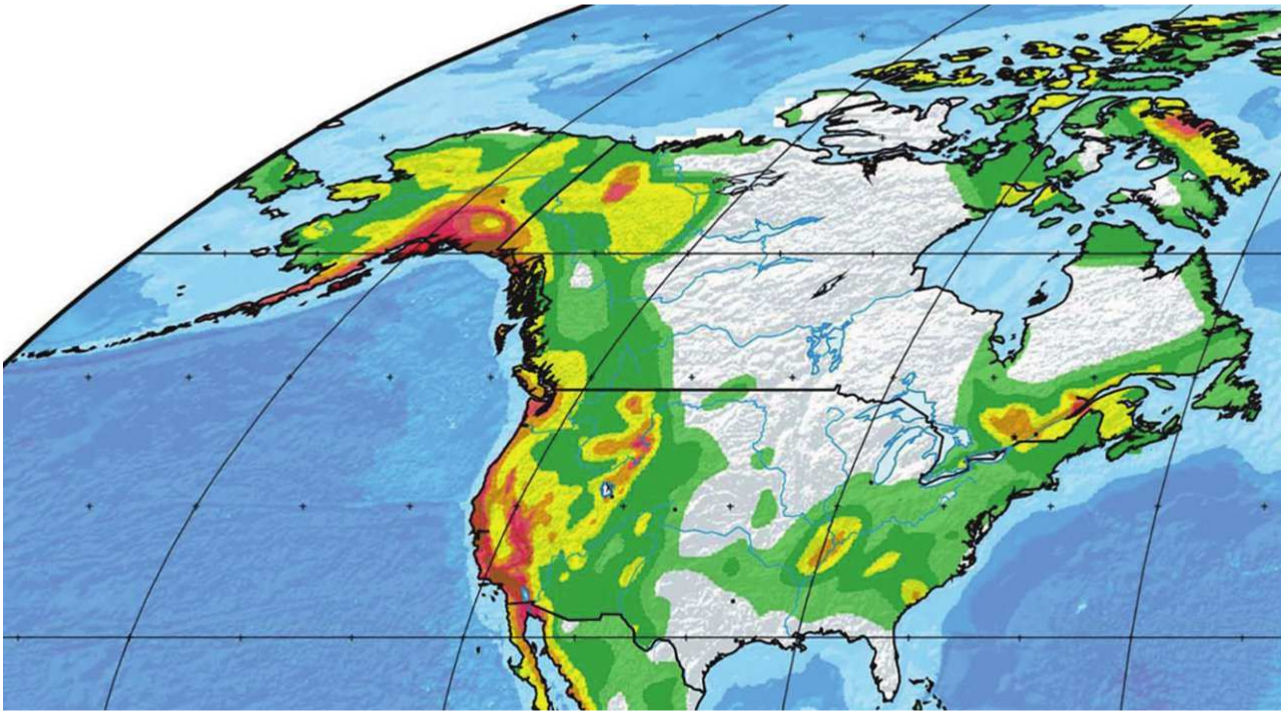
A zone where the value of A_a is estimated as more than [0,05] but less or equal to [0,10], may be deemed as a ***low seismic hazard*** zone.

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 sesimc 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

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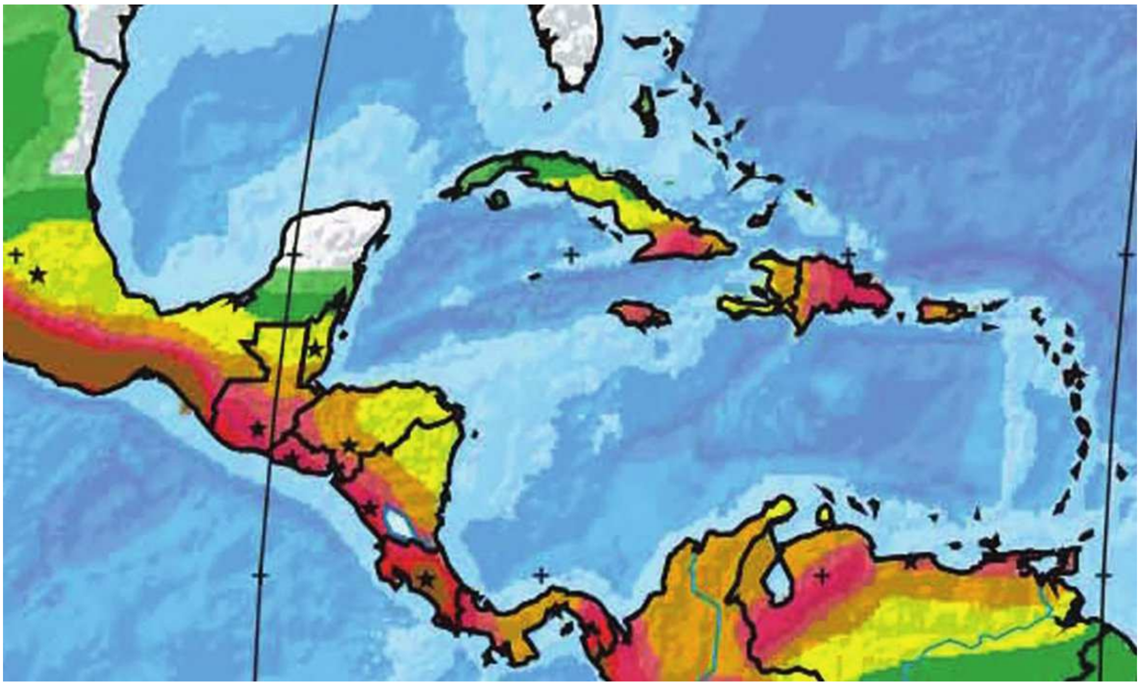

b) Central America and the Caribbean

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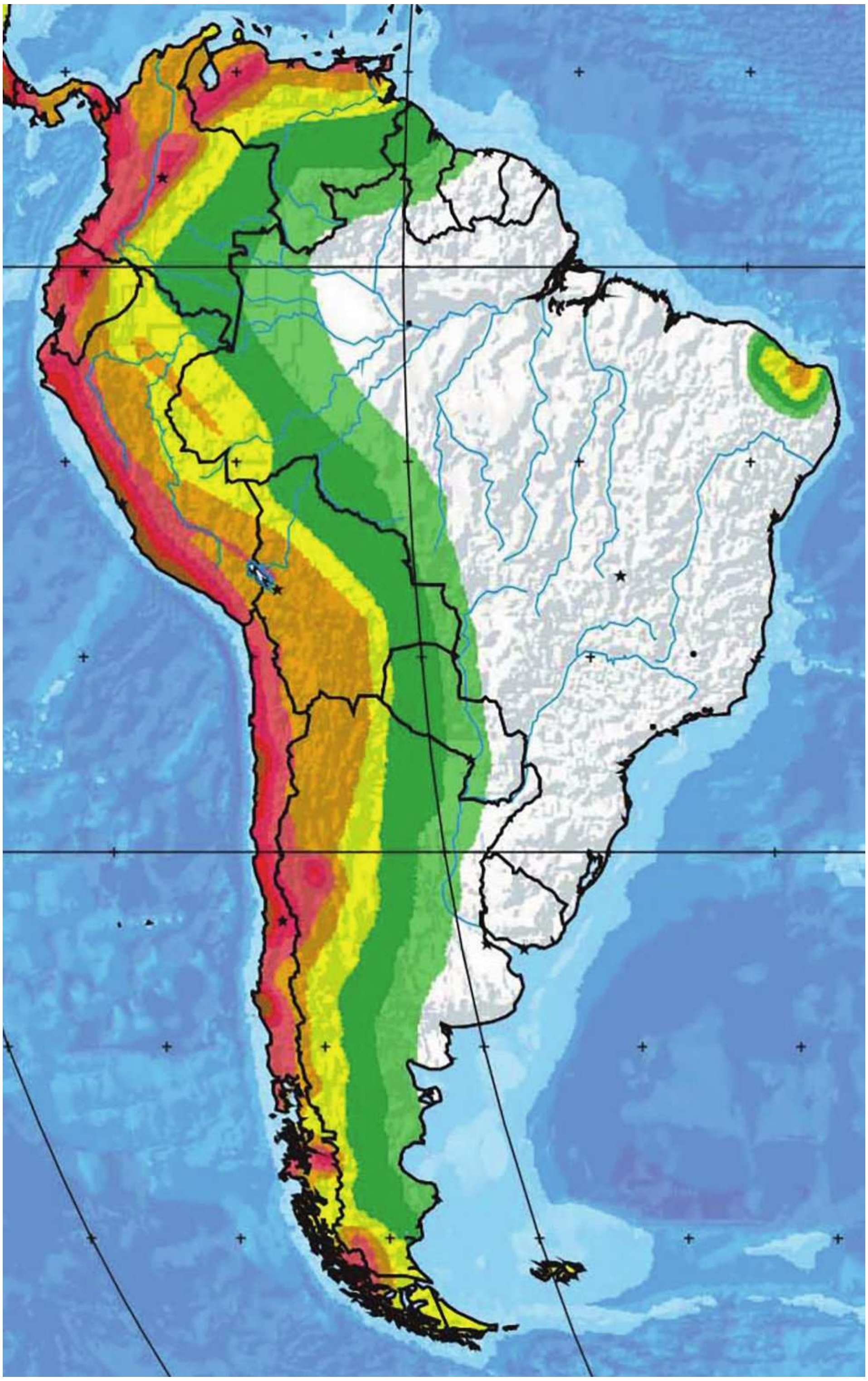

c) South America

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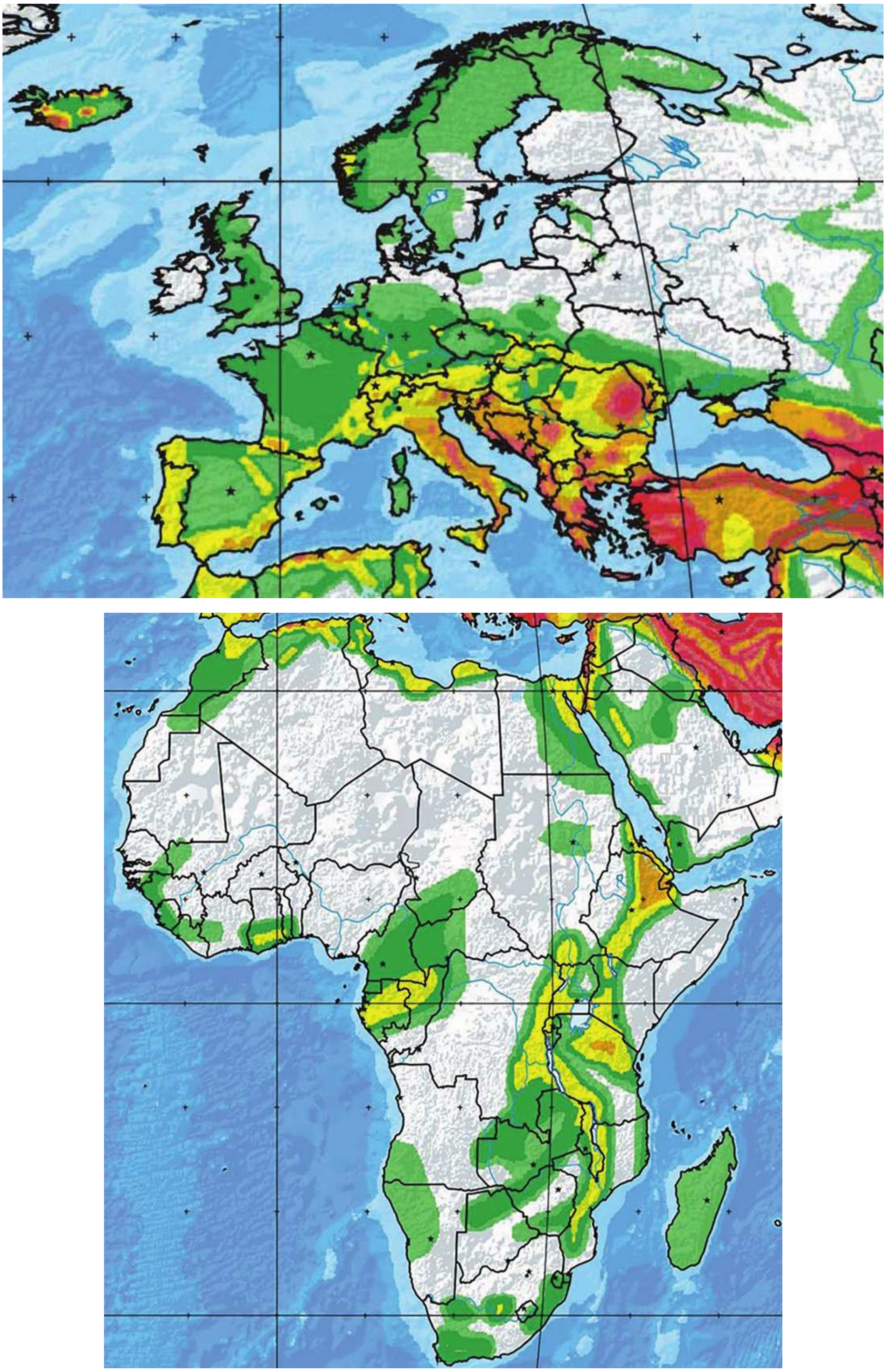

d) Europe and Africa

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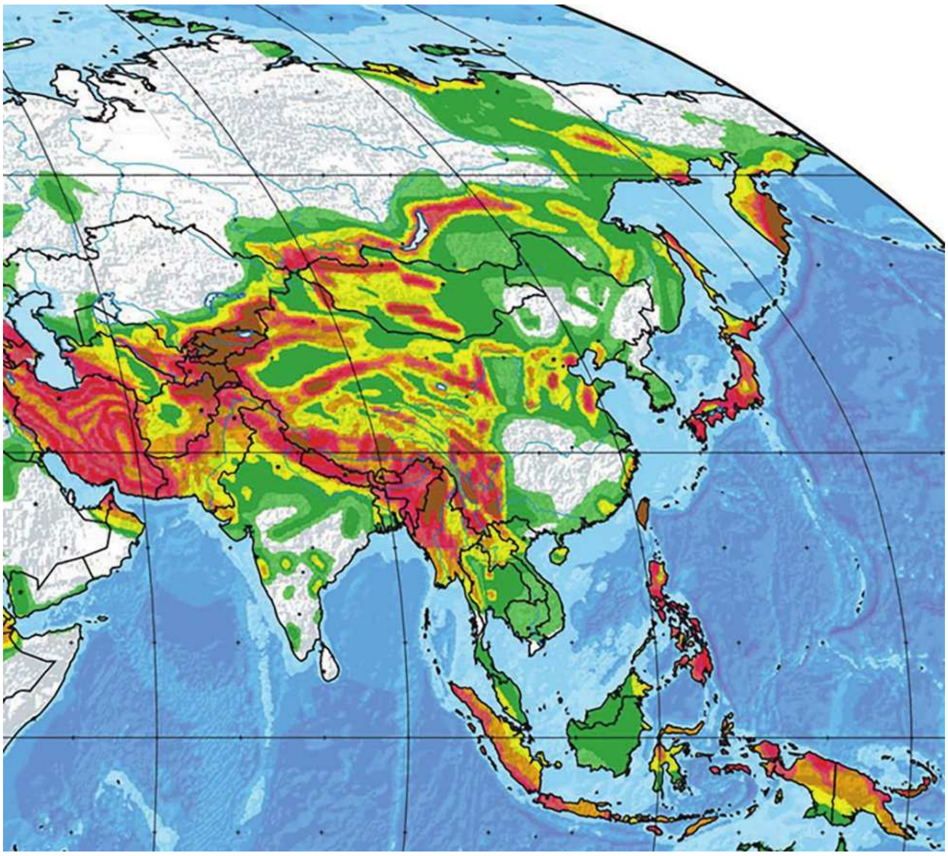
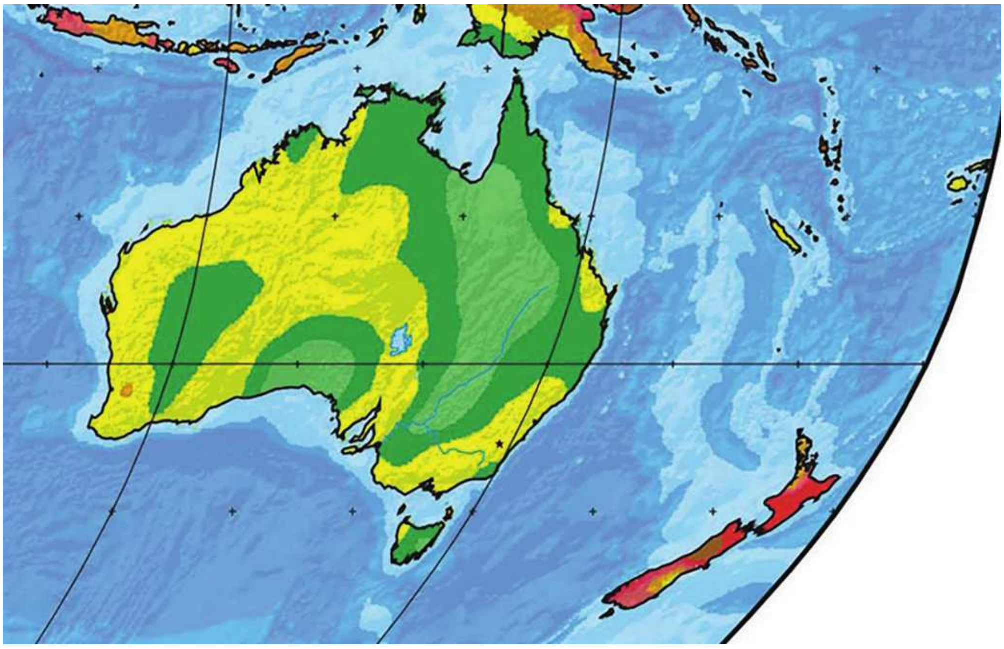


e) Asia



f) Oceania

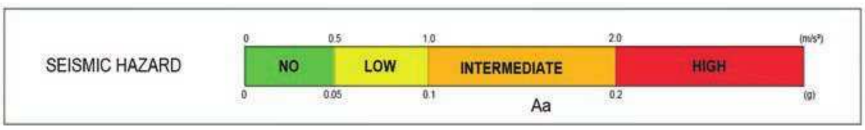


Figure 6 — Global seismic hazard map

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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_A: hard rock with a measured shear wave velocity, v_s > 1 500 m/s.

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

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

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

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

— 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 classsify 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 ≤ [0,1]	A_a = [0,2]	A_a = [0,3]	A_a = [0,4]	A_a ≥ [0,5]
S_A	[0,80]	[0,80]	[0,80]	[0,80]	[0,80]
S_B	[1,00]	[1,00]	[1,00]	[1,00]	[1,00]
S_C	[1,20]	[1,20]	[1,10]	[1,00]	[1,00]
S_D	[1,60]	[1,40]	[1,20]	[1,10]	[1,00]
S_E	[2,50]	[1,70]	[1,20]	[0,90]	[0,90]

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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):

$$S_a = 2{,}5\, A_a F_a \tag{15}$$

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):

$$V_s = \frac{S_a \cdot W}{R} \tag{16}$$

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^2.

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.

$$F_x = C_{vx} \cdot V_s \tag{17}$$

and

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

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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,

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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 9.3.5.2)	10 mm	25 mm
b) Wire for welded-wire reinforcement (see 9.3.5.3)	4 mm	10 mm
c) For stirrups and ties	6 mm	16 mm
d) Plain reinforcing bars (see 9.3.5.4)	6 mm	16 mm

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9.3.9 Concrete mixture specification

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

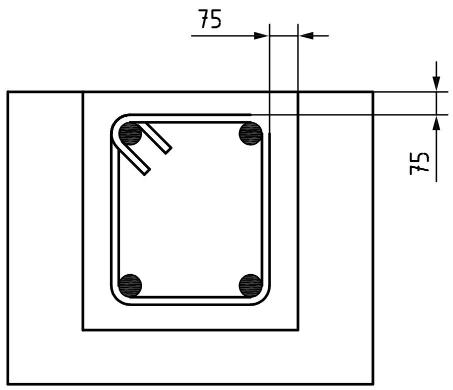
— ISO 22965-1;

— ISO 22965-2.

9.3.10 Concrete cover to reinforcement

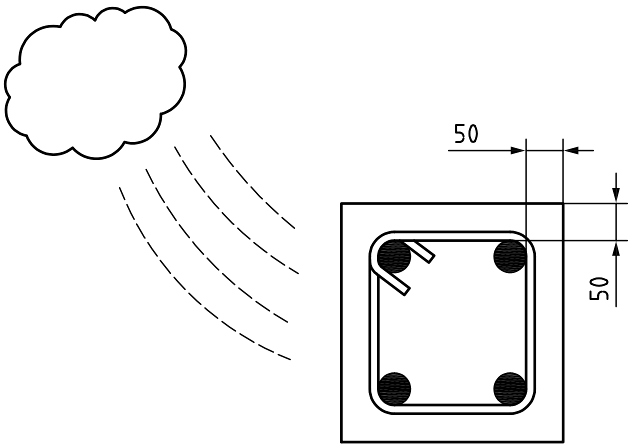
9.3.10.1 Minimum concrete cover

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

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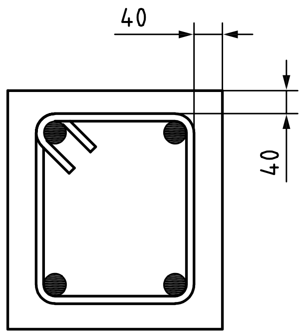
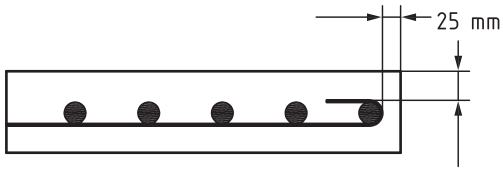


NOTE Minimum concrete cover 40 mm.

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.

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

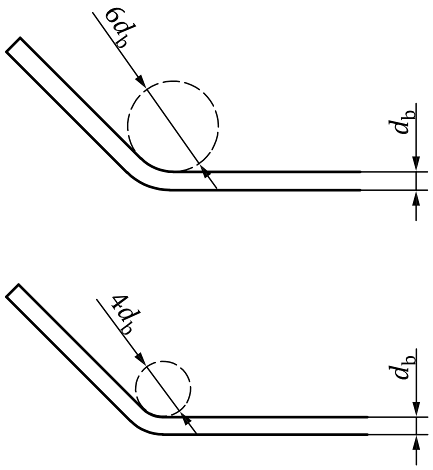

Figure 11 — Minimum reinforcement bend diameter

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

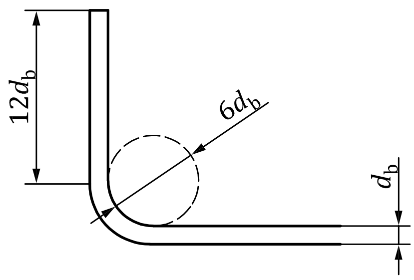

Figure 12 — 90° hook

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

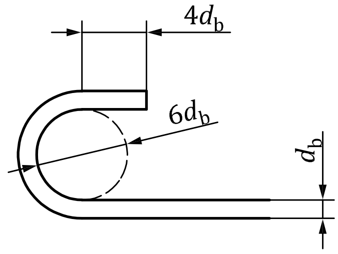

Figure 13 — 180° hook

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c) For stirrup and tie hooks

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

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

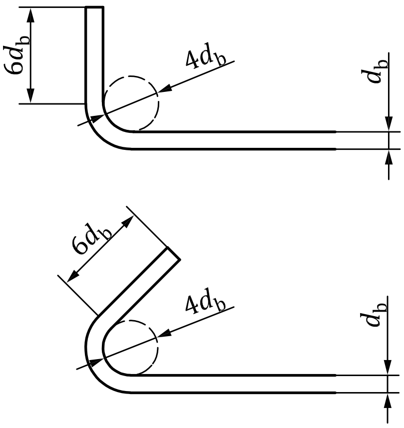

Figure 14 — For stirrup and tie hooks

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

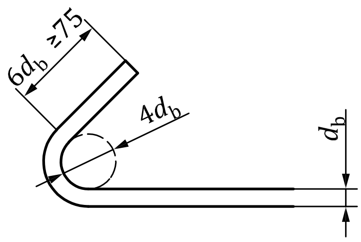

Figure 15 — For confinement stirrups and ties in seismic zones

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:

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

b) 1/3 of the depth of slabs;

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

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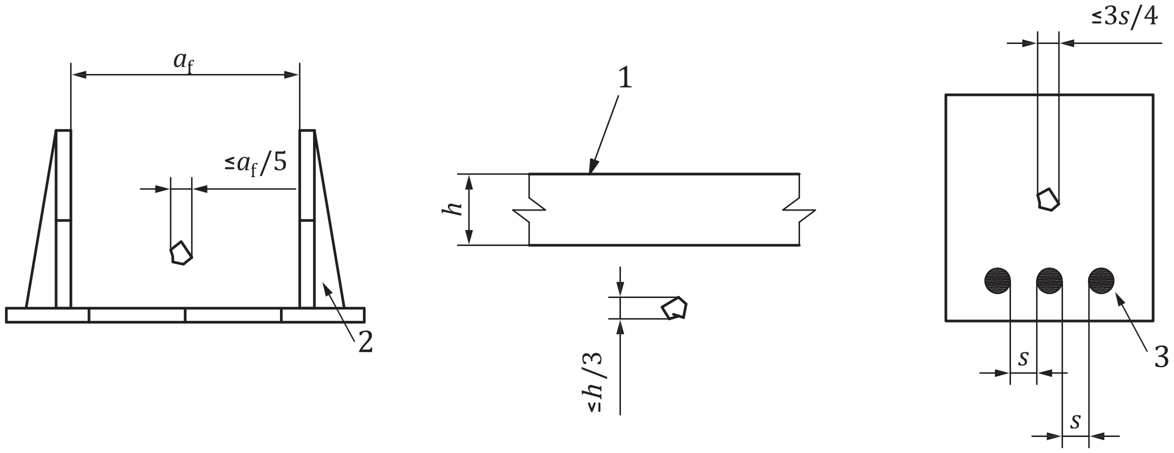

Key

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.

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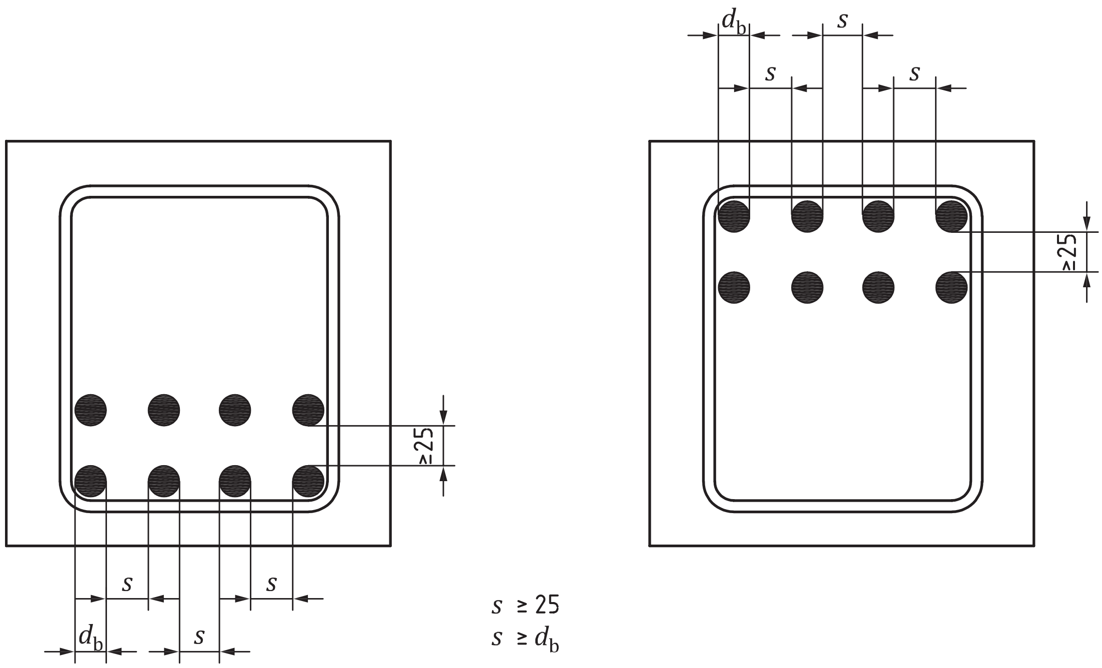

Figure 17 — Minimum clear spacing between parallel bars in a layer and clear distance between parallel layers of reinforcement

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.

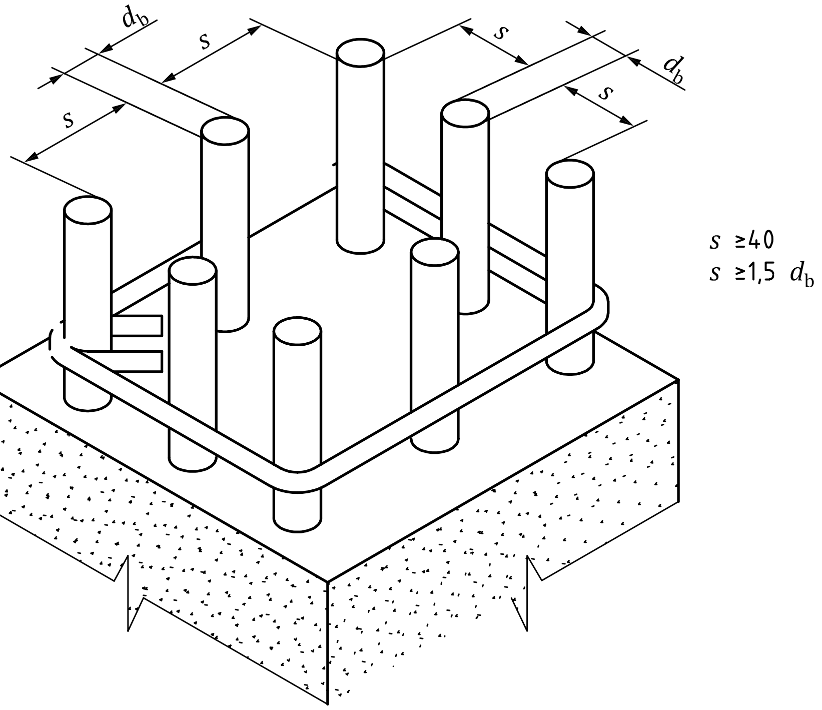

Figure 18 — Clear distance between longitudinal bars in columns

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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).

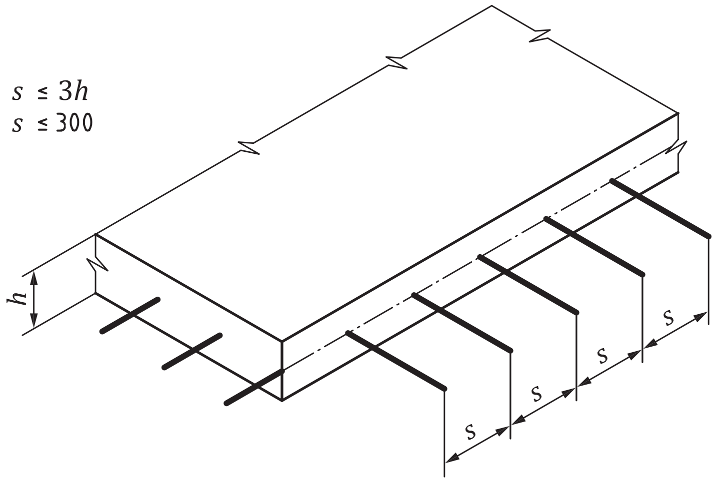

Figure 19 — Spacing between flexural reinforcement in solid slabs

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).

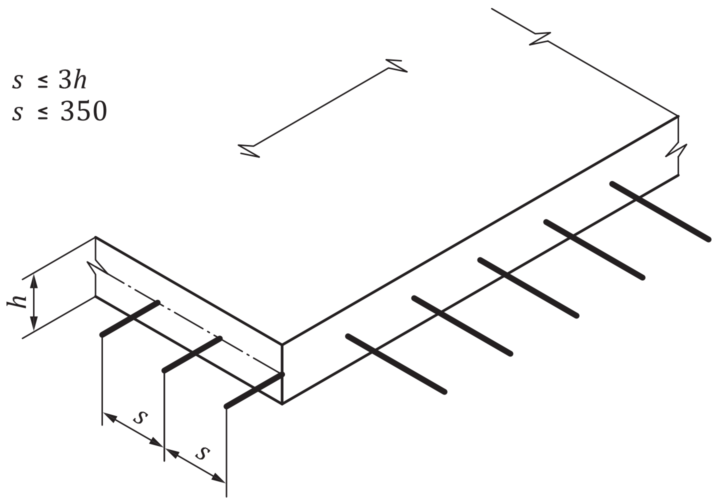

Figure 20 — Spacing between shrinkage and temperature reinforcement in slabs

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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.

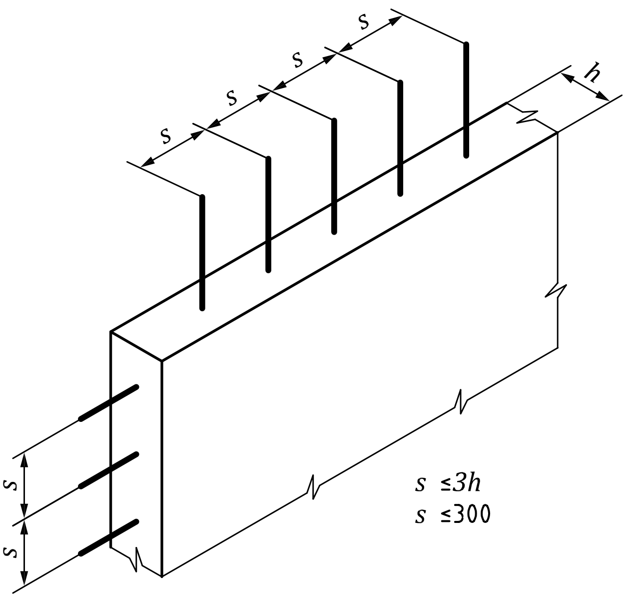

Figure 21 — Spacing between reinforcement in structural concrete walls

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.

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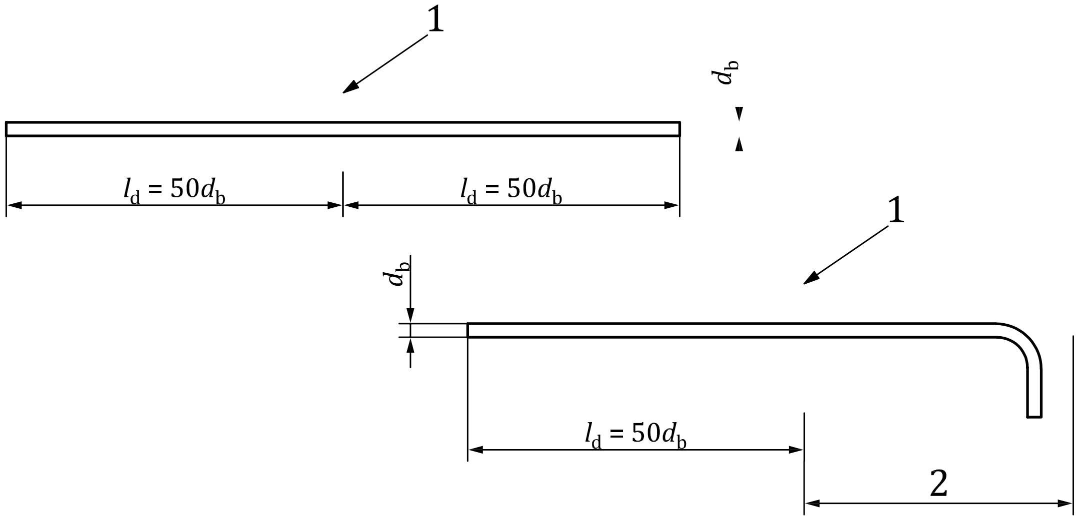


Key

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.

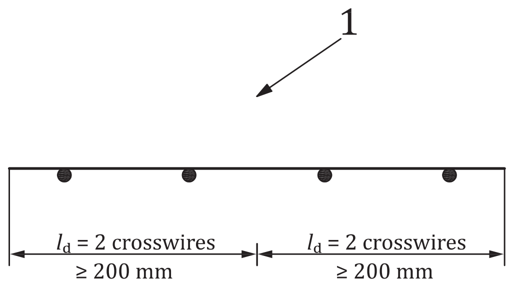
In a more accurate manner, the minimum length of embedment, l_d, required on each side of a critical section, for a reinforcing bar to develop its full strength in tension may be calculated to a more accurate value by using Formula 19:

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

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

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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.

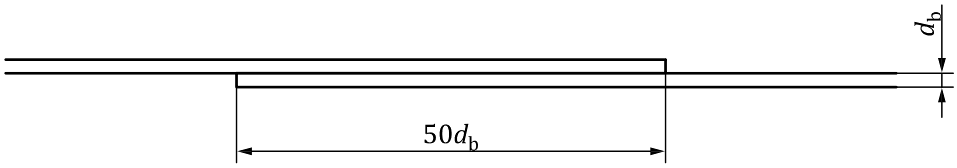


Figure 24 — Minimum lap splice length for reinforcing bars

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.

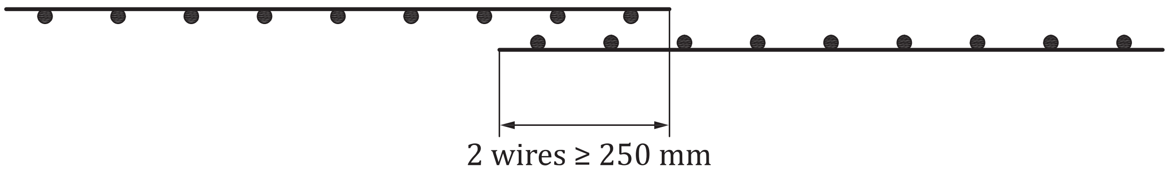
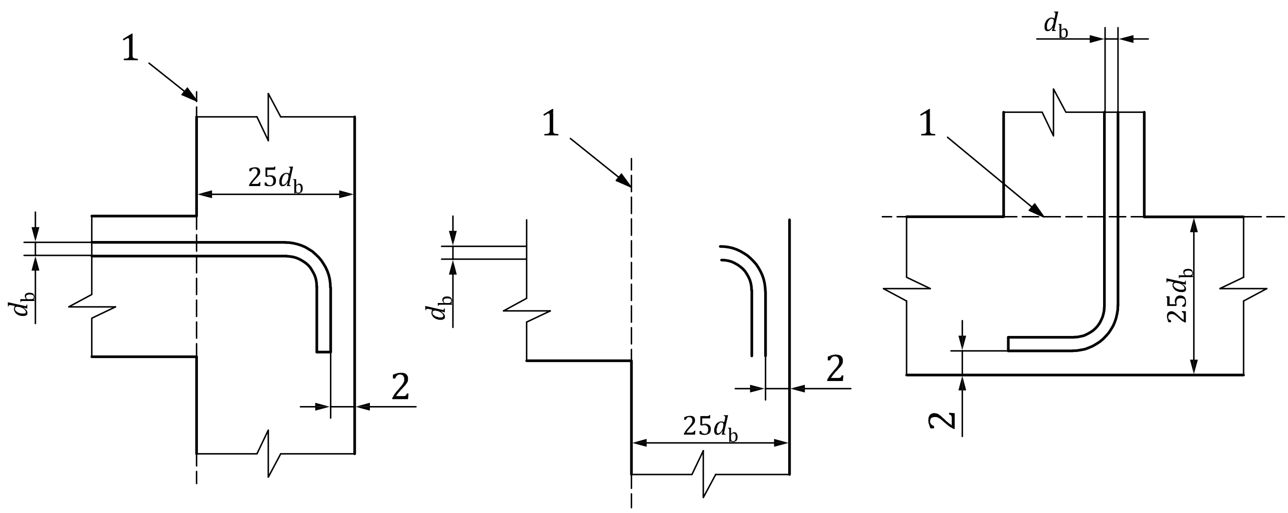


Figure 25 — Minimum lap splice length for welded-wire reinforcement

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

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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, ρ_t, should be provided for shrinkage and temperature:

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

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

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.

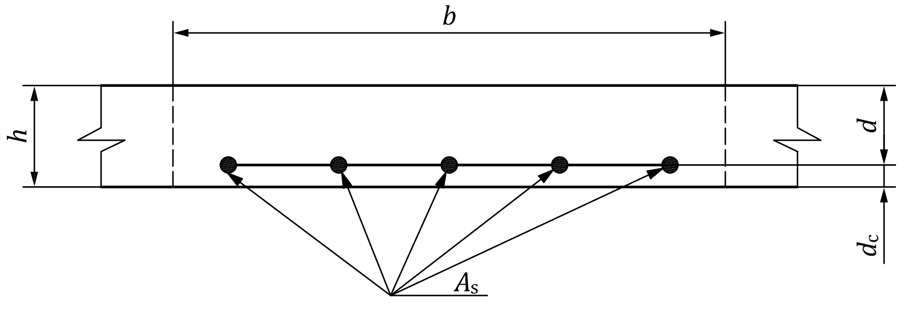

Figure 27 — Slab or footing section

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 ρ_{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.

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Table 4 — Maximum flexural reinforcement ratio, ρ_{max} for solid slabs and footings

		f_y (MPa)		
		240	**300**	**400**
f'_c (MPa)	20	0,022 0	0,016 0	0,011 0
	25	0,027 0	0,020 0	0,014 0
	30	0,032 0	0,024 0	0,016 0
	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 ρ_{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):

$$A_{s,min} = \rho_{min} \cdot d \cdot b_w \tag{20}$$

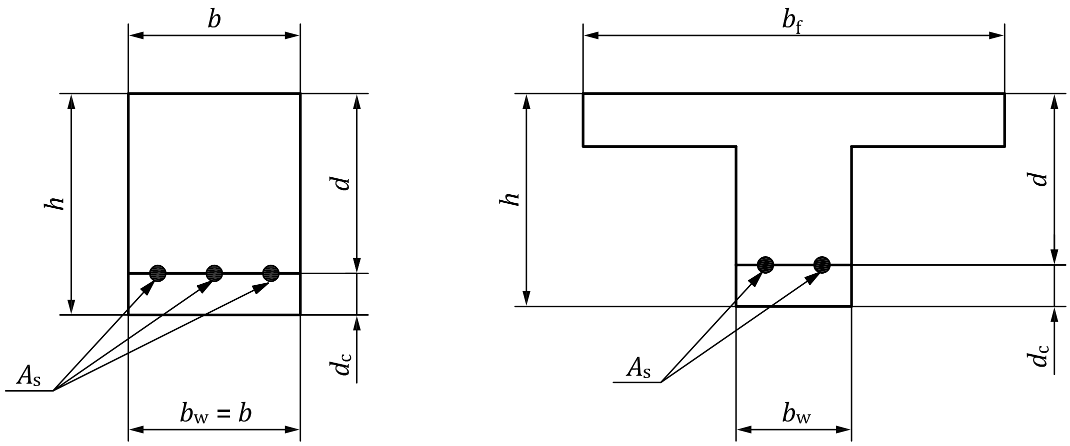

Figure 28 — Rectangular section and T-shaped section with flange in compression

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):

$$A_{s,min} = 2 \cdot \rho_{min} \cdot d \cdot b_w \tag{21}$$

$$A_{s,min} = \rho_{min} \cdot d \cdot b_f \tag{22}$$

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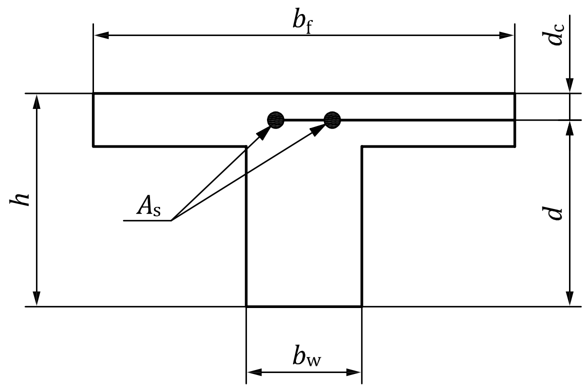

Figure 29 — T-shaped section with flange in tension

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

	f_y (MPa)		
	240	**300**	**400**
ρ_{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, ρ, should not exceed the following values expressed in function of ρ_{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):

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

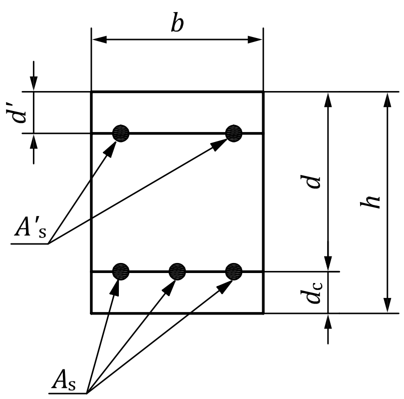

Figure 30 — Section with tension and compression reinforcement

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Table 6 — Maximum flexural reinforcement ratio, ρ_{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, ρ_v, of vertical reinforcement area to gross concrete horizontal section area should be 0,002 5.

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9.5.5.2 Maximum area of vertical reinforcement

The maximum ratio, ρ_v, of vertical reinforcement area to gross structural concrete wall horizontal section area should be 0,06, but when the ratio, ρ_v, exceeds 0,01, the vertical reinforcement should be enclosed with ties as prescribed for columns in 9.6.4.1.

$$0,0025 \le \rho_v = \left[\frac{A_{st}}{b_w \cdot l_w}\right] \le 0,06 \tag{28}$$

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 ($d_b \ge 10$ mm).

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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.

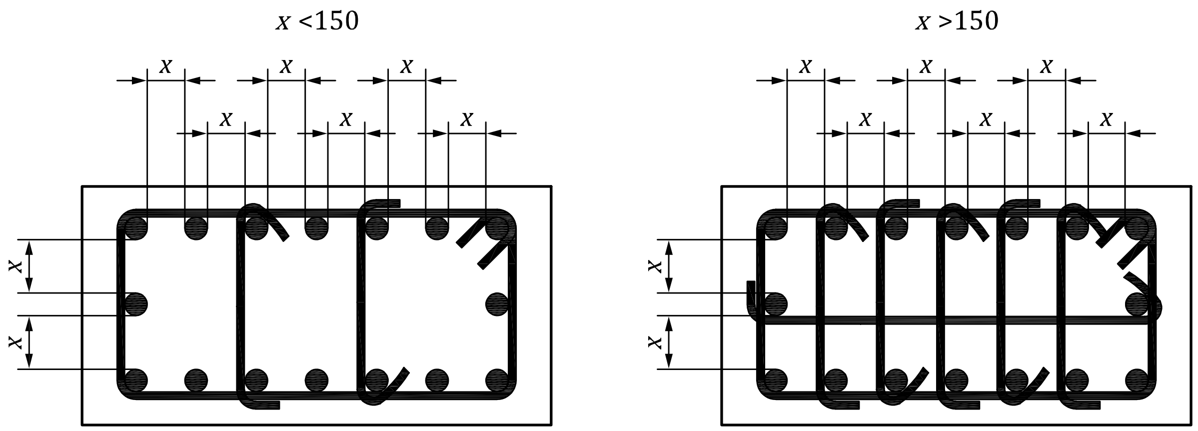

Figure 31 — Arrangement of ties in a tied column section

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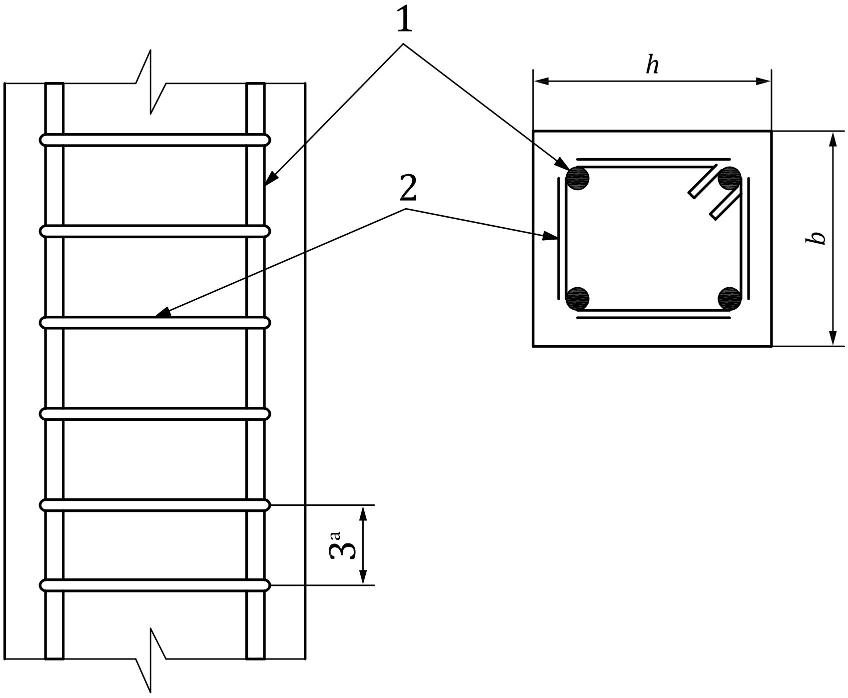

Key

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, ρ_s, defined as ratio of the volume of reinforcement contained in one loop of the spiral to the volume of concrete in the core of the column confined by the same loop of spiral, should be not less than any of the values given by Formula (29) (see Figure 33):

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$$\rho_s = \frac{A_b \cdot \pi \cdot d_c}{A_c \cdot s} \geq \left[\begin{array}{c} 0,12 \cdot \dfrac{f_c'}{f_{ys}} \\ 0,45 \cdot \left[\dfrac{A_g}{A_c} - 1 \right] \cdot \dfrac{f_c'}{f_{ys}} \end{array} \right. \qquad (29)$$

where

A_b is the area of the bar of spiral;

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

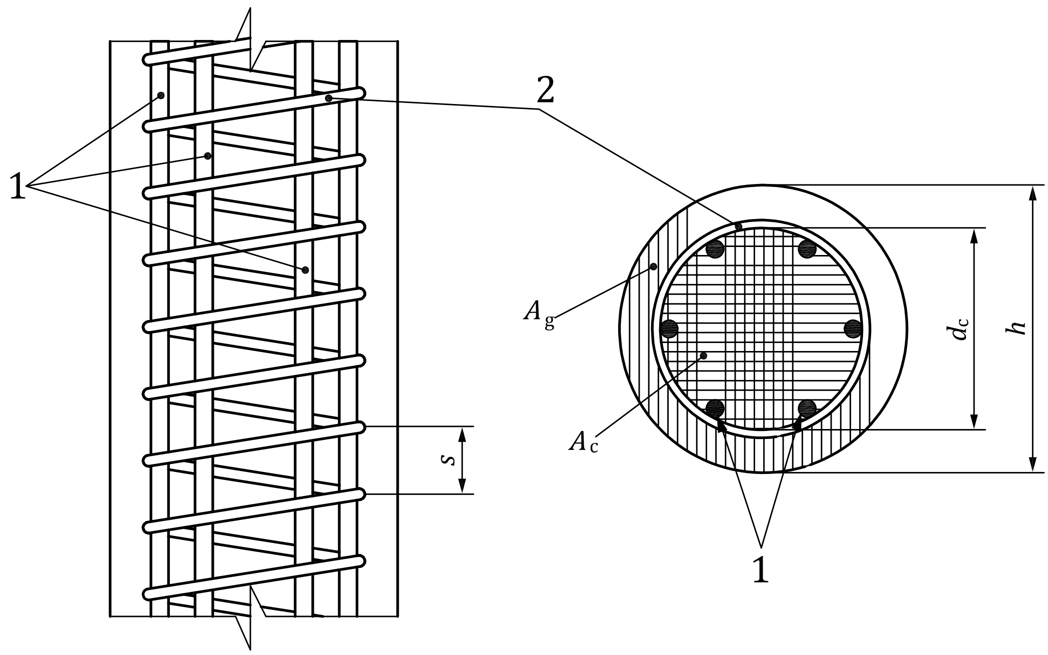
s is the vertical spacing of the spiral;

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

A_g is the gross column section area;

f_c' is the specified concrete strength of the column;

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



Key

1 longitudinal bars

2 spiral

Figure 33 — Spiral reinforcement of column

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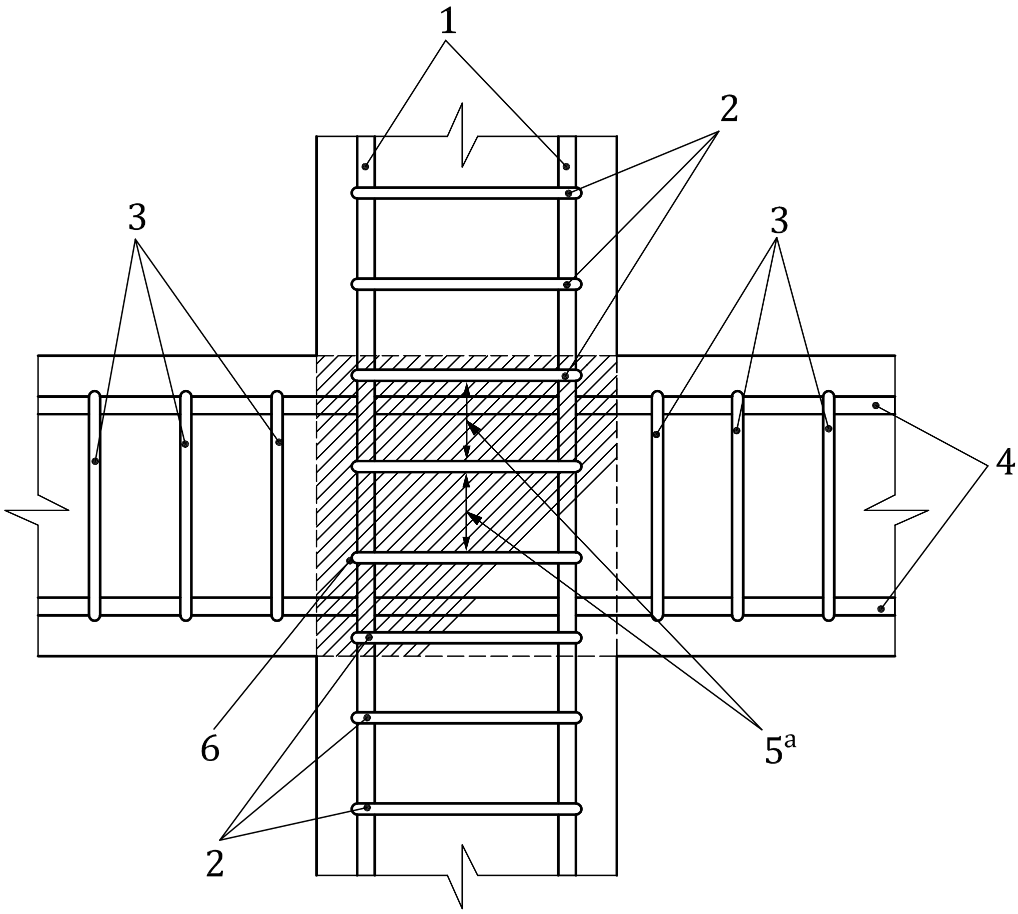
9.6.4.3 Column-girder joints

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



Key

1 column longitudinal reinforcement

2 column ties

3 girder stirrups

4 gider longitudinal reinforcement

5 joint ties

6 joint

a $s \leq 150$ mm.

Figure 34 — Column ties in column-girder joints

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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 $\left(0{,}10 \cdot f'_c \cdot A_g\right)$, 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).

$$\phi \cdot M_n \geq M_u \tag{30}$$

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):

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

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

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

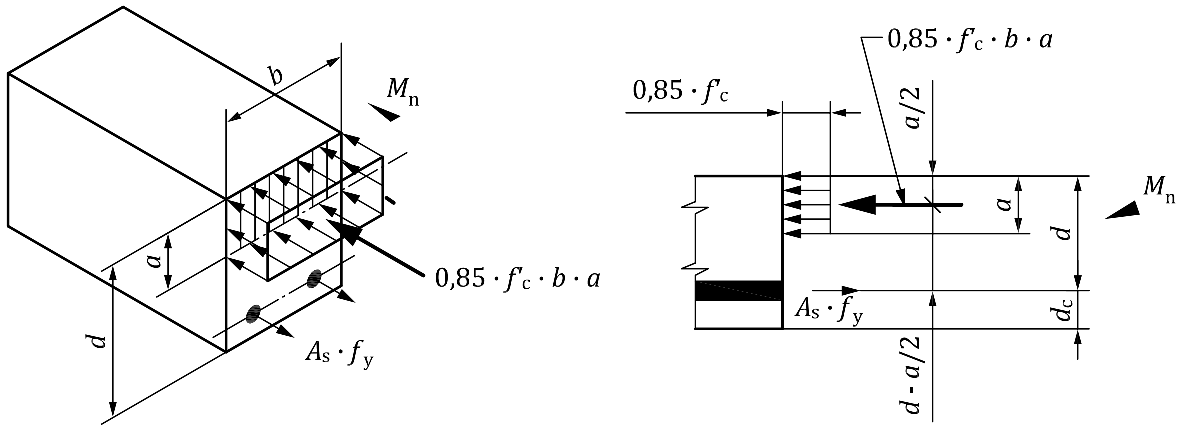

Figure 35 — Flexural nominal moment strength

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

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

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 ρ_{max} from Tables 4 and 6, respectively, as Formula (34):

$$\phi \cdot M_n \approx \phi \cdot A_s \cdot f_y \cdot 0{,}85 \cdot d \tag{34}$$

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):

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

where $\alpha = \frac{f'_c}{1{,}18 \cdot f_y}$, or using the approximate Formula (34), in slabs where $\rho < \rho_{max}$, with ρ_{max} from Table 4, and in girders, beams and joists where $\rho < \frac{\rho_{max}}{2}$, with ρ_{max} from Table 6, as given in Formula (36):

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

In Formula (35) and Formula (36), ϕ = [0,90] (see 6.3.3). If the value obtained from Formula (35) or Formula (36) is smaller than ρ_{min} from 9.5.2.2 and 9.5.3.1, ρ should be increased to that value. For slabs, if the obtained value of ρ is greater than ρ_{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 ρ is greater than ρ_{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, ρ, is less than ρ_{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.

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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):

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

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

$$a = \frac{\left(A_s - A_s' \right) \cdot f_y}{0,85 \cdot f_c' \cdot b} \tag{38}$$

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

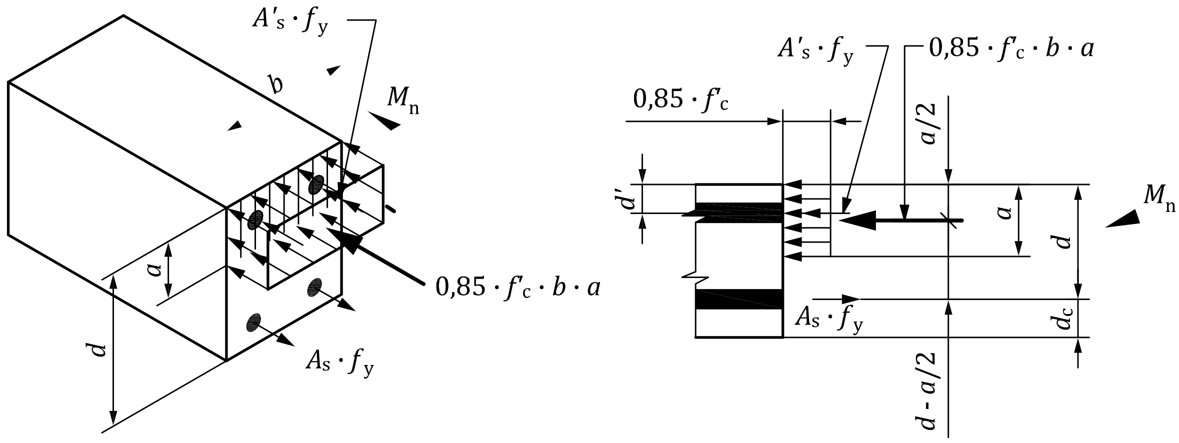

Figure 36 — Flexural nominal moment strength for doubly reinforced sections

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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:

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

and

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

In Formula (39) and Formula (40), ϕ = [0,90]; see 6.3.3 a). The steel ratio, ρ_{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.

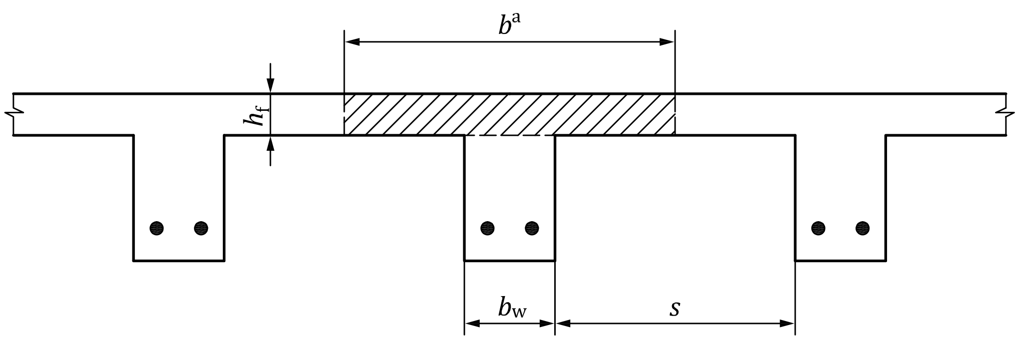
9.7.6.1 Effective flange width for beams with slab in both sides

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

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

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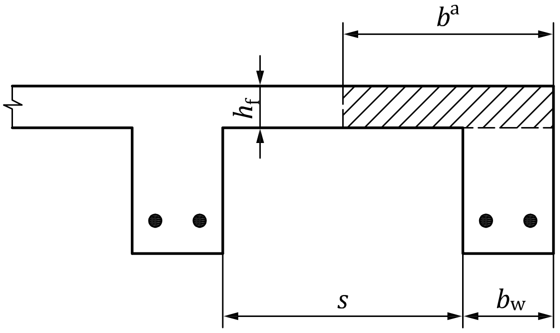
9.7.6.2 Effective flange width for beams with slab in one side only

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.

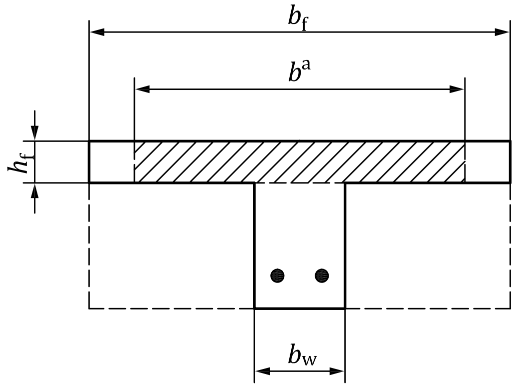


a $b \leq \text{min. of} \begin{cases} l/12 + b_w \\ 6 \cdot h_f + b_w \\ s/2 + b_w \end{cases}$

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

Figure 39 — Effective flange width for isolated T-beams

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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.

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

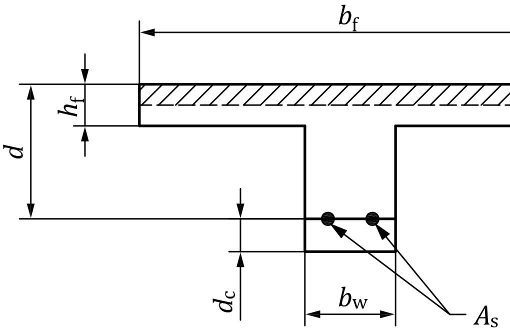

Figure 40 — Effective cross section for moment strength calculation of T-beams

9.7.6.5 Obtaining the flexural tension reinforcement area

The required ratio of flexural reinforcement, $\rho < \frac{A_s}{\left(b \cdot d\right)}$ for T-beams, should be obtained from Formula (35) or Formula (36), and the flexural reinforcement ratio, ρ, 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 ρ_{min} from 9.5.3.1, ρ should be increased to that value. If the obtained value of ρ is greater than ρ_{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.

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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):

$$\phi \cdot V_n \geq V_u \tag{43}$$

In Formula (43), ϕ = [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):

$$\phi \cdot V_n = \phi \cdot \left(V_c + V_s\right) \tag{44}$$

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), ϕ = [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 ϕ = [0,85]; see 6.3.3 d).

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

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):

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

where

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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), ϕ = [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.

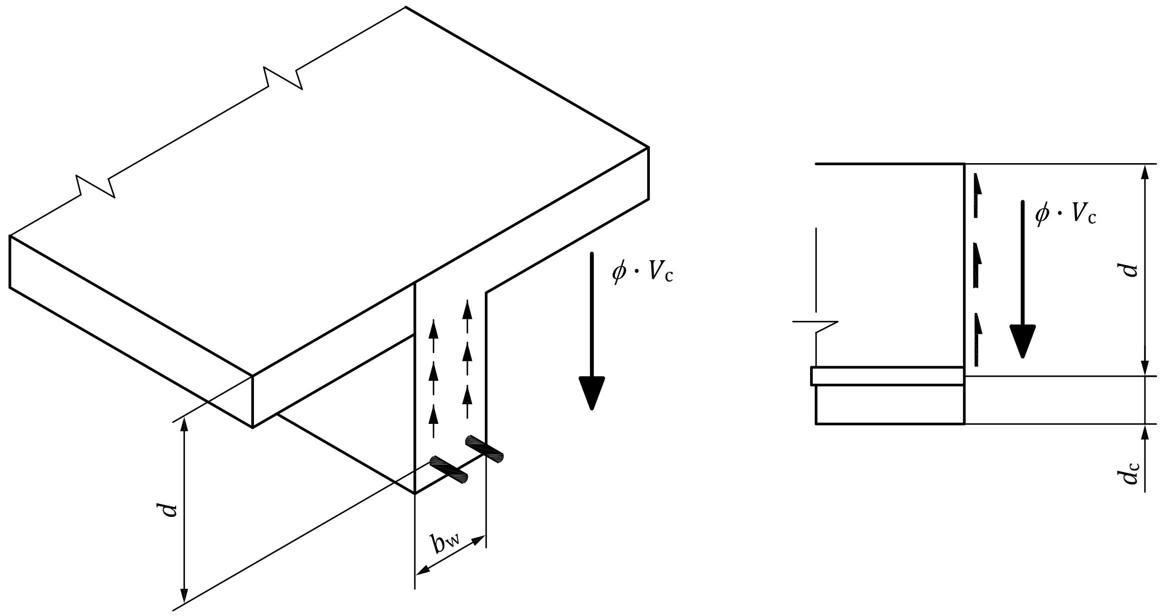

Figure 41 — Contribution of concrete to beam-action shear strength in girders, beams, and joists

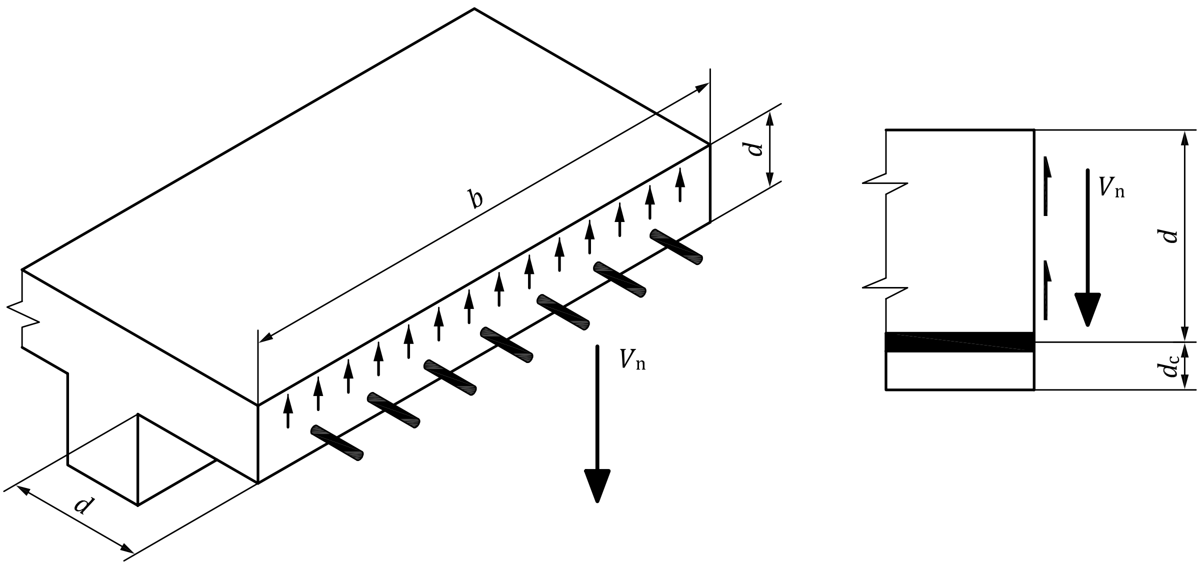

Figure 42 — Contribution of concrete to beam-action shear strength in solid slabs

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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.

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

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.

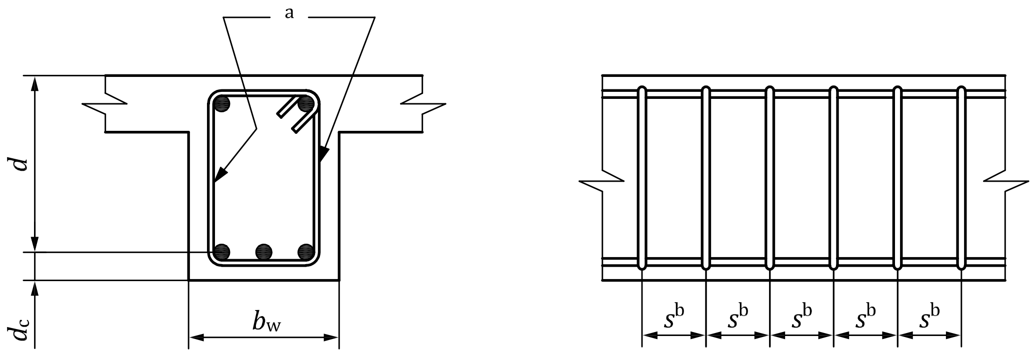
c) Where the factored shear, V_u exceeds $\phi \cdot V_c$ the difference $(V_u - \phi \cdot V_c)$ should be provided for by shear reinforcement, using Formula (44), Formula (45) and Formula (46) and the following limitations should be employed (see Table 8).

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 $(\phi \cdot V_c/2 \leq V_u < \phi \cdot V_c)$

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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	
$(\phi V_c) > V_u \geq \frac{(\phi V_c)}{2}$		$A_v = \frac{1}{16}\sqrt{f'_c}\,\frac{b_w \cdot s}{f_{ys}} \geq \frac{b_w \cdot s}{3 \cdot f_{ys}}$	$s \leq \text{min. of} \begin{cases} d/2 \\ 600\ \text{mm} \end{cases}$
$V_u \geq (\phi V_c)$	$2 \cdot \phi \cdot V_c > \phi \cdot V_s$	$A_v = \frac{(V_u - \phi \cdot V_c) \cdot s}{\phi \cdot f_{ys} \cdot d}$	$s \leq \text{min. of} \begin{cases} d/2 \\ 600\ \text{mm} \\ 3 \cdot A_v \cdot f_{ys}/b_w \end{cases}$
	$4 \cdot \phi \cdot V_c > \phi \cdot V_s \geq 2 \cdot \phi \cdot V_c$	$A_v = \frac{(V_u - \phi \cdot V_c) \cdot s}{\phi \cdot f_{ys} \cdot d}$	$s \leq \text{min. of} \begin{cases} d/4 \\ 300\ mm \\ 3 \cdot A_v \cdot f_{ys}/b_w \end{cases}$
	$\phi \cdot V_s \geq 4 \cdot \phi \cdot V_c$	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.

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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 ϕ = [0,85]; see 6.3.3 d):

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

where β_c is the ratio of long side to short side of the column, concentrated load or reaction area, as given in Formula (50):

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

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

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

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.

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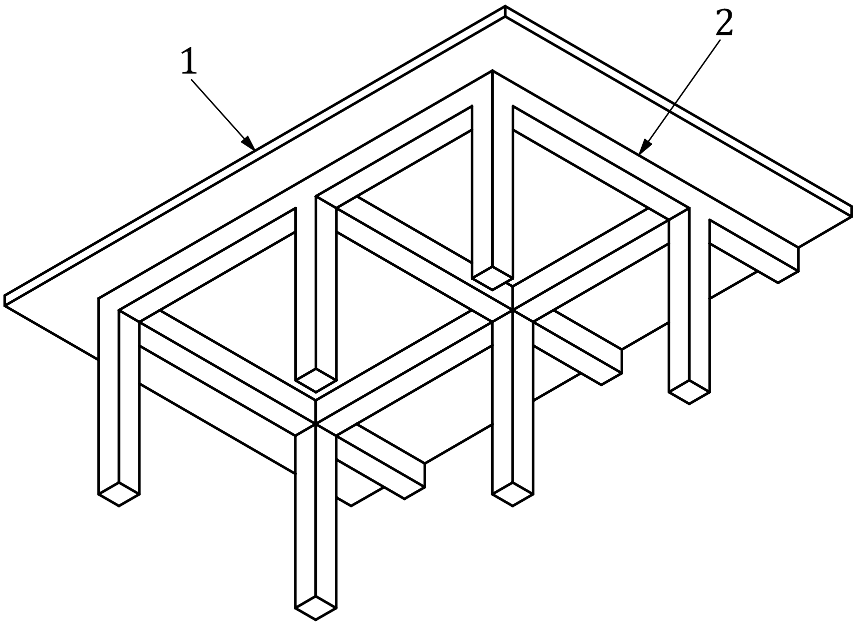

Key

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.

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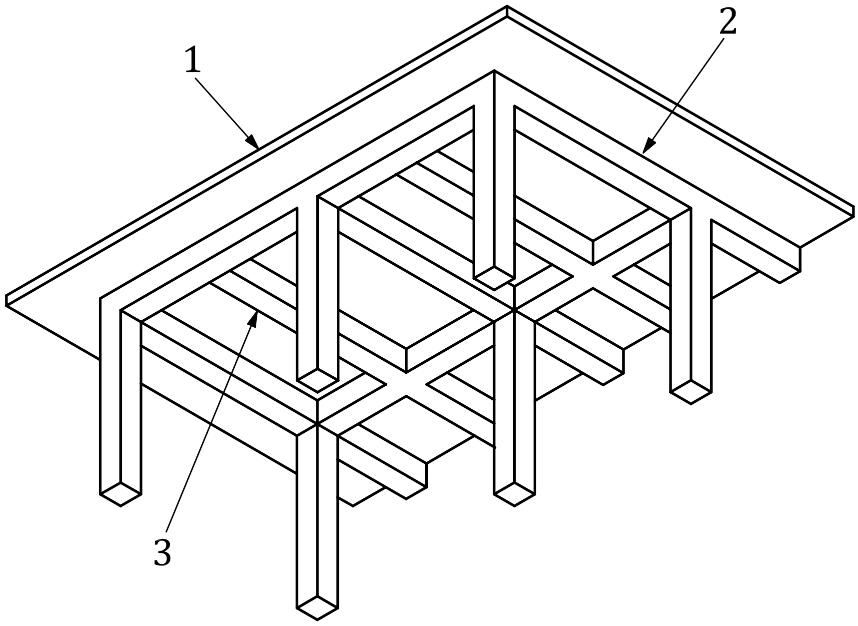


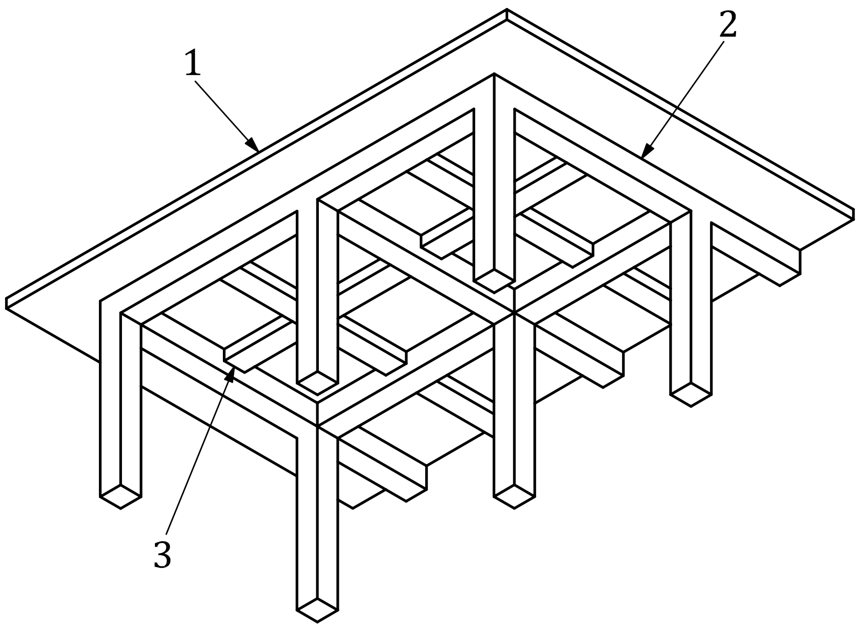
Key

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

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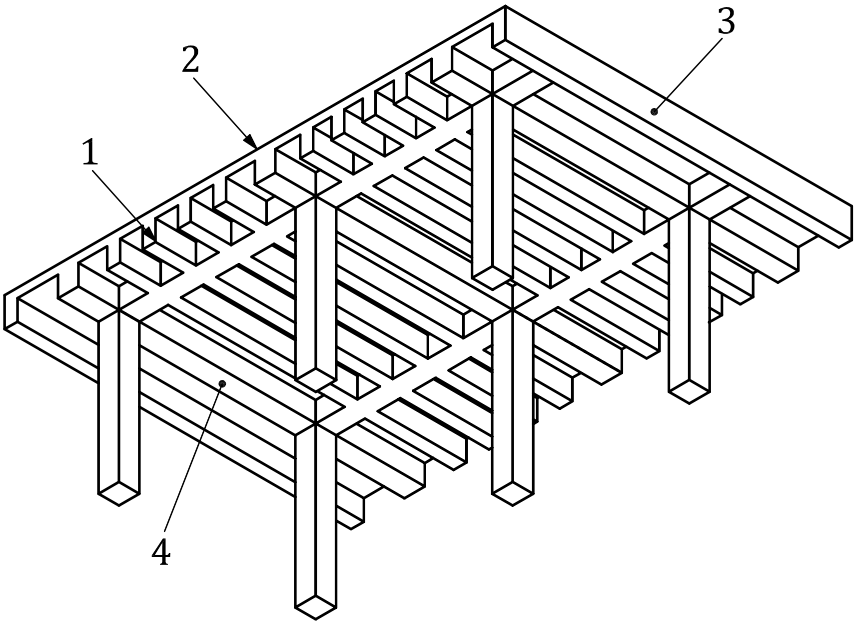
10.1.4 Joist systems

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

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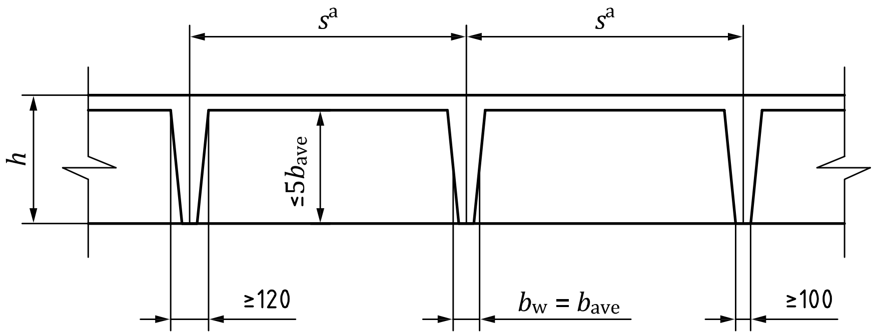

a $s \leq 2,5\ h$
$s \leq 1,2\ m$

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.

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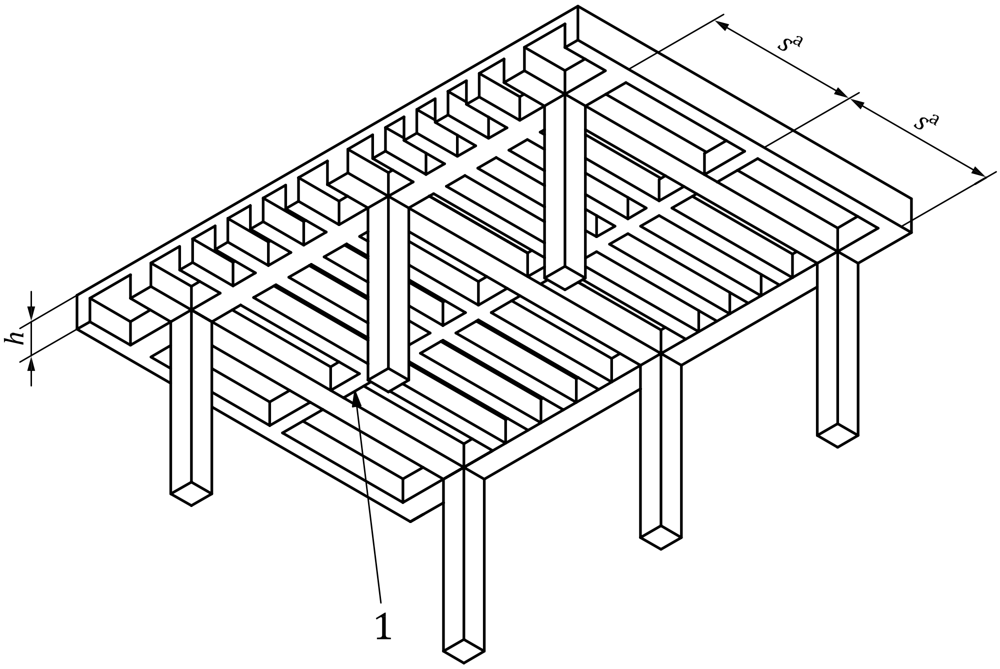

Key

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.

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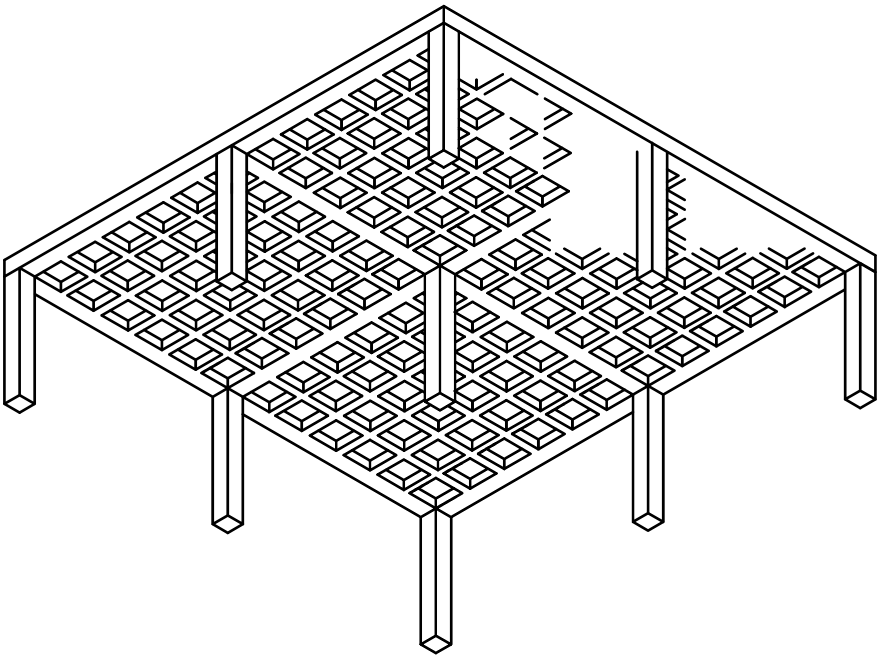

Figure 50 — Two-way joist system or waffle-on-beams system

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;

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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;

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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.

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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.

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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 nor less than 120 mm for spans, l_n, greater than 3 m, and should not be less than 100 mm for spans, l_n, less or equal to 3 m.

$$h = \frac{l_n}{30 + 3 \cdot \beta} \tag{52}$$

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

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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.

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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, ρ_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.

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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.

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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.

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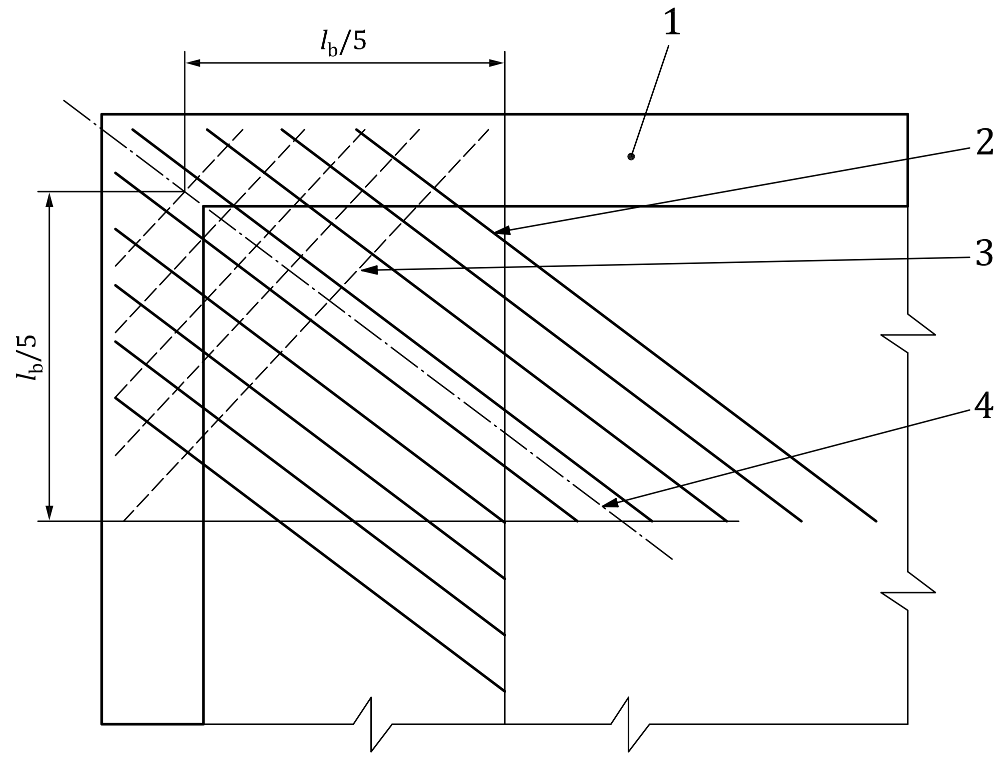

Key

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.

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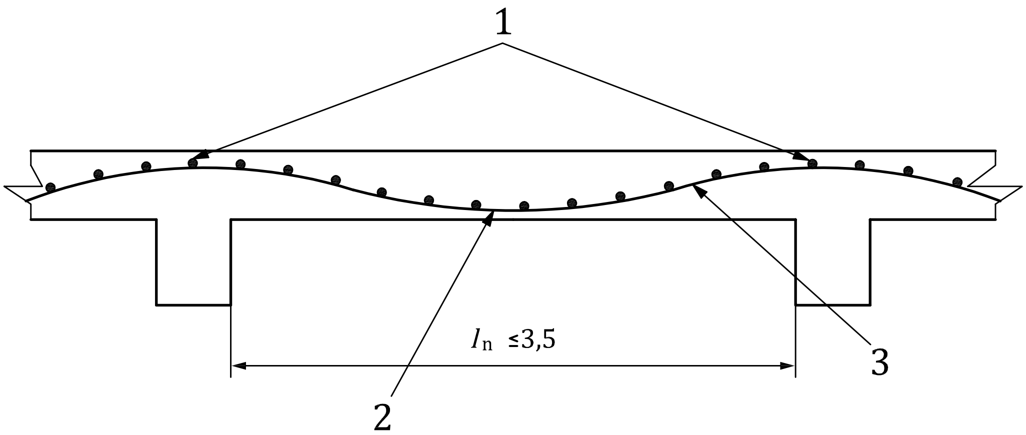

Key

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 · 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/m². See Figure 53.

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

11.4.3 Reinforcement

The flexural reinforcement ratio, ρ, 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 · mm (1 N · m/m = 10^3 · N · mm/m), using d in mm as one-half the thickness of the thin slab, and b = 1 000 mm. ρ should be made greater 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.

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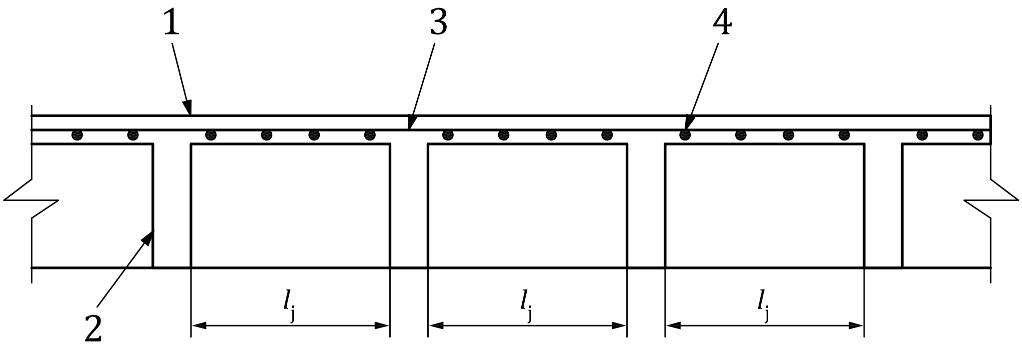

Key

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^2. 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-

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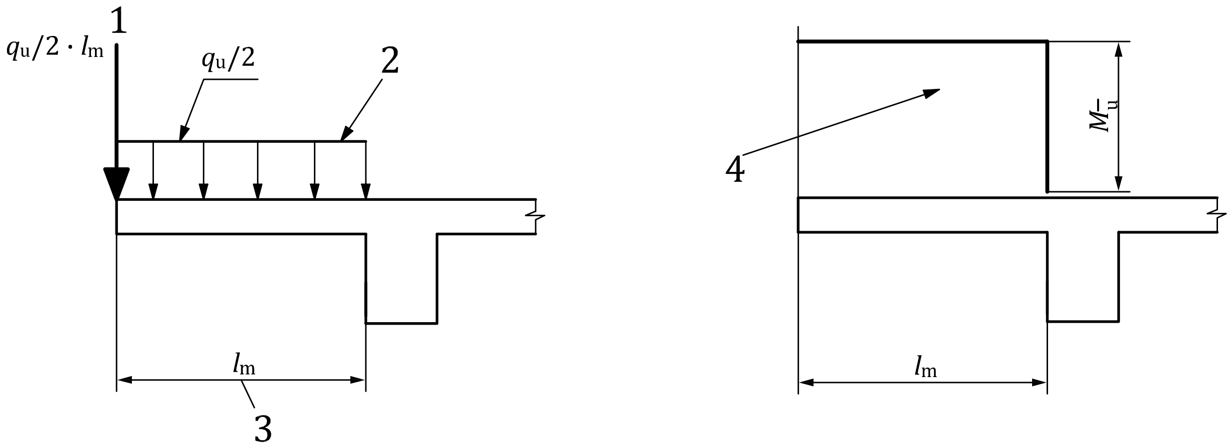
half acts as uniformly distributed load over the full span, using Formula (56), but it should not be less than the factored negative bending moment of the first interior span at the exterior supporting girder, beam or structural concrete wall, nor less than one-third of the positive bending moment, in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

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

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



Key

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

Figure 54 — Calculation of negative moment at slab cantilevers

11.5.3 Reinforcement

11.5.3.1 Negative flexural reinforcement

The negative flexural reinforcement ratio, ρ, 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 = 10^3 · 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.

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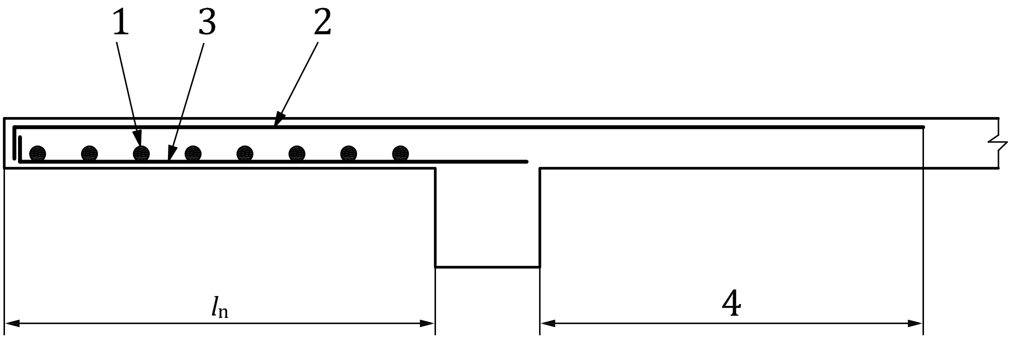

Key

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.

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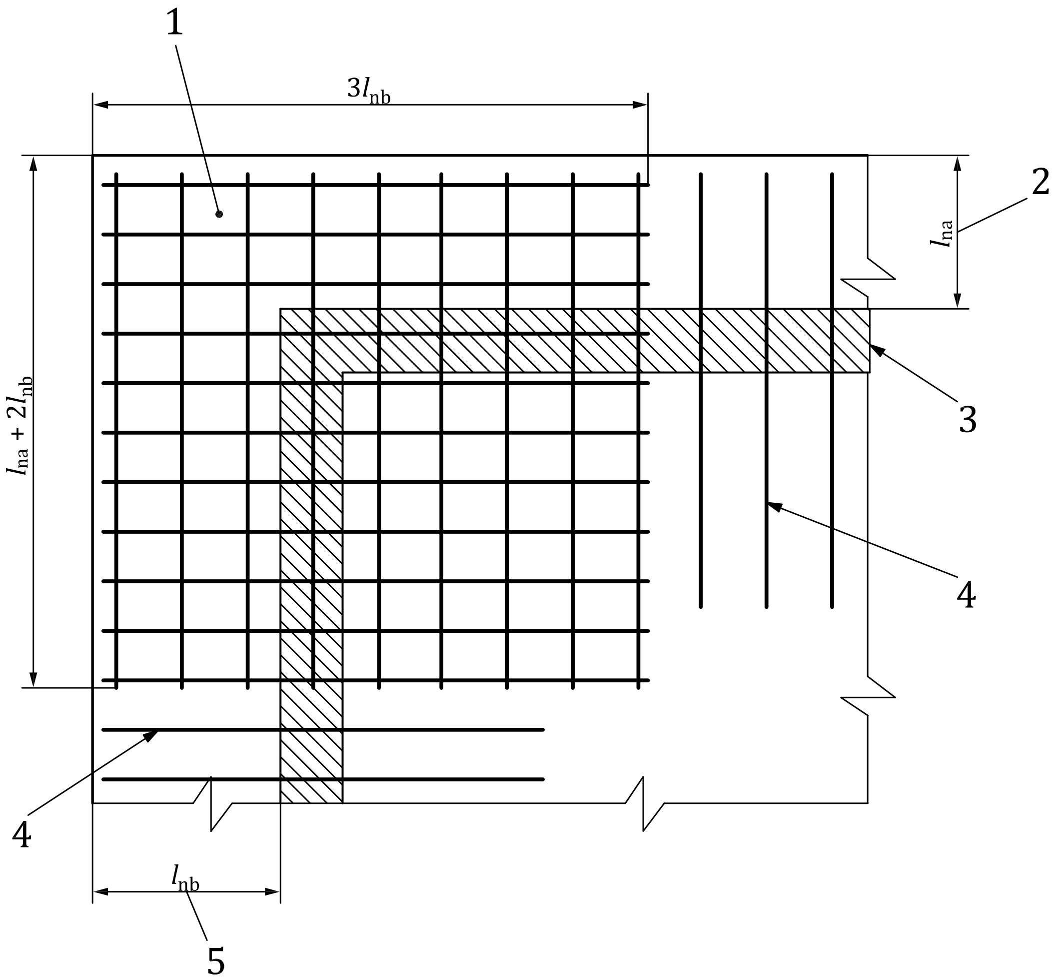

Key

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/m².

$$V_u = q_u \cdot l_m \tag{57}$$

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 $\phi \cdot V_n$, 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.

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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

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

11.6.3 Longitudinal flexural reinforcement

11.6.3.1 Positive flexural reinforcement

The positive reinforcement ratio, ρ, 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 = 10^3 · 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

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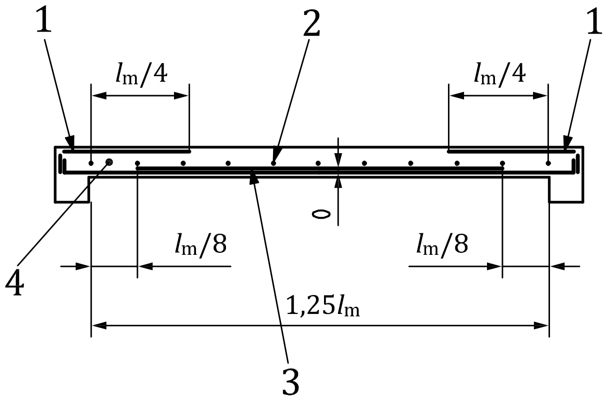
reinforcement at a distance equal to $l_m/8$ measured from the internal face of the supports into the span. See Figure 57.



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, ρ, 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 · mm (1 N · m/m = 10^3 · N · 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.

$$V_u = \frac{q_u \cdot l_m}{2} \tag{61}$$

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.

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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:

$$r_u = \frac{V_u \cdot l}{l_m} \tag{62}$$

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 cantelìver, 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:

$$M_u^+ = \frac{q_u \cdot l_n^2}{11} \tag{63}$$

— at interior spans:

$$M_u^+ = \frac{q_u \cdot l_n^2}{16} \tag{64}$$

Negative moment at supports:

— at interior face of external support:

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$$M_u^- = \frac{q_u \cdot l_n^2}{24} \tag{65}$$

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

$$M_u^- = \frac{q_u \cdot l_n^2}{9} \tag{66}$$

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

$$M_u^- = \frac{q_u \cdot l_n^2}{10} \tag{67}$$

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

$$M_u^- = \frac{q_u \cdot l_n^2}{12} \tag{68}$$

11.7.3 Longitudinal flexural reinforcement

11.7.3.1 Positive flexural reinforcement

The positive reinforcement ratio, ρ, 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 · mm/m (1 N · m/m = 10^3 · N · mm/m), using d in mm, and b = 1 000 mm. This reinforcement should comply with 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, ρ, 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 · mm/m (1 N · m/m = 10^3 ·N · mm/m), using d in mm, and b = 1 000 mm. This reinforcement should comply with 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.

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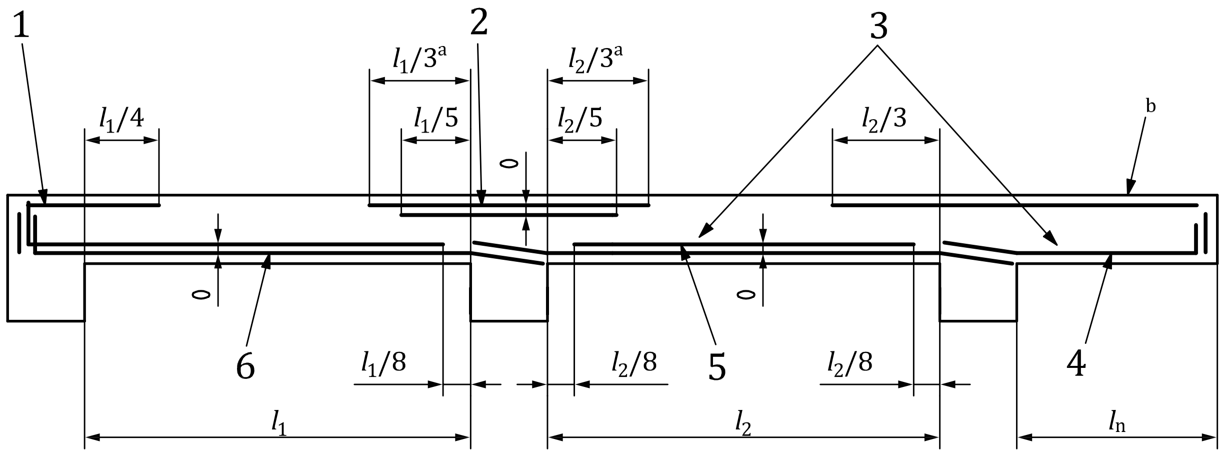


Key

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

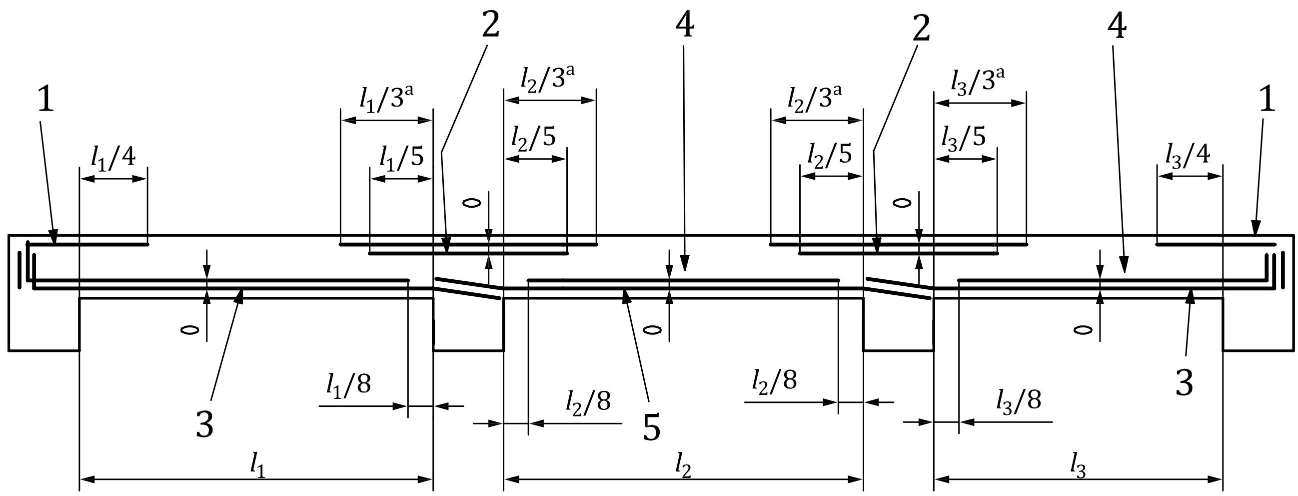
5 positive reinforcement interior span

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

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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^2. See Figure 58 and Figure 59.

At exterior face of first interior support:

$$V_u = 1{,}15 \cdot \frac{q_u \cdot l_m}{2} \tag{69}$$

At faces of all other supports

$$V_u = \frac{q_u \cdot l_m}{2} \tag{70}$$

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):

$$r_u = \frac{V_u \cdot l}{l_m} \tag{71}$$

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 canteliver, 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);

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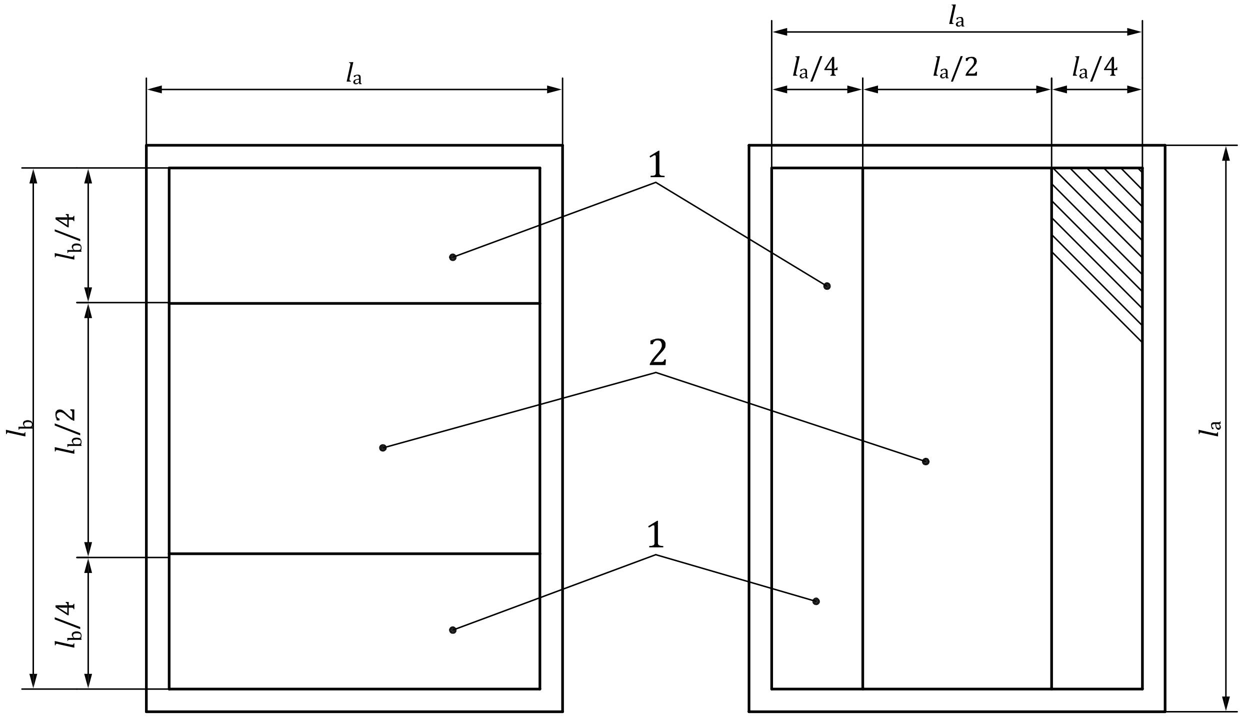
c) the supporting girders or beams should be cast monolithically with the slab and should have a total depth not less than three times the slab thickness;

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, β, of long clear span, l_b, to short clear span, l_a, and the corresponding edge continuity conditions.

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

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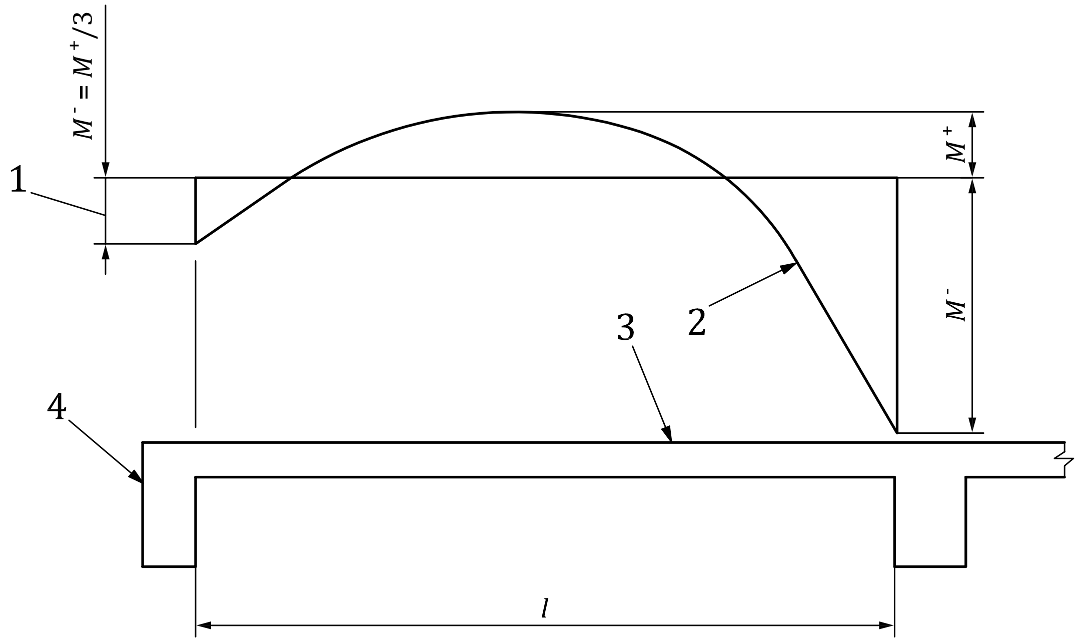

Key

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.

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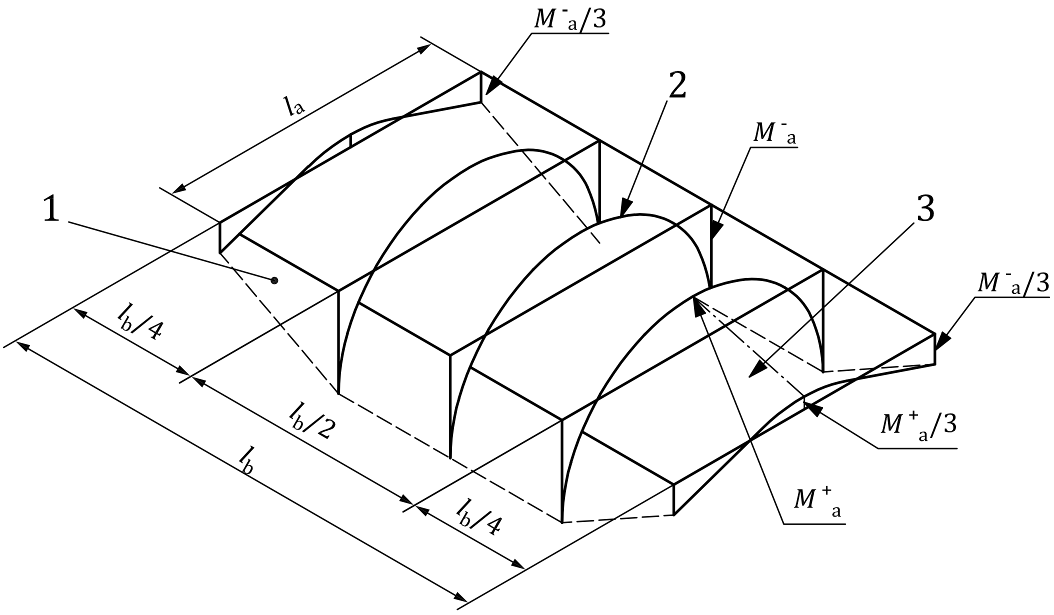


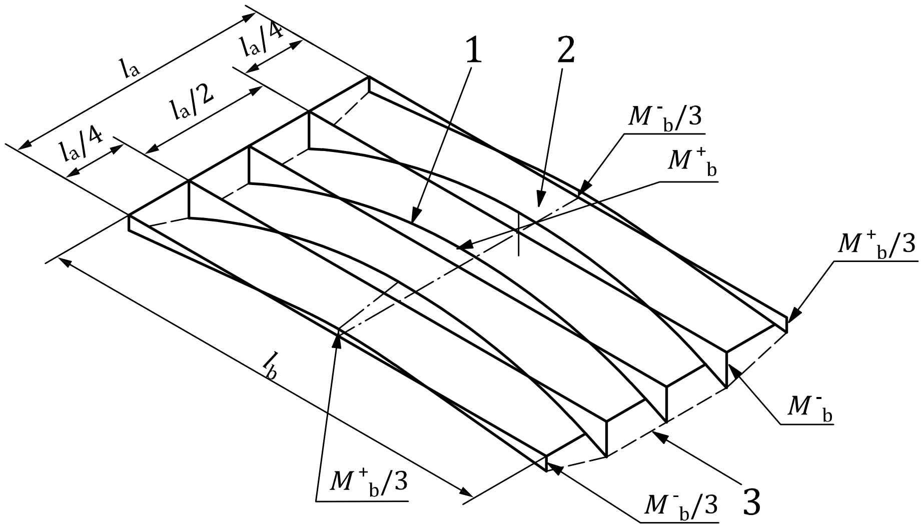
Key

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

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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		$\alpha_b = 0{,}00$

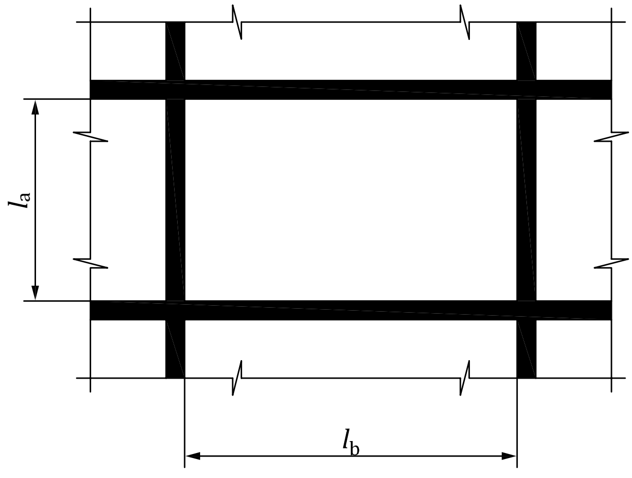

Figure 64 — Central panel of two-way slabs supported on girders, beams or structural concrete walls

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Table 15 — Edge panel with la 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		$\alpha_b = 0{,}00$

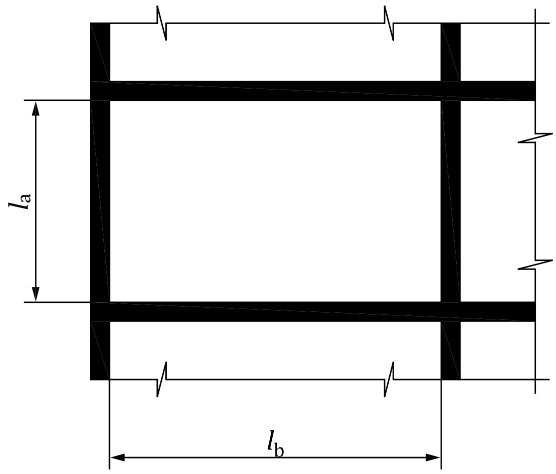

Figure 65 — Edge panel with l_a parallel to edge, of two-way slabs supported on girders, beams or walls

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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)		
Panel span ratio	**Negative moment**	**Positive moment**	**Load fraction**	**Negative moment**	**Positive moment**	**Load fraction**
1,0	$M_a^- = \frac{q_u l_b^2}{30}$	$M_a^+ = \frac{q_u l_b^2}{39}$	$\alpha_a = 0,33$	$M_b^- = \frac{q_u l_b^2}{16}$	$M_b^+ = \frac{q_u l_b^2}{35}$	$\alpha_b = 0,67$
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		$\alpha_b = 0,00$

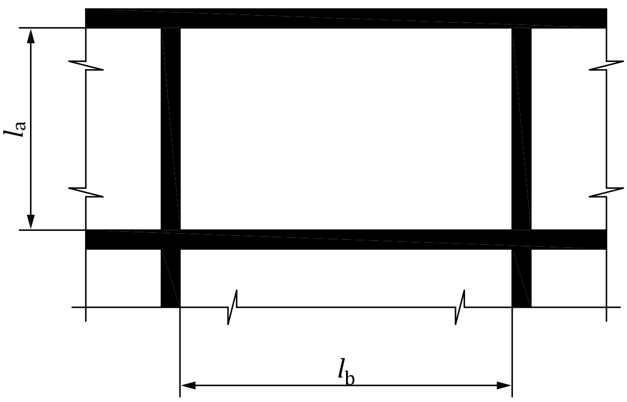

Figure 66 — Edge panel with l_b parallel to edge, of two-way slabs supported on girders, beams or walls

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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		$\alpha_b = 0{,}00$

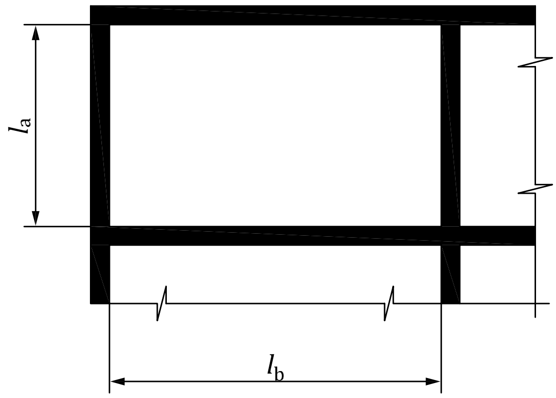

Figure 67 — Corner panel of two-way slabs supported on girders, beams or structural concrete walls

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, ρ, 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

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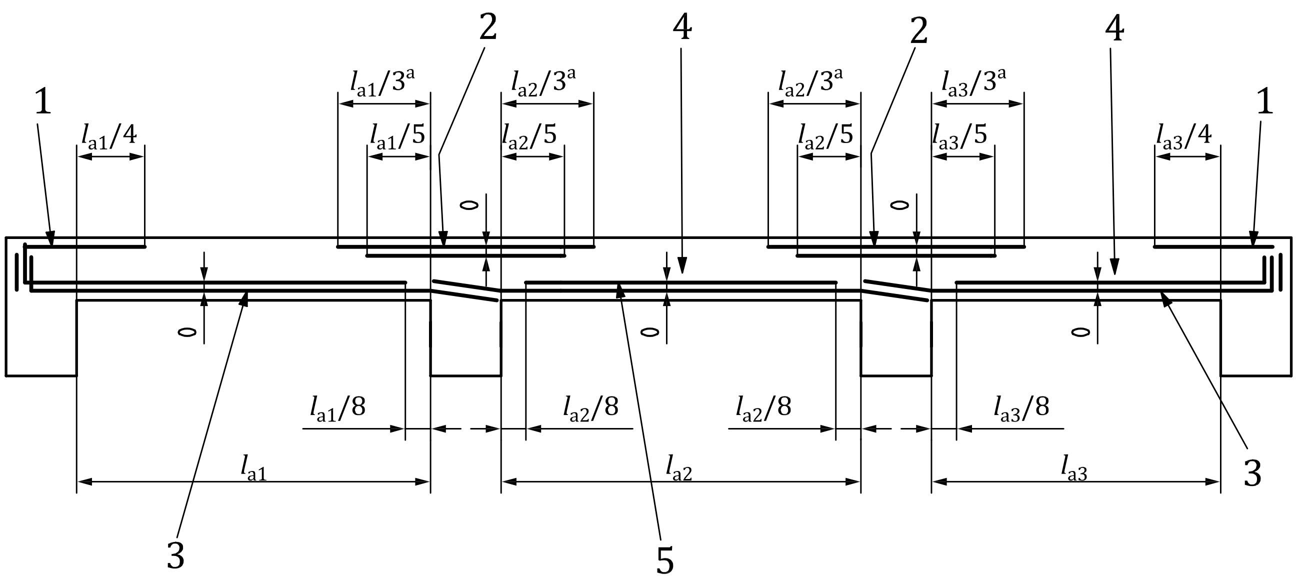
(1 N · m/m = 10³ · N · mm/m), using d in mm and b = 1 000 mm. All guides for positive flexural reinforcement of 11.3.3 should be met. At a distance equal to $l_a/8$ or $l_b/8$ measured from the face of any interior support, it should be permitted to suspend up to one-half of the positive flexural reinforcement 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, ρ, 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 N · m/m = 10³ · N · mm/m), using d in mm and b = 1 000 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

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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, α_a and α_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, β. See Figure 69. The factored shear should not be less than the factored shear caused by factored design load, q_u (in N/m^2) 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.

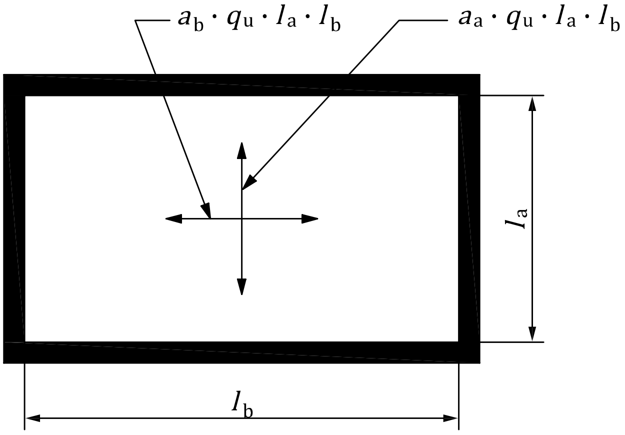

Figure 69 — Fraction of the total load in the panel travelling in each direction in two-way slabs supported on girders, beams or structural concrete walls

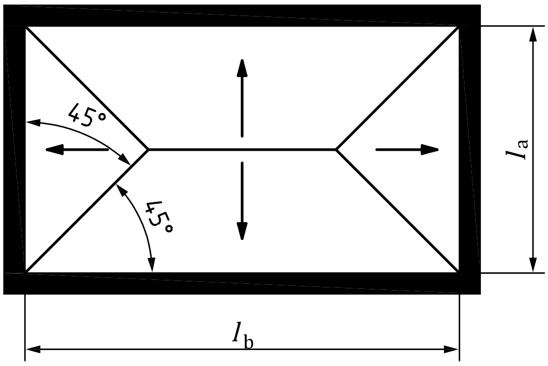

Figure 70 — Tributary areas for minimum shear at the supports of two-way slabs supported on girders, beams or structural concrete walls

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.

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

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

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

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

$$d \geq \frac{3 \cdot q_u \cdot \alpha_a \cdot l_a}{\phi \cdot \sqrt{f_c'}} \tag{74}$$

$$d \geq \frac{3 \cdot q_u \cdot \alpha_b \cdot l_b}{\phi \cdot \sqrt{f_c'}} \tag{75}$$

$$d \geq \frac{3 \cdot q_u \cdot l_a}{2 \cdot \phi \cdot \sqrt{f_c'}} \tag{76}$$

In Formula (74) to Formula (76), q_u should be used in N/m^2, l_a and l_b in m, f_c' in MPa, and ϕ = [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).

$$r_u = \frac{V_u \cdot l}{l_a} \tag{77}$$

$$r_u = \frac{V_u \cdot l}{l_b} \tag{78}$$

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.

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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.

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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.

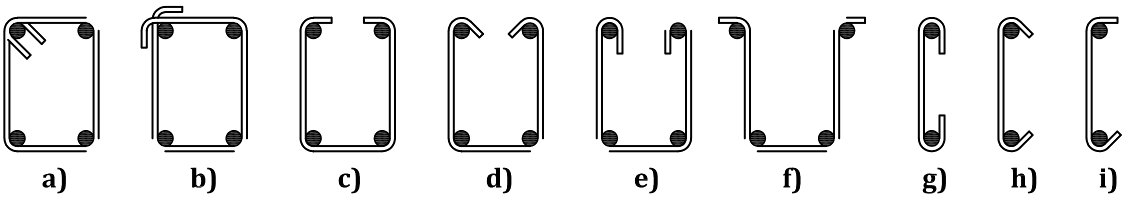

Figure 71 — Typical stirrup shapes

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.

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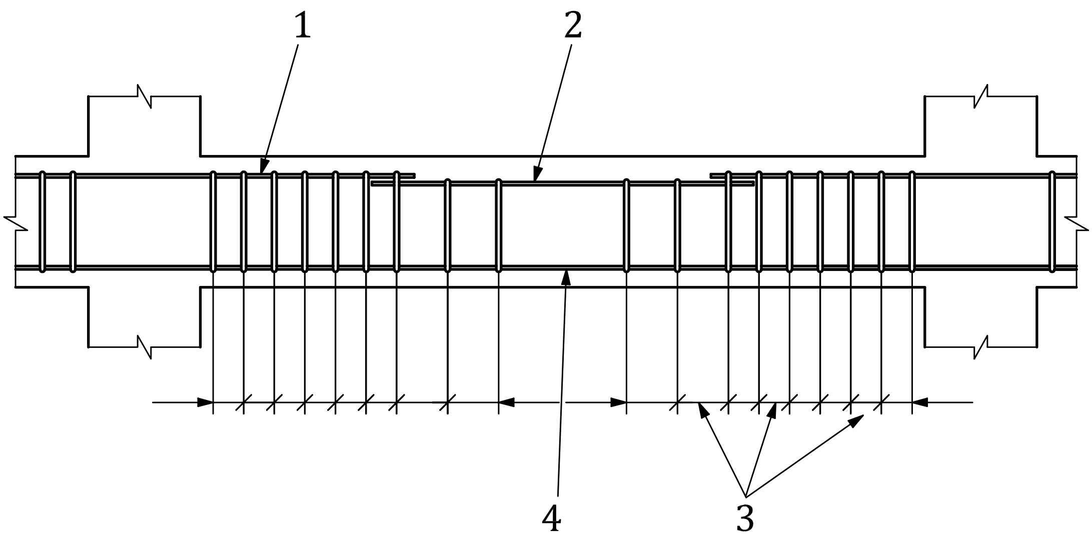
12.3.2.7 Support of stirrups

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.

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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.

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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.

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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.

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

12.3.4.13 Girders and beams with bw < 300 mm

Three longitudinal bars should be permitted for girders and beams whose width, b_w, is less than 300 mm and greater 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 mm $\leq b_w <$ 250 mm	2 bars
250 mm $\leq b_w <$ 300 mm	3 bars
300 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.

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

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12.3.4.17 Interior exposure

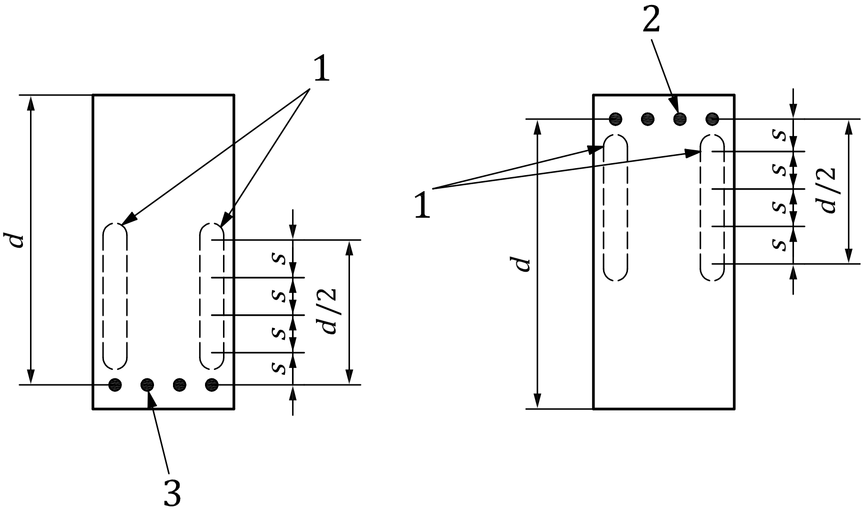
The minimum number of longitudinal bars in a layer that should be employed for girders, and beams, that are not exposed to earth or weather, should be greater than or equal to the value given by Formula (81), where b_w is the girder or beam width in mm.

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

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

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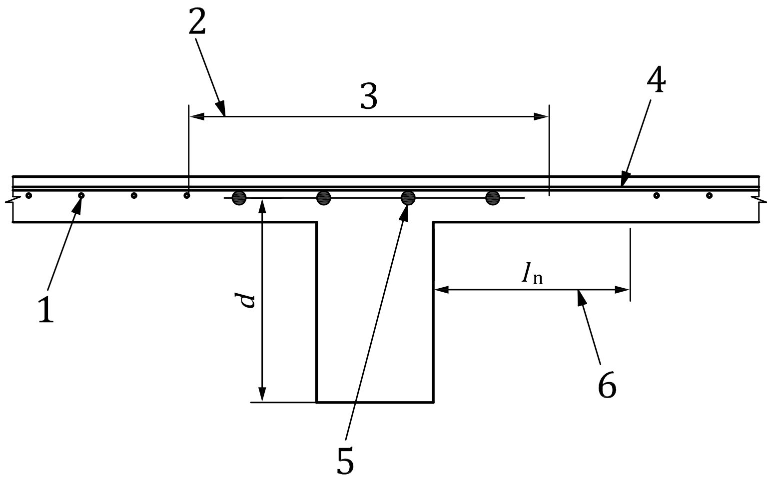
beam greater than or equal to the shrinkage and temperature reinforcement for slabs of 9.5.2.1. See Figure 74. For this case, the guides of 12.3.4.11 should not apply.



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 minimun 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.

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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, ΣP_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.

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

where l_m should be the clear span of the cantilever in m, W_u should be employed in N/m, ΣP_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 $\sum P_u$ should be in N.

Positive moment:

$$M_u^+ = \frac{W_u \cdot l_m^2}{8} + \frac{l_m}{4} \cdot \sum P_u \tag{84}$$

Negative moment at supports:

$$M_u^- = \frac{W_u \cdot l_m^2}{24} + \frac{l_m}{16} \cdot \sum P_u \tag{85}$$

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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 · 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 ΣP_u should be in N.

Positive moment:

— at end spans:

$$M_u^+ = \frac{W_u \cdot l_m^2}{11} + \frac{l_m}{9} \cdot \sum P_u \tag{86}$$

— at interior spans:

$$M_u^+ = \frac{W_u \cdot l_m^2}{16} + \frac{l_m}{5} \cdot \sum P_u \tag{87}$$

Negative moment at supports:

— at interior face of external support:

$$M_u^- = \frac{W_u \cdot l_m^2}{24} + \frac{l_m}{16} \cdot \sum P_u \tag{88}$$

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

$$M_u^- = \frac{W_u \cdot l_m^2}{9} + \frac{l_m}{6} \cdot \sum P_u \tag{89}$$

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

$$M_u^- = \frac{W_u \cdot l_m^2}{10} + \frac{l_m}{7} \cdot \sum P_u \tag{90}$$

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

$$M_u^- = \frac{W_u \cdot l_m^2}{12} + \frac{l_m}{8} \cdot \sum P_u \tag{91}$$

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.

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

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

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

d) The modulus of elasticity of concrete should be permitted to be taken as $E_c = 4\,500\sqrt{f_c'}$, in MPa.

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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 ΣP_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 ΣP_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 ΣP_u should be in kN.

At exterior face of first interior support:

$$V_u = 1{,}15 \cdot \frac{W_u \cdot l_m}{2} + 0{,}80 \cdot \sum P_u \tag{94}$$

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.

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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 · mm (1 N · m = 10^3 · N · mm), using d and b in mm. When a slab is present in the upper part of the section or when the beam or joist is T-shaped, it should be permitted to employ the T-beam effect as indicated in 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 · mm (1 N · m = 10^3 · N · 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.

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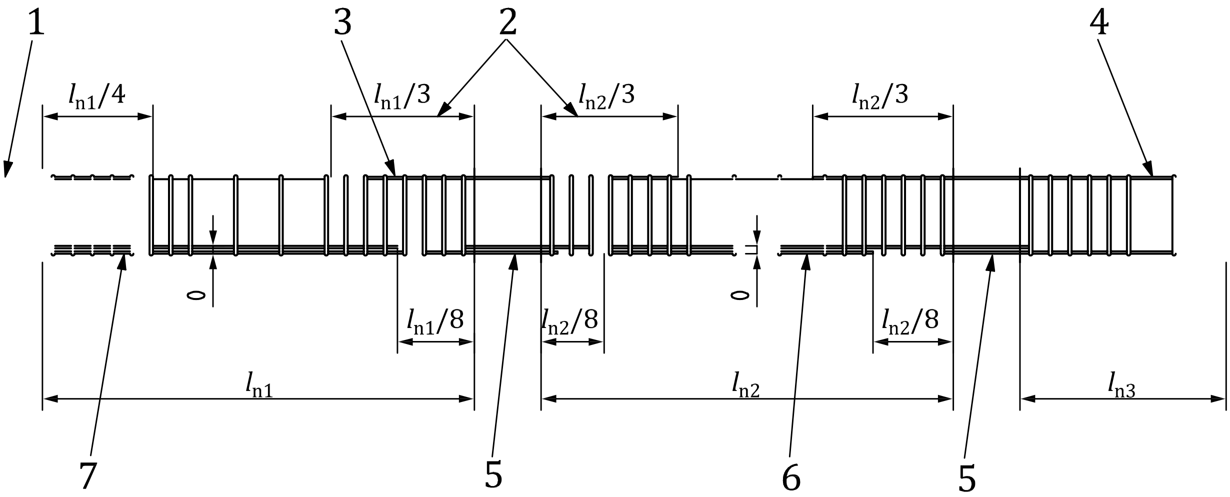

Key

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 were a concentrated load is applied, the value of P_u in N, should be subtracted from the value of shear shown in the diagram at the left of the load. Proceeding as described, at the face of the left support, the negative value of V_u in N there should be reached in the diagram. See Figure 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.

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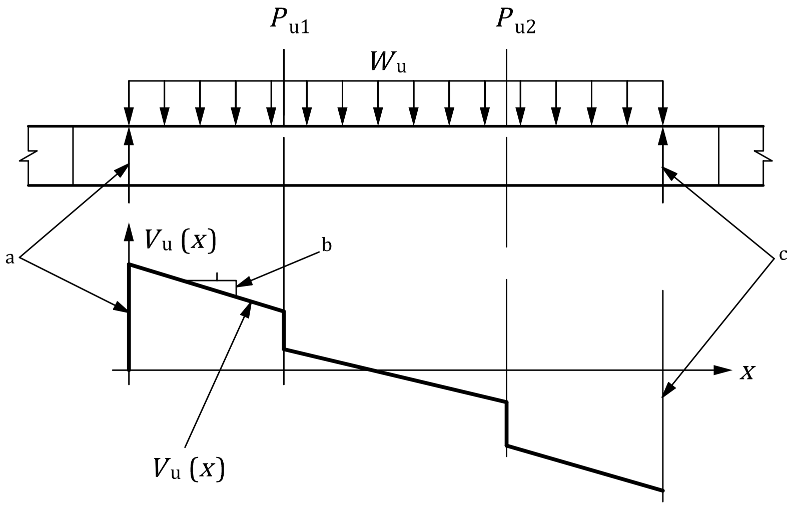

a $(V_u)_{\text{left supp.}}$

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 ϕ = [0,85]. See 6.3.3 d).

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

c) The hanger reinforcement should be full depth closed stirrups with a total area A_i as determined from Formula (96).

$$A_i \geq \frac{\left(1 - \frac{h_b}{h_s}\right) V_u}{\phi \cdot f_y} \qquad (96)$$

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 ϕ = [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

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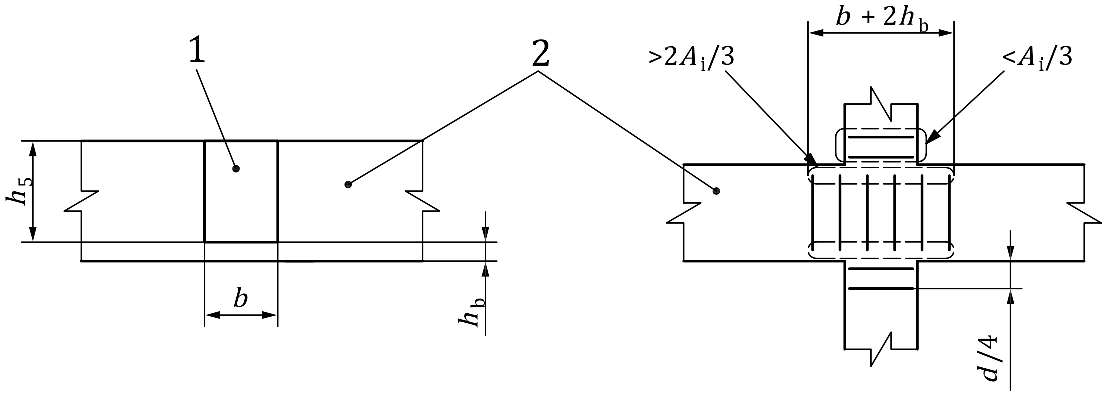
area of legs of the additional stirrups that are located at the side of the supported beam should be taken into account.

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).

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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:

$$R_u = \frac{V_u \cdot l}{l_m} \tag{98}$$

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.

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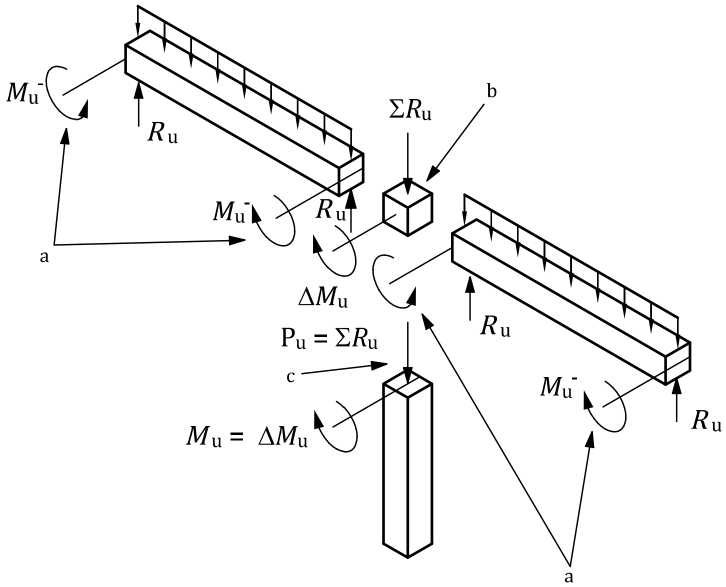

a Reactions at the ends of the element.

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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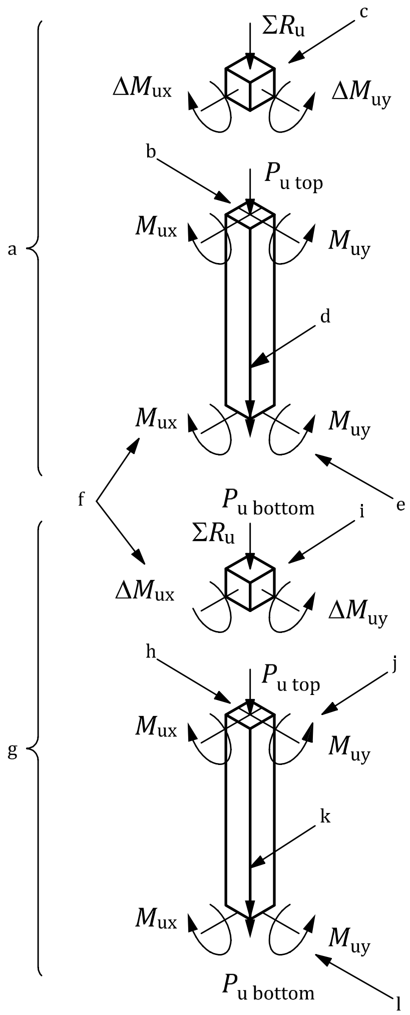

a Story *n*.

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 $P_{u\ bottom}$ equal to $P_{u\ top}$ 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 $P_{u\ top}$ of story $n-1$ equal to $P_{u\ bottom}$ from story n plus $\sum R_u$ from story $n-1$.

j Selfweight of the column.

l $P_{u\ bottom}$ equal to $P_{u\ top}$ 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 24×10^3 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

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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.

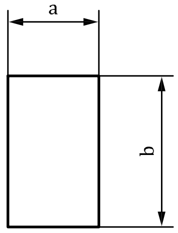
13.3.2 Limiting section dimensions

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 $b_c \geq 300$ 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.

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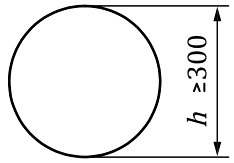
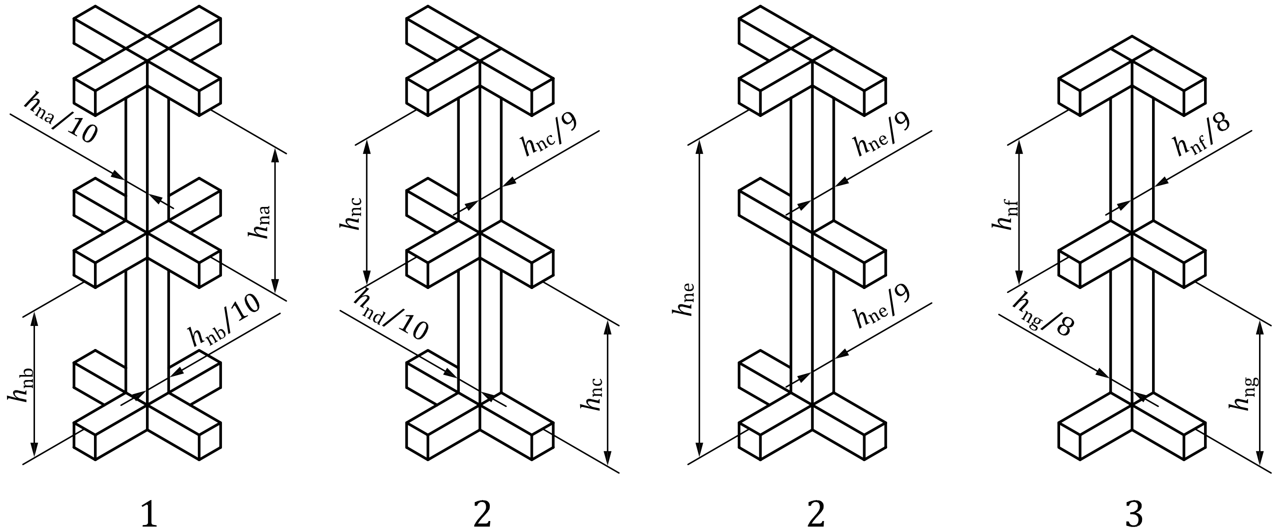


Figure 81 — Minimum cross-section dimension for circular columns

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.

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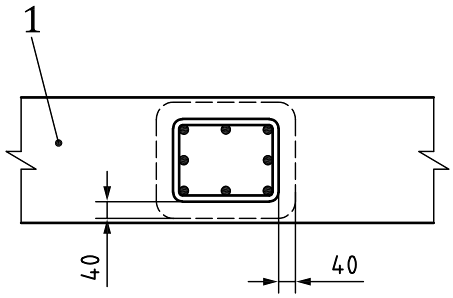
13.3.4 Column built monolithically with wall

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 ($0{,}01 \le \rho_t \le 0{,}06$).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).

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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.

a) Slope of inclined portion of an offset bar with axis of column should not exceed 1 in 6.

b) Portions of bar above and below an offset should be parallel to axis of column.

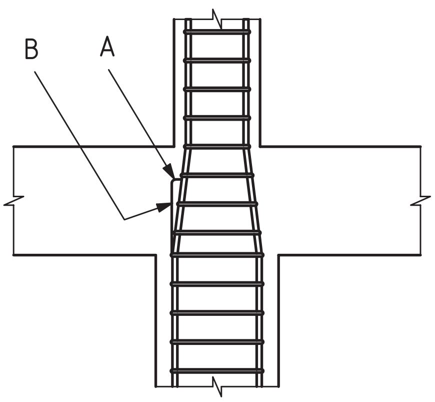
c) Horizontal support at offset bends should be provided by lateral ties or spirals.

d) Horizontal support provided should be designed to resist 1,5 times the horizontal component of the computed force in the inclined portion of an offset bar.

e) Lateral ties or spirals should be placed not more than 150 mm from points of bend.

f) Offset bars should be bent before placement in the forms.

g) Where a column face is offset from the face of the column below more than 1/6 of the depth of the girder or slab, or 80 mm, longitudinal bars should not be offset bent. Separate dowels, lap spliced with the longitudinal bars adjacent to the offset column faces should be provided. Lap splices should conform to 9.4.2.1.



The ratio of A/B should not exceed 1/6.

Figure 84 — Longitudinal bar offset

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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.

$$\text{No. of bars per face} \le \frac{b_c}{68} - 1 \qquad (99)$$

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 mm $\le b_c <$ 400 mm	3 bars
400 mm $\le h$	$\le \left(\frac{b_c}{68} - 1\right)$ 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.

$$\text{No. of bars} \le \frac{h}{22} - 6 \qquad (100)$$

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 mm $\le h$	$\le \left(\frac{h}{22} - 6\right)$ bars

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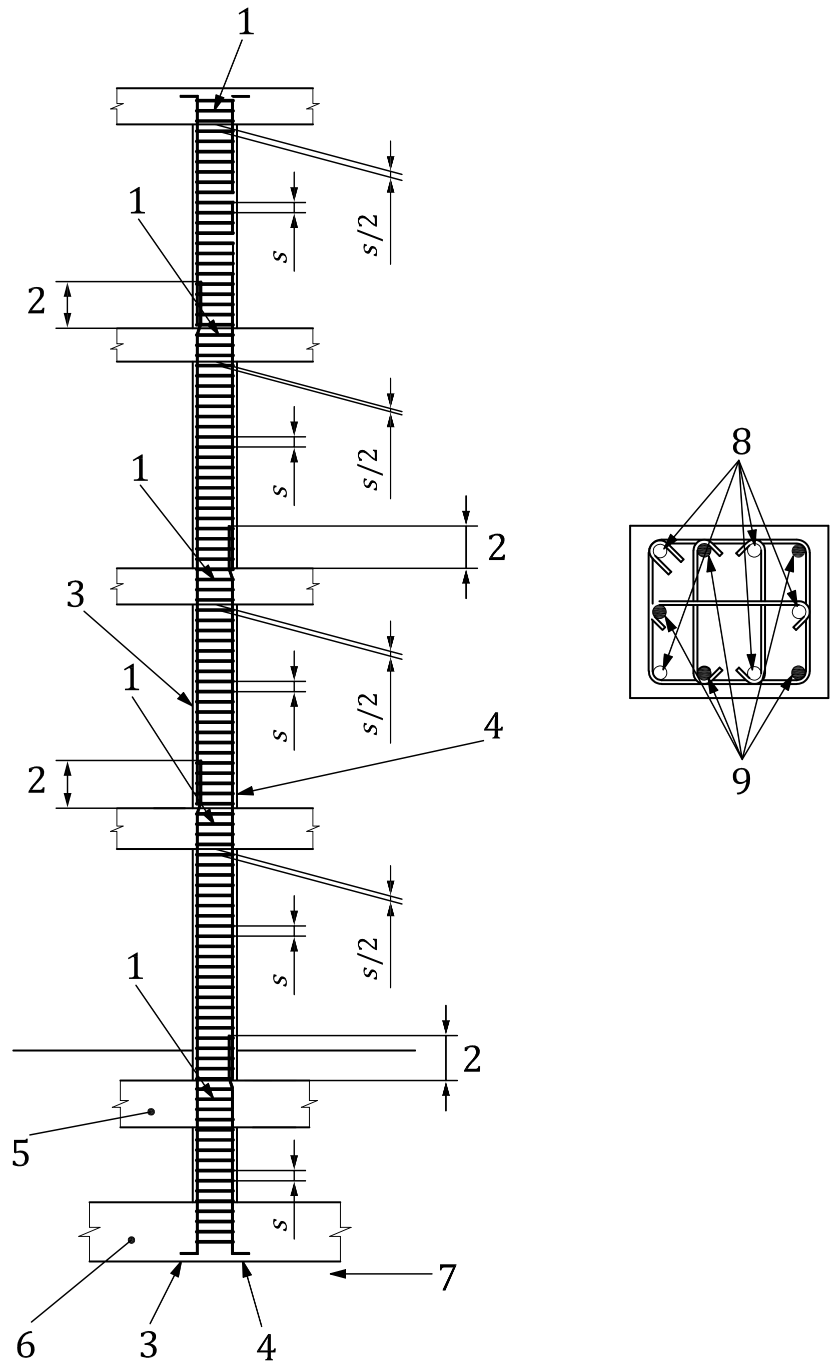

Key

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

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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):

$$A_g \approx \frac{2 \cdot (P_u)_{max.}}{f'_c} \tag{101}$$

b) For rectangular cross-sections least dimension, b, should comply with Formula (102):

$$b \begin{cases} \geq 300 \text{ mm} \\ \geq h/3 \\ \geq h_n/10 & \text{for central columns} \\ \geq h_n/9 & \text{for edge columns} \\ \geq h_n/8 & \text{for corner columns} \end{cases} \tag{102}$$

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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.

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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.

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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:

$$\phi \cdot P_{0\mathrm{n}} = \phi \cdot \left[0,85 \cdot f_\mathrm{c}' \left(A_\mathrm{g} - A_\mathrm{st} \right) + A_\mathrm{st} \cdot f_\mathrm{y} \right] \tag{109}$$

In Formula (109), ϕ = [0,70] for columns with ties and structural concrete walls and ϕ = [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:

$$\phi \cdot P_{\mathrm{n}(\mathrm{max})} \leq 0,80 \cdot \phi \cdot P_{0\mathrm{n}} \tag{110}$$

where ϕ = [0,70].

b) Columns with spiral reinforcement:

$$\phi \cdot P_{\mathrm{n}(\mathrm{max})} \leq 0,85 \cdot \phi \cdot P_{0\mathrm{n}} \tag{111}$$

where ϕ = [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, $\phi \cdot P_{\mathrm{bn}}$, and moment, $\phi \cdot M_{\mathrm{bn}}$, 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.

$$\phi \cdot P_\mathrm{bn} = \phi \cdot 0,40 \cdot f_\mathrm{c}' \cdot h \cdot b \tag{112}$$

$$\phi \cdot M_\mathrm{bn} = \phi \cdot P_\mathrm{bn} \cdot 0,25 \cdot h + \phi \cdot \left(0,95 A_\mathrm{se} + 0,16 A_\mathrm{si} \right) \cdot f_\mathrm{y} \cdot \left(\frac{h}{2} - d' \right) \tag{113}$$

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 $A_{\mathrm{se}} + A_{\mathrm{si}} = A_{\mathrm{st}}$; see Figure 86. In Formula (112), ϕ = [0,70] while in Formula (113), ϕ = [0,90]; see 6.3.3 c).

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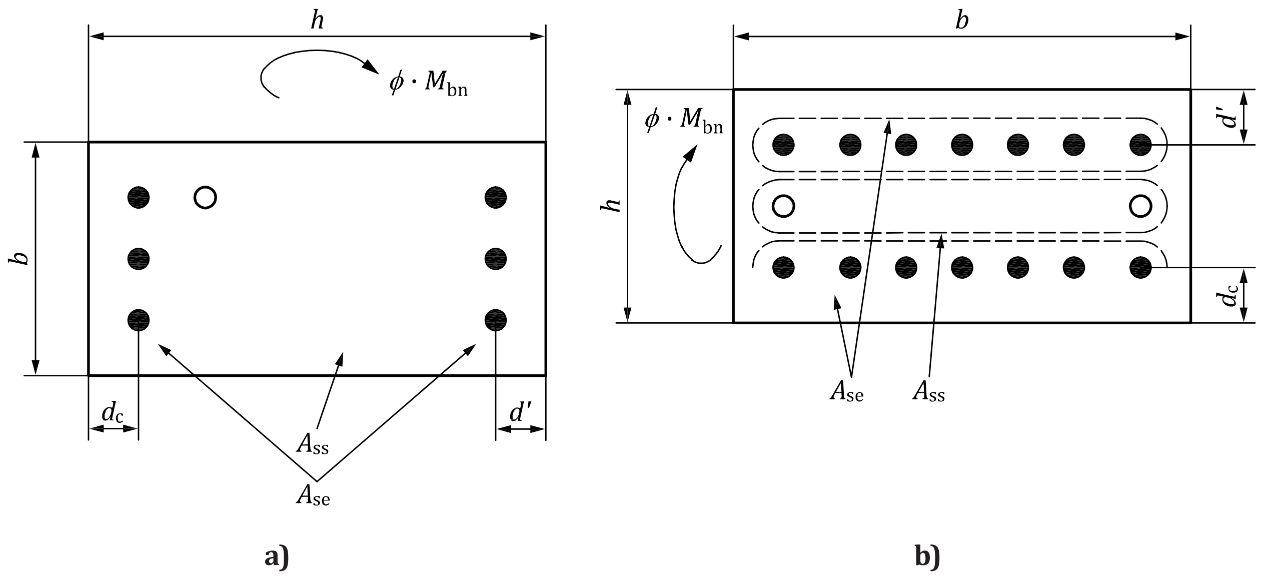

Figure 86 — Dimensions for calculation of balanced moment design strength

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:

$$\phi \cdot P_{bn} = \phi \cdot 0.4 \cdot f_c^{'} \cdot A_c \tag{114}$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0,2 \cdot h + \phi \cdot 0,5 \cdot A_{st} \cdot f_y \cdot \left(\frac{h}{2} - d' \right) \tag{115}$$

For Formula (115), h should be taken as the diameter of the section of the column. In Formula (114), ϕ = [0,75] while in Formula (115), ϕ = [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.

$$\phi \cdot P_{tcn} = \phi \cdot 0.18 \cdot f_c^{'} \cdot h \cdot b \tag{116}$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0,45 \cdot h + \phi \cdot \left(0,95 A_{se} + 0,16 A_{si}\right) \cdot f_y \cdot \left(\frac{h}{2} - d' \right) \tag{117}$$

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), ϕ = [0,70] and in Formula (117), ϕ = [0,90]; see 6.3.3 c).

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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:

$$\phi \cdot P_{tcn} = \phi \cdot 0,1 \cdot f_c^{'} \cdot A_c \tag{118}$$

$$\phi \cdot M_{tcn} = \phi \cdot P_{tcn} \cdot 0,5 \cdot h + \phi \cdot 0,6 \cdot A_{st} \cdot f_y \cdot \left(\frac{h}{2} - d^{'} \right) \tag{119}$$

For Formula (119), h should be taken as the diameter of the section of the column. In Formula (118), ϕ = [0,75] while in Formula (119), ϕ = [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):

$$\phi \cdot P_{tn} = \phi \cdot A_{st} \cdot f_y \tag{120}$$

In Formula (120), ϕ = [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):

$$\phi \cdot M_n \geq M_u \tag{121}$$

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.

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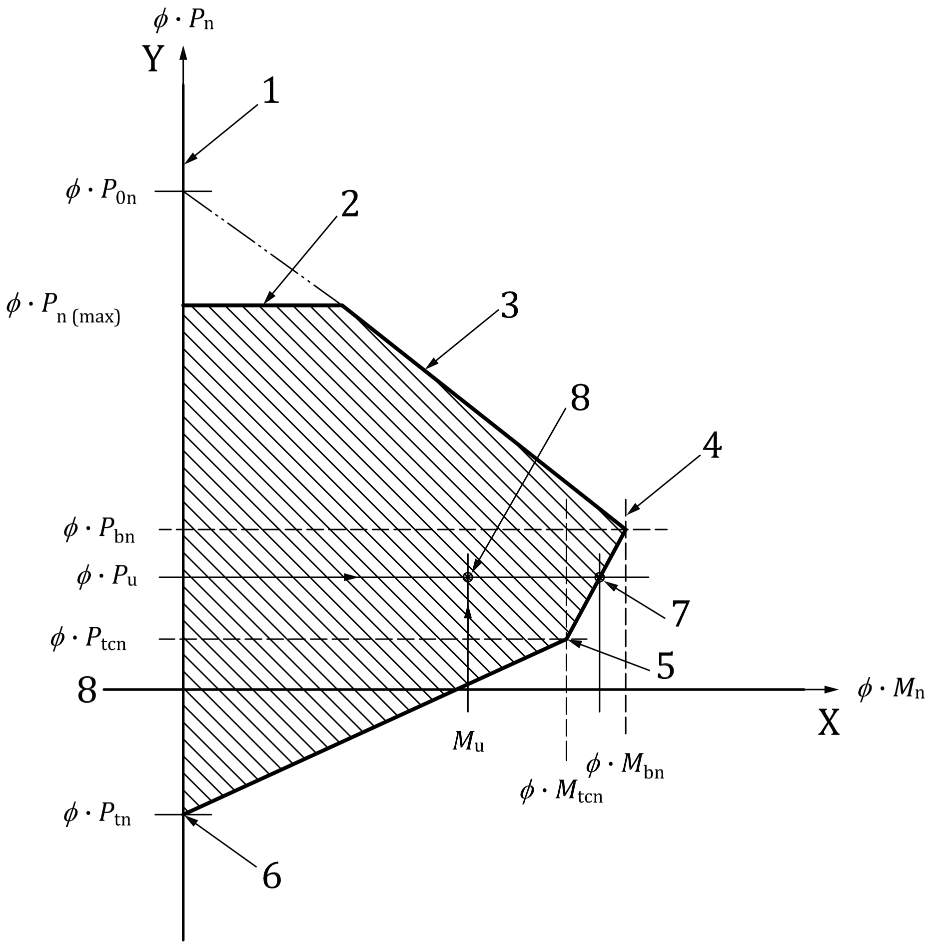

Key

1 design strength for axial compression
2 maximum allowable axial compression load
3 interaction design strength point
4 balance design strength point
5 tension-controlled strength point
6 design strength for axial tension
7 design moment strength at factored axial load level, P_u
8 required factored axial load and moment
X moment
Y axial load

Figure 87 — Interaction diagram for ($\phi \cdot M_n$, $\phi \cdot P_n$)

The conditions specified in Formula (122) to Formula (126) should be met for all couples of P_u and M_u that act on the column section:

$$P_u \leq \phi \cdot P_{n(max)} \tag{122}$$

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$$P_u \geq -(\phi \cdot P_{tn}) \tag{123}$$

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

$$M_u \leq \phi \cdot M_n = \frac{(\phi \cdot P_{0n}) - P_u}{(\phi \cdot P_{0n}) - (\phi \cdot P_{bn})} \cdot (\phi \cdot M_{bn}) \tag{124}$$

For values of $\phi \cdot P_{tcn} \leq P_u < \phi \cdot P_{bn}$:

$$M_u \leq \phi \cdot M_n = \frac{(\phi \cdot M_{bn}) - (\phi \cdot M_{tcn})}{(\phi \cdot P_{bn}) - (\phi \cdot P_{tcn})} \cdot (P_u - \phi \cdot P_{bn}) + (\phi \cdot M_{bn}) \tag{125}$$

For values of $P_u < \phi \cdot P_{tcn}$:

$$M_u \leq \phi \cdot M_n = \frac{P_u + (\phi \cdot P_{tn})}{(\phi \cdot P_{tcn}) + (\phi \cdot P_{tn})} \cdot (\phi \cdot M_{tcn}) \tag{126}$$

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, ϕ, 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):

$$\frac{(M_u)_x}{(\phi \cdot M_n)_x} + \frac{(M_u)_y}{(\phi \cdot M_n)_y} \leq 1{,}0 \tag{127}$$

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 $P_u \cdot (\phi \cdot M_n)_x$ 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.

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b) Selfweigth of the structural concrete wall.

c) Lateral forces from wind, earthquake or soil lateral pressures.

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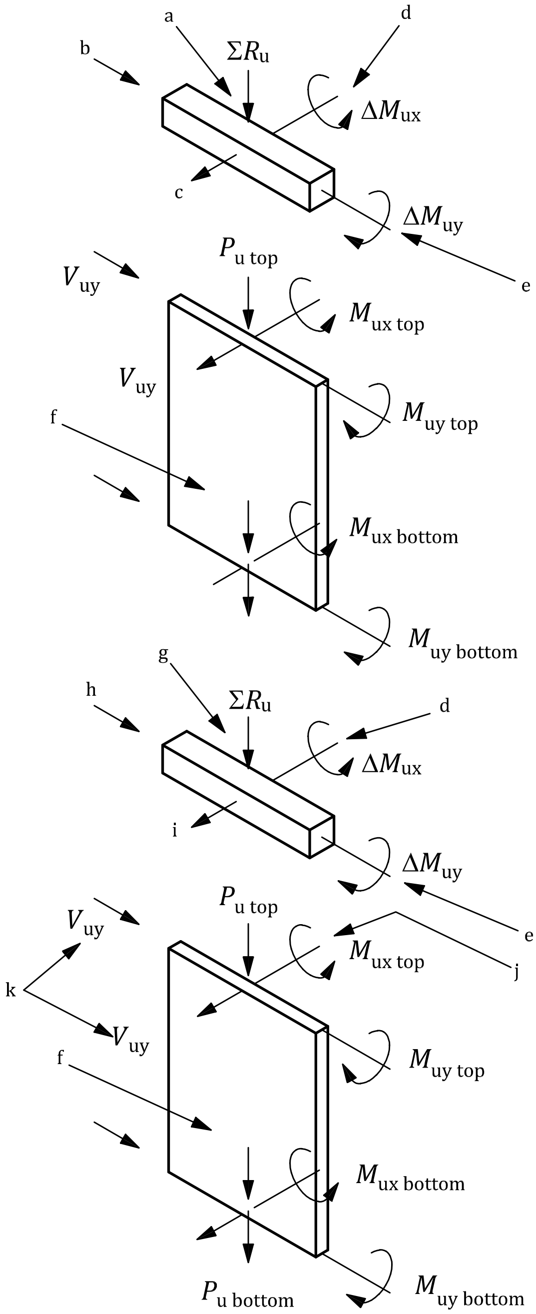

[a] Actions at the joint of story *n* from tributary elements.

[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 $\sum R_u$ 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

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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 × 10^3 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 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

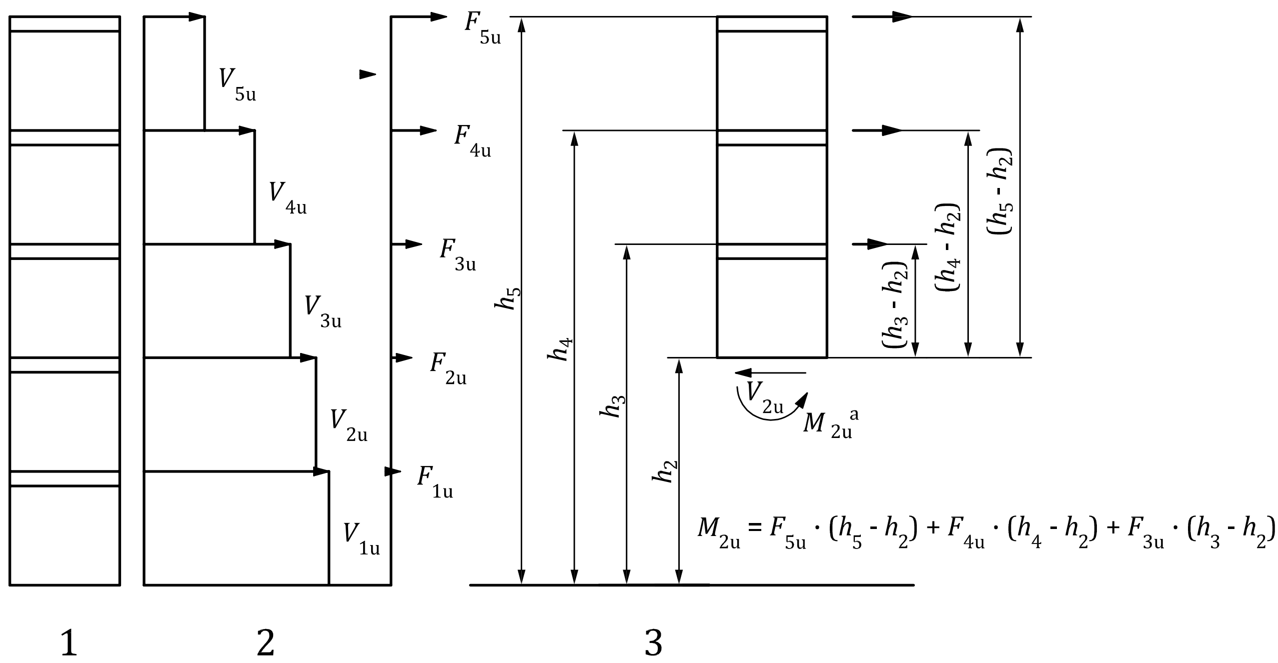
The value of the factored lateral load moment, M_u, (see Figure 89) should be established at the upper and lower part of the structural concrete wall in both principal plan direction at each story in the following manner.

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).

$$M_{xu} = \sum_{i=x}^{n} \left[F_{iu} \cdot \left(h_i - h_x \right) \right] \tag{128}$$



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

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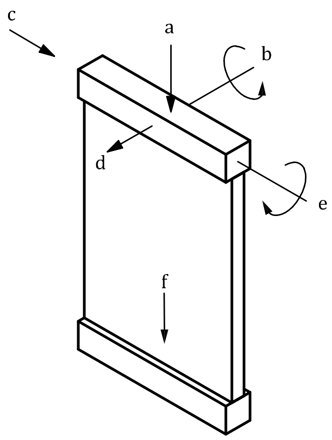
14.2.4 Factored design load

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.

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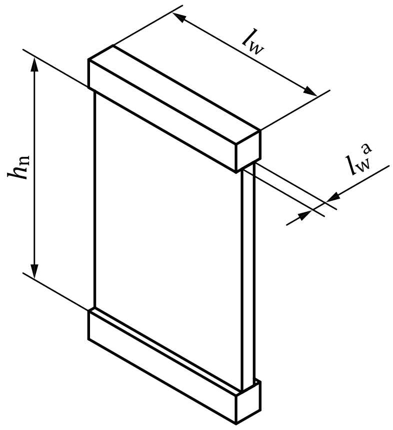

a $b_w \geq \begin{cases} 150\text{ mm} \\ h_n/20 \\ l_w/25 \end{cases}$

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.

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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, ρ_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, ρ_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, ρ_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).

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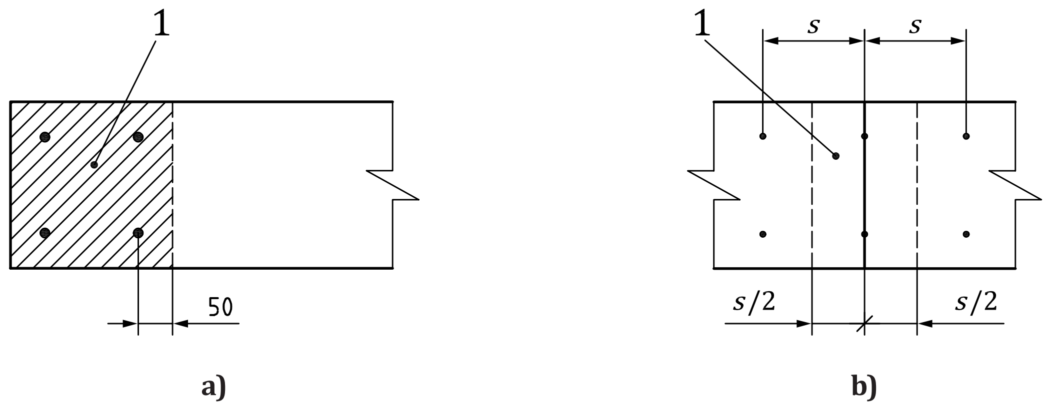

Key

1 area for computation of the steel ratio

Figure 92 — Computation of the vertical reinforcement ratio

14.4.3.3 Maximum reinforcement separation

Vertical reinforcement should not be spaced further apart than guided by 9.3.21.

14.4.3.4 Reinforcement splicing

Lap splices of vertical wall reinforcement should comply with the lap splice length of 9.4.2. It should be permitted to lap-splice all the vertical reinforcement at any given section, except at the supported element of the floor system.

14.4.3.5 End anchorage of reinforcement

Vertical reinforcement at the upper end of the structural concrete walls and at the foundation elements that transmit the loads to the underlying soil should extend to the extreme and end with a standard hook.

14.4.4 Horizontal reinforcement

14.4.4.1 General

Horizontal reinforcement should consist in one or two layers of bars or welded-wire reinforcement placed parallel with the faces of the walls and under the circumstances described in 14.4.3.2. Transverse reinforcement, as in columns, should be provided. The amount of horizontal reinforcement should be that required to resist the factored in-plane shear at the section of the structural concrete wall.

14.4.4.2 Walls with transverse reinforcement as in columns

Where the vertical reinforcement ratio, ρ_V, exceeds 0,01, vertical reinforcement should be enclosed by ties complying with the guides for column tie transverse reinforcement as guided by 9.6.4.1. See 9.5.5.2 and 14.4.4.2 a). The vertical spacing of this tie reinforcement should meet the guides for columns.

14.4.4.3 Minimum horizontal reinforcement area

The minimum horizontal reinforcement area should comply with the guides of 9.6.5.

14.4.4.4 Maximum horizontal reinforcement spacing

The maximum vertical spacing of the horizontal reinforcement should be as indicated in 9.3.21.1.

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14.4.4.5 Reinforcement splicing

It should be permitted to lap-splice the horizontal reinforcement complying with the lap splice length of 9.4.2.

14.4.4.6 End anchorage of reinforcement

Horizontal reinforcement terminating at the edges of structural walls should have a standard hook engaging the edge vertical reinforcement or should have U-shaped stirrups having the same size and spacing as, and spliced to, the horizontal reinforcement.

14.4.5 Structural concrete wall reinforcement in seismic zones

Structural concrete walls that are part of the lateral load resisting system in seismic zones, reinforcement should comply with the additional guides of Clause 16.

14.5 Flexural design guidelines

14.5.1 Required factored loads

The required factored axial load, P_u, and moment, M_u, at section under study should be obtained following the requirements of 14.2.

14.5.2 Initial trial vertical reinforcement

It should be permitted to establish initial trial area of vertical reinforcement, A_{st}, employing the minimum vertical reinforcement ratio of 9.5.5.1.

14.5.3 Required factored moment strength verification

Interaction diagrams for the structural concrete wall dimensions and reinforcement should be calculated in both directions using the guides of 13.7.2 to 13.7.4.4. The total vertical reinforcement area, A_{st}, should be divided into total extreme steel area, A_{se}, and total side steel area, A_{si}, for the direction under study, as guide by 13.7.4.3.1. The design moment in both directions should be verified employing the guides of 13.7.4.5. If the required factored moment, M_u, at required factored axial load, P_u, exceeds the design moment strength at axial load level, P_u, the area of vertical reinforcement should be increased, without exceeding the maximum reinforcement area permitted by 9.5.5.2. If an increase of the structural concrete wall dimensions is required because these limits are exceeded, the wall selfweight should be corrected, and the wall should be verified for the new dimensions. These verifications should be performed at the upper and lower sections of the same story.

14.6 Shear guides

14.6.1 Factored shear

The factored in-plane shear, V_u, should be determined from the vertical loads, and from the horizontal loads, as guided by 14.2.4.

14.6.2 Shear strength verification

The wall shear strength should be verified as follows.

a) The out-of-plane wall shear strength should be verified in accordance with the provisions for solid slabs in 9.8.4. If the factored shear, V_u, exceeds $(\phi \cdot V_c)$ as given by Formula (45) employing the wall horizontal length l_w instead of b_w, the wall thickness should be increased correcting the wall selfweight.

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b) The in-plane shear strength should be verified employing the guides of 9.8.3. The contribution of concrete to shear strength should be evaluated using 9.8.4.2. The contribution of the horizontal reinforcement of the structural concrete wall should be evaluated using 9.8.4.3. If the factored shear, V_u, exceeds ($\phi \cdot V_n$) as given by Formula (44), the amount of horizontal reinforcement should be increased complying with 9.8.4.4, where the required horizontal reinforcement ratio, ρ_h, should be obtained using Formula (46). If the factored shear, V_u, exceeds ($\phi \cdot V_n$) as given by Formula (48), the wall thickness should be increased correcting the wall selfweight.

14.7 Calculation of reactions at the foundation

14.7.1 Load reaction

The vertical load reaction, R_u, at the foundation should be equal to the value of P_u at the lower end of the structural concrete wall that is supported directly by the foundation.

14.7.2 Moment reaction

The unbalanced moment reaction ΔM_u in each principal direction at the foundation should be equal to the value of M_u in that direction at the lower end of the structural concrete wall that is supported directly by the foundation. This unbalanced moments should be distributed to the foundation elements as prescribed in Clause 15.

15 Foundations

15.1 Dimensioning of the foundation elements

The foundation elements should be dimensioned to be able to support factored loads and induced reactions, according to the appropriate design guides. The forces on the foundation elements should be transferred to the soil on which they are supported but not exceeding the allowable stresses on soil.

For footings on piles, estimate of moments and shears may be based on the assumption that any pile's reaction is applied on its centre.

The foundation elements base support area or the number and distribution of the piles should be stated from the stresses and external moments without factoring and the allowable stress on soil or allowable capacity of the piles, determined through the soil mechanics guidelines.

Due to seismic reasons and in order to avoid differential settlements, the foundation elements shall be connected between them.

15.2 Footings

15.2.1 Footings supporting circular or regular polygon-shaped columns or pedestals

At the location of critical sections of moment, shear, and development of footing reinforcement circular or regular polygon-shaped columns or pedestals concrete, may be treated as square elements with the same area.

15.2.2 Moment in footings

External moment at any section of a footing should be determined by passing a vertical plane through the footing and estimating the stresses acting on the whole of the footing area on a side of that vertical plane.

The maximum factored moment for an isolated footing should be estimated based on 15.2.2 at the following critical sections:

a) in the column, pedestal or wall face, for footings that support columns, pedestals or concrete walls;

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b) in the middle of the distance between the wall centre and the wall edge, for footings that support a masonry wall.

In one way footings and in rectangular (or square) two way footings and geometrically similar to the columns that serve as foundations, reinforcement should be distributed uniformly across its width.

15.2.3 Shear in footings

Shear strength in slabs and footings in the vicinity of the concentrated loads and reactions is ruled by the most severe of the two following conditions.

a) Action as beam for slab or footing, with a critical section that is extended over a plane through the total width and is located at a d distance from the face of the concentrated load or reaction area. For this condition, slab or footing should be designed as guide in 9.8.4.

b) Action in two directions (punching shear) for slab or footing, with a critical section perpendicular to the slab plane and located as to its perimeter, b_0, will be minimum, but not nearest to less than $d/2$ of the column sides and corners, concentrated loads or supports, or changes in thickness of the slab, such capitals or drop panels edges. Design for this condition should be performed according to the 9.8.5.

Shear in any section through a footing supported on piles should be performed according to the following indications.

a) The whole of the reaction of any pile, whose centre is located $d_p/2$ or more out the section, should be deemed as producing shear in this section.

b) Reaction of any pile, whose centre is located $d_p/2$ or more in the section, should be deemed as not producing shear in this section.

c) For middle positions from the pile centre, the pile reaction portion that is assumed to produce shear in the section should be based on a linear interpolation between the total value in $d_p/2$ out the section and the zero value in $d_p/2$ inside the section.

15.2.4 Development of reinforcement in footings

The estimate of the reinforcement development in footings should be according to 9.4.

Tension or compression reinforcement in each section should be developed on each side of that section by the appropriate length of anchoring, external anchoring, hooks or by combining all of these.

Critical sections for the reinforcement development should be assumed on the same locations defined on 15.2.2 for the maximum factored moment and all the other vertical planes where section or reinforcement changes occur.

15.2.5 Minimum footing depth

Footing thickness over the lower reinforcement should not be less than 150 mm for footings on the soil nor less than 300 mm for footings on piles.

15.2.6 Transfer of forces at base of column, wall or reinforced pedestal

All the moments and forces applied to the column, wall or pedestal base should be transferred to the footing by bearing on concrete and by reinforcement. If conditions of the required loads include lifting forces, total tensional stress should be withstood by the reinforcement.

Contact forces on surface between the supporting item and the supported one should not exceed concrete bearing strength in any of the two surfaces.

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15.2.7 Sloped or stepped footings

In sloped or stepped footings, angle of slope or depth and location of steps should be such that the design guides are satisfied at every section.

Sloped or stepped footings that are designed as a whole unit should be constructed in order to ensure its action as such.

15.3 Foundation mats

Footings supporting more than one column, pedestal or wall should be proportioned to resist the factored loads and induced reactions, in accordance with the appropriate design guides stated on these guidelines.

Soil stresses distribution under combined footings and foundation slabs should be accordingly to the soil properties and the structure and to the soil mechanics guidelines.

15.4 Footings on piles

15.4.1 General

Guides introduced in this subclause correspond to the minimum structural guides, without taking into account the digging impact effects, earth pressure and seismic effects.

15.4.2 Anchorage of reinforcement

The piles longitudinal reinforcement should be anchored on the footing, at least, at a distance equal to the length of development under tension.

15.4.3 Maximum axial stresses

Allowable maximum axial compression stresses resulting from gravitational loads are as follows.

$$D + L = 0{,}20 f_c' \cdot A_g$$

$$1{,}4D + 1{,}7L = 0{,}3 f_c' \cdot A_g$$

15.4.4 Minimum reinforcement ratios and lengths

Unless a major stress is required, the following minimum reinforcement ratios and lengths should be used:

- minimum strength of concrete 17,5 MPa;
- minimum longitudinal reinforcement ratio 0,05;
- minimum number of longitudinal bars 4;
- minimum longitudinal reinforcement length Upper half of the pile, but not less than 6,0 m;
- stirrup bar diameter 10 mm;
- maximum stirrups separation 75 mm within the upper 120 mm of the pile and 16 diameters of longitudinal bar along the reinforced area.

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15.5 Foundation beams

15.5.1 Dimensional guidelines

Grade beams dimensions should be established according to the stresses that affect them. However, it may be used as a minimum section whose highest dimension should be higher or equal to the span divided by 20.

15.5.2 Longitudinal reinforcement

Grade beams that connect foundations on piles or footings should have a continuous longitudinal reinforcement developing their yield stress, f_y, in their anchorage at the external column of the end span.

15.5.3 Transverse reinforcement

Closed stirrups should be placed along all the length with a maximum separation equal to the half of the lowest dimension of the section or 300 mm.

15.6 Retaining walls

15.6.1 Lateral earth pressure

15.6.1.1 General

Values given in 15.6 shall be used when the backfill has appropriate drainage and where hydrostatic pressure from water accumulation in the backfill is not possible. When a portion or the whole of the adjacent soil is below the water table, computation shall be based on the weight of the soil diminished by buoyancy plus full hydrostatic pressure. In the design of approximately vertical structures below grade, provisions shall be made for the lateral pressure of adjacent soil. Due allowance shall be made for possible surcharge from fixed or moving loads.

[*Earth pressure is stated generally in the form of simple linear equations. Although this treatment overlooks some characteristics of the actual behaviour, it is preferred for simplicity. The designer, however, should always bear in mind that often the lateral earth pressure distribution is not linear and earth loads tend to migrate from the more flexible to the stiffer portions of the system. Construction stages and procedures have a great influence in this load migration.*]

15.6.1.2 Internal friction and interface friction angles

For soils, the angle of internal friction, ϕ, and the interface friction angle, δ, corresponds to the relevant parameters for lateral earth pressure determination. Table 21 shall be used for the determination of these angles of the soil.

Table 21 — Angle of internal friction, ϕ and the interface friction angle, δ

SOIL	Internal friction angle, ϕ	Interface friction angle, δ
GRAVEL		
Medium size	40	22
Sandy	35	17

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Table 21 *(continued)*

SOIL	Internal friction angle, ϕ	Interface friction angle, δ
SAND		
Loose dry	28	19
Loose satured	29	19
Dense dry	35	19
Dense satured	43	19
SILT OR SILTY SAND		
Loose	20	17
Dense	25	17
CLAY		
Typical	20	17

15.6.2 Types of retaining walls

Earth-retaining structures are commonly constructed as parts of buildings construction projects and, in the form of abutment walls and wingwalls, as parts of bridge structures themselves. However, there are many different types of retaining structures.

Retaining walls are often classified in terms of their relative mass, flexibility and anchorage conditions.

Shows several types of retaining walls. Gravity walls, Type (a) are the oldest and simplest type of retaining wall. Gravity walls are thick and stiff enough that they do not bent; their movement occurs essentially by rigid-body translation and/or rotation. Cantilever walls, Type (b) which bend, as well as translate and rotate, rely on their flexural strength to resist lateral earth pressures. The actual distribution of lateral earth pressure on a cantilever wall is influenced by the relative stiffness and deformation of both the wall and the soil. Braced walls are constrained against certain types of movement by the presence of external bracing elements. In the case of bridge abutments walls, type (d), lateral movements of the tops of the walls may be restrained by the structures they support. Tieback walls and anchored bulkheads are restrained against lateral movements by anchors embedded in the soil behind the walls.

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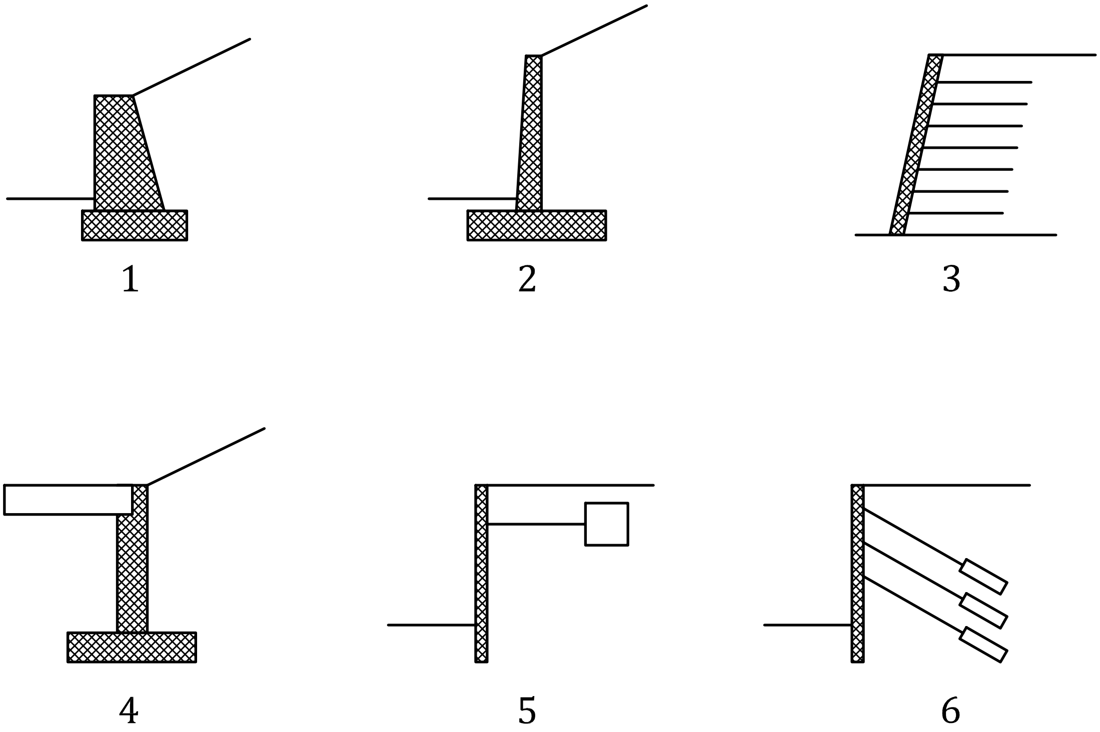

Key

1 (a) gravity wall
2 (b) cantilever wall
3 (c) reinforced soil wall
4 (d) bridge abutment wall
5 (e) anchored bulkhead
6 (f) tieback wall

Figure 93 — Types of retaining walls

15.6.3 Types of retaining wall failures

To design retaining walls, it is necessary to define failure and to know how walls can fail. Under static conditions, retaining walls are acted upon by body forces related to the mass of the wall, by soil pressures, and by external forces such as those transmitted by braces. A properly designed retaining wall will achieve equilibrium of these forces without inducing shear stresses that approach the shear strength of the soil. Failure, whether by sliding, tilting, bending, or some other mechanism, occurs when these permanent deformations become excessive. The level deformation depends on many factors and is best addressed on a sitespecific basis.

Gravity walls usually fail by rigid-body mechanism such as slading and/or overturning or by gross instability. Sliding occurs when horizontal force equilibrium is not maintained, i.e. when the lateral pressures on the back of the wall produce a thrust that exceeds the available sliding resistance on the base of the wall. Overurning failures occur when moment equilibrium is not satisfied. Bearing failures at the base of the wall are often involved. Gravity walls may also be damaged by gross instability of the soils behind and beneath them. Such failures may be treated as slope stability failures that encompass the wall.

Cantilever walls are subject to the same failure mechanisms as gravity walls and also to flexural failure mechanisms. Soil pressures and bending moments in cantilever walls depend on the geometry, stiffness, and strength of the wall-soil system. If the bending moments required for equilibrium exceed the flexural strength of the wall, flexural failure can occur. The structural ductility of the wall itself may influence the level of deformation produced by flexural failure.

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Braced walls usually fail by gross instability, tilting, flexural failure, and/or failure of bracing elements. Tilting of braced walls typically involves rotation about the point at which the brace acts on the wall, often the top of the walls as the case of bridge abutment walls.

Anchored walls with inadequate penetration may tilt by moving out their toes. Anchored walls may fail in flexure, although the point of failure is likely to be different. Failure of bracing elements can include anchor pullout, tierod failure. Backfill settlements can also impose additional axial and transverse loading on bracing elements such as tierods and tiebacks.

15.6.4 Static pressures on retaining walls

15.6.4.1 General

The seismic behavior of retaining walls depends on the total lateral earth pressures that develop during earthquake shaking. These total pressures include both the static gravitational pressures that exist before an earthquake occurs and the transient dynamic pressures induced by the earthquake.

Static earth pressures on retaining structures are strongly influenced by wall and soil movements. Active earth pressures develop as a retaining wall moves away from the soil behind it, inducing extensional lateral strain in the soil. Passive earth pressures develop as a retaining wall moves toward the soil, thereby producing compressive lateral strain in the soil. The stability of many freestanding retaining walls depends on the balance between active pressures acting predominantly on one side of the wall and passive pressures acting on the other. The distribution of the active and passive pressures can be shown triangular for linear backfill surfaces with no surface loads; in such cases, P_A and P_P act at a point h = H/3 above the base of the wall.

15.6.4.2 Active earth pressure

When the wall movement is sufficient to fully mobilize the strength of the soil behind the wall, minimum active pressures act on the wall. Because very little wall movement is required to develop minimum active earth pressures, free standing retaining walls are usually designed on the basis of minimum active earth pressures.

Under minimum active earth pressure conditions, the active thrust on a wall with the geometry shown in Figure 94 a) is obtained from the force equilibrium Figure 94 b). For the critical failure surface, the active thrust on a wall retaining a cohesionless soil can be expressed as shown in Formula (129):

$$P_A = \frac{1}{2} \cdot K_A \cdot \gamma \cdot H^2 \tag{129}$$

where K_A is the active earth pressure coefficient and shall be calculated using Formula (130):

$$K_A = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cdot \cos(\delta - \theta) \cdot \left[1 + \sqrt{\dfrac{\sin\left(\delta + \varphi\right) \cdot \sin\left(\phi - \beta\right)}{\cos\left(\delta + \theta\right) \cdot \cos\left(\beta - \theta\right)}}\right]^2} \tag{130}$$

δ is the angle of interface friction between the wall and the soil, β and θ are shown in Figure 94 a). The critical failure surface is inclined to the horizontal at an angle given in Formula (131):

$$\alpha_A = \phi + \tan^{-1}\left[\frac{\tan\left(\phi - \beta\right) + C_1}{C_2}\right] \tag{131}$$

where C_1 and C_2 are as given in Formula (132) and Formula (133), respectively:

$$C_1 = \sqrt{\tan\left(\phi - \beta\right)\left[\tan\left(\phi - \beta\right) + \cot\left(\phi - \theta\right)\right]\left[1 + \tan\left(\delta + \theta\right)\cot\left(\phi - \theta\right)\right]} \tag{132}$$

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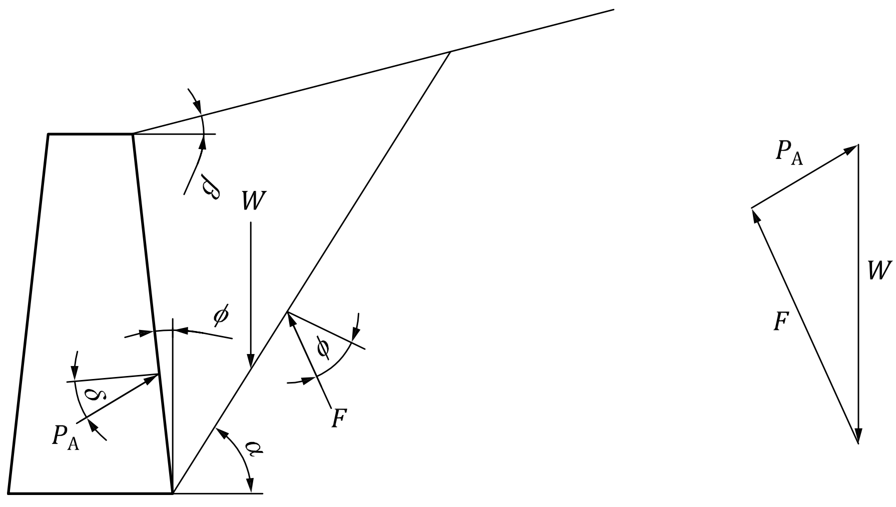
$$C_2 = 1 + \left\{ \tan\left(\delta + \theta\right) \left[\tan\left(\phi - \beta\right) + \cot\left(\phi - \theta\right) \right] \right\} \quad (133)$$



a) Triangular active wedge bounded by planar backfill surface

b) Force plygon for active wedge

Figure 94 — Active thrust on a wall under minimum active earth pressure conditions

15.6.4.3 Passive earth pressure

When the strength of the soil is fully mobilized, maximum passive earth pressures act on the wall. For maximum passive conditions in cohesionless backfills, passive thrust can be expressed as given in Formula (134):

$$P_P = \frac{1}{2} \cdot K_P \cdot \gamma \cdot H^2 \quad (134)$$

where K_p is the passive earth pressure coefficient and shall be claculated using Formula (135):

$$K_P = \frac{\cos^2\left(\phi - \theta\right)}{\cos^2\theta \cdot \cos\left(\delta - \theta\right) \cdot \left[1 + \sqrt{\frac{\sin\left(\delta + \phi\right) \cdot \sin\left(\phi + \beta\right)}{\cos\left(\delta - \theta\right) \cdot \cos\left(\beta - \theta\right)}} \right]^2} \quad (135)$$

The critical failure surface for maximum passive earth pressure conditions is inclined to the horizontal at an angle given by Formula (136):

$$\alpha_P = -\phi + \tan^{-1}\left[\frac{\tan\left(\phi + \beta\right) + C_3}{C_4} \right] \quad (136)$$

where C_3 and C_4 are as given in Formula (137) and Formula (138), respectively:

$$C_3 = \sqrt{\tan\left(\phi + \beta\right) \left[\tan\left(\phi + \beta\right) + \cot\left(\phi + \theta\right) \right] \left[1 + \tan\left(\delta - \theta\right) \cot\left(\phi + \theta\right) \right]} \quad (137)$$

$$C_4 = 1 + \left\{ \tan\left(\delta - \theta\right) \left[\tan\left(\phi + \beta\right) + \cot\left(\phi + \theta\right) \right] \right\} \quad (138)$$

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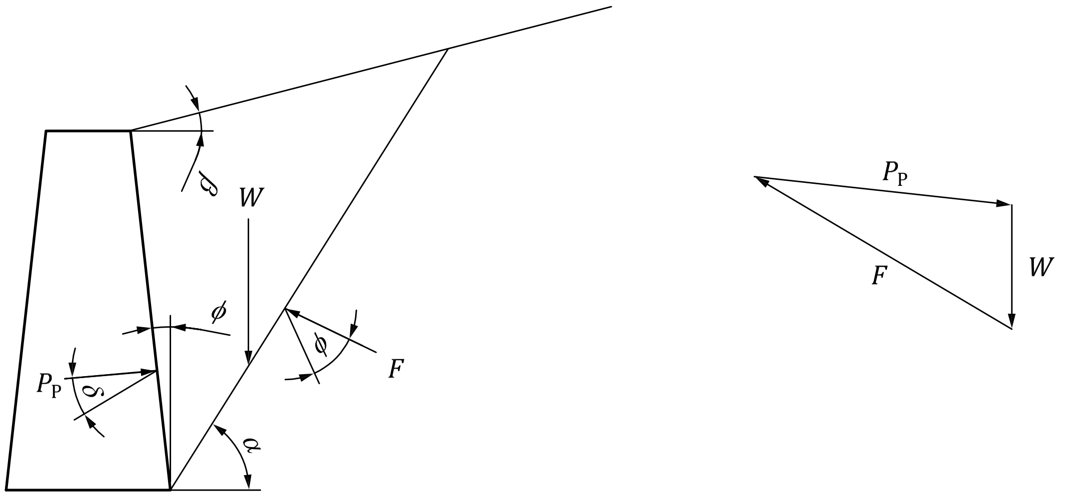

a)Triangular passive wedge bounded by planar backfill surface **b) Force plygon for passive wedge**

Figure 95 — Passive thrust on a wall under maximum passive earth pressure conditions

15.6.5 Seismic pressures on retaining walls

15.6.5.1 General

One common approach to the seismic design of retaining walls involves estimating the loads imposed on the wall during earthquake shaking and then ensuring that the wall can resist those loads. Beacause the actual loading on the retaining wall during earthquakes is extremely complicated, seismic pressures on retaining walls are usually estimated using simplificated methods.

15.6.5.2 Yielding walls

Retaining walls that can move sufficiently to develop minimum active and/or maximum passive earth pressures referred to as yielding walls.

15.6.5.2.1 Active earth pressure

In addition to the forces that exist under static conditions, the wedge is also acted upon by horizontal and vertical pseudostsatic force whose magnitudes are related to the mass of the wedge by the

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pseudostatic accelerations $a_h = k_h g$ and $a_v = k_v g$. The total active thrust can be expressed in a form similar to that developed for static conditions, that is, as given in Formula (139):

$$P_{AE} = \frac{1}{2} \cdot K_{AE} \cdot \gamma \cdot H^2 \cdot \left(1 - k_v\right) \tag{139}$$

where K_{AE} is the dynamic active earth pressure coefficient and shall be calculated using Formula (140):

$$K_{AE} = \frac{\cos^2(\phi - \theta - \psi)}{\cos\psi \cdot \cos^2\theta \cdot \cos\left(\delta - \theta + \psi\right) \cdot \left[1 + \sqrt{\frac{\sin\left(\delta + \varphi\right) \cdot \sin\left(\phi - \beta - \psi\right)}{\cos\left(\delta + \theta + \psi\right) \cdot \cos\left(\beta - \theta\right)}}\right]^2} \tag{140}$$

where $\phi - \beta \geq \psi$, $\psi = \tan^{-1}[k_h/(1 - k_v)]$

$$\alpha_{AE} = \phi - \psi + \tan^{-1}\left[\frac{-\tan\left(\phi - \psi - \beta\right) + C_{1E}}{C_{2E}}\right] \tag{141}$$

where C_{1E} and C_{2E} are as given in Formula (142) and Formula (143), respectively:

$$C_{1E} = \sqrt{\tan\left(\phi - \psi - \beta\right)\left[\tan\left(\phi - \psi - \beta\right) + \cot\left(\varphi - \psi - \theta\right)\right]\left[1 + \tan\left(\delta + \psi + \theta\right)\cot\left(\phi - \psi - \theta\right)\right]} \tag{142}$$

$$C_{2E} = 1 + \left\{\tan\left(\delta + \psi + \theta\right)\left[\tan\left(\phi - \psi - \beta\right) + \cot\left(\phi - \psi - \theta\right)\right]\right\} \tag{143}$$

The total active thrust P_{AE}, can be divided into a static component, P_A, and a dynamic component, ΔP_{AE}, as given in Formula (144):

$$P_{AE} = P_A + \Delta P_{AE} \tag{144}$$

The total active thrust will act at a height above the base of the wall, as given in Formula (145):

$$h = \frac{P_A \cdot H/3 + \Delta P_{AE} \cdot \left(0{,}6H\right)}{P_{AE}} \tag{145}$$

15.6.5.2.2 Passive earth pressure

The total passive thrust on a wall retaining a dry, cohensionless backfill is given by Formula (146):

$$P_{PE} = \frac{1}{2} \cdot K_{PE} \cdot \gamma \cdot H^2 \cdot \left(1 - k_v\right) \tag{146}$$

where K_{PE} is the dynamic passive earth pressure coefficient and shall be calculated using Formula (147):

$$K_{PE} = \frac{\cos^2(\phi + \theta - \psi)}{\cos\psi \cdot \cos^2\theta \cdot \cos(\delta - \theta + \psi) \cdot \left[1 + \sqrt{\frac{\sin\left(\delta + \phi\right) \cdot \sin\left(\phi + \beta - \psi\right)}{\cos\left(\delta - \theta + \psi\right) \cdot \cos\left(\beta - \theta\right)}}\right]^2} \tag{147}$$

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$$\alpha_{PE} = \psi - \phi + \tan^{-1}\left[\frac{-\tan\left(\phi + \psi + \beta\right) + C_{3E}}{C_{4E}}\right] \quad (148)$$

where

$$C_{3E} = \sqrt{\tan\left(\phi + \beta - \psi\right)\left[\tan\left(\varphi + \beta - \psi\right) + \cot\left(\varphi + \theta - \psi\right)\right]\left[1 + \tan\left(\delta + \psi - \theta\right)\cot\left(\phi + \theta - \psi\right)\right]} \quad (149)$$

$$C_{4E} = 1 + \left\{\tan\left(\delta + \psi - \theta\right)\left[\tan\left(\phi + \beta - \psi\right) + \cot\left(\phi + \theta - \psi\right)\right]\right\} \quad (150)$$

The total passive thrust P_{PE}, can be divided into a static component, P_P, and a dynamic component, ΔP_{PE}, as shown in Formula (151):

$$P_{PE} = P_A + \Delta P_{PE} \quad (151)$$

The total passive thrust will act at a height above the base of the wall, as given in Formula (152):

$$h = \frac{P_P \cdot H/3 + \Delta P_{PE} \cdot \left(0,6H\right)}{P_{PE}} \quad (152)$$

15.6.5.3 Non yielding walls

When retaining walls are braced against lateral movement at top and bottom, as can occur with abutment walls, the shear strength of the soil will not be fully mobilized under static or seismic conditions. As a result, the limiting conditions of minimum active or maximum passive conditions cannot be developed.

Wall pressures can be obtained from the elastic solution for the case of a uniform, constant, horizontal acceleration applied throughout the soil. For smooth rigid walls, the dynamic thrust and dynamic overturning moment can be expressed as given in Formula (153) and Formula (154):

$$\Delta P_{eq} = \gamma \cdot H^2 \cdot \frac{a_h}{g} \cdot F_p \quad (153)$$

$$\Delta M_{eq} = \gamma \cdot H^3 \cdot \frac{a_h}{g} \cdot F_m \quad (154)$$

where

a_h is the amplitude of the harmonic base acceleration;

F_p and F_m are the dimensionless dynamic thrust and moment factors shown in Figure 96.

The point of application of the dynamic thrust is at a height as given by Formula (155):

$$h = \frac{\Delta M_{eq}}{\Delta P_{eq}} \quad (155)$$

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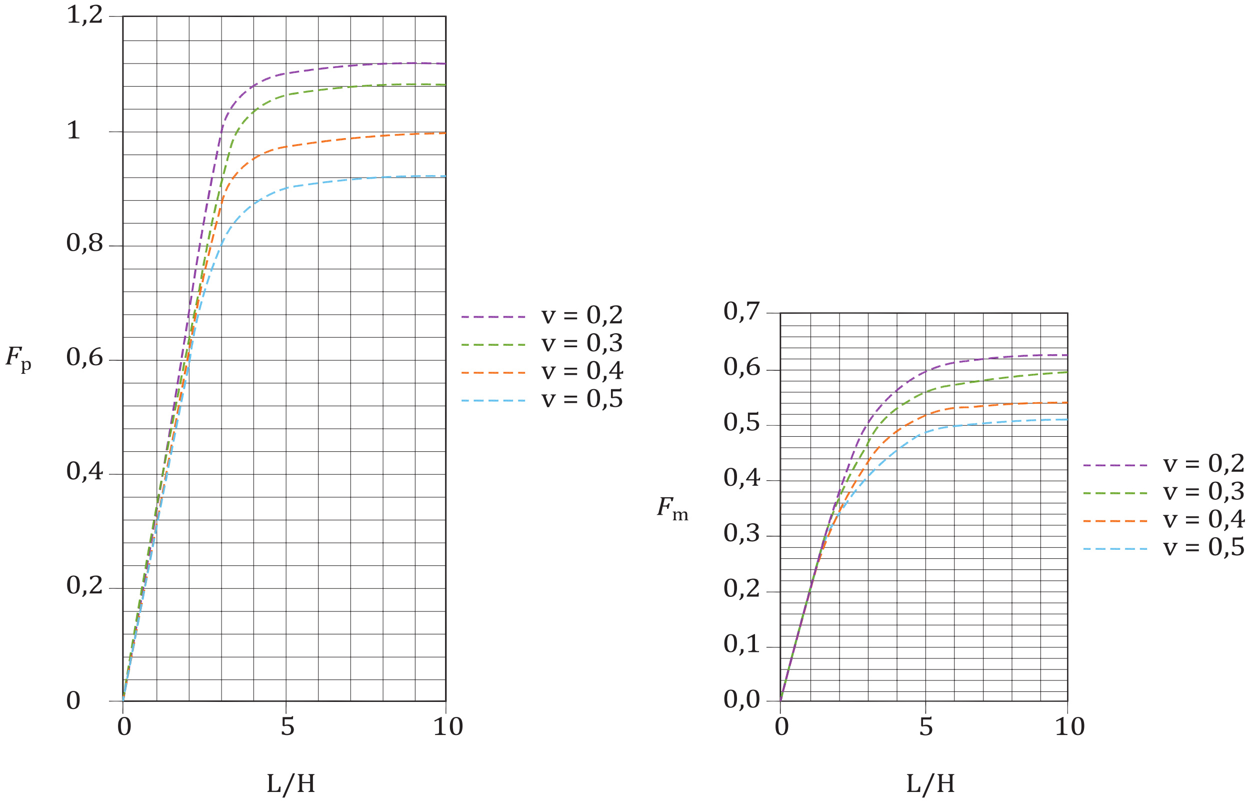

Figure 96 — Determination of dimensionless thrust and moment factor

15.6.5.4 Water effects on wall pressures

The procedures for estimation of seismic loads on retaining walls described in previous subclauses have been limited to conditions after earthquakes. Water within a backfill can also affect the dynamic pressures that act on the back of the wall. ases of dry backfills. The presence of water plays a strong role in determining the loads on waterfront retaining walls both during and the total water pressures that act on retaining walls in the absence of seepage within the backfill can be divided in two components: hydrostatic pressure, which increases linearly with depth and acts on the wall before, during and after an earthquake, and hydrodynamic pressure, which results from the dynamic response of the water itself.

15.6.5.4.1 Water outboard of wall

The resultant hydrodynamic thrust is given by Formula (156):

$$P_W = \frac{7}{12} \cdot \frac{a_h}{g} \cdot \gamma_w \cdot H^2 \tag{156}$$

Another important consideration in the design of a waterfront retaining wall is the potential for rapid drawdown of the water outboard of the wall.

15.6.5.4.2 Water in backfill

The presence of water in the backfill behind a retaining wall can influence hydrodynamic water pressures under free pore-water conditions and should be added to the computed soil and hydrostatic pressures to obtain the total loading on the wall.

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For restrained porewater conditions, 15.6.5.1 method can be modified to account the presence of porewater in the backfill. Representing the excess porewater pressure in the backfill by the pore pressure ratio, r_w, the active soil thrust acting on a yielding wall can be computed from Formula (139).

$$\gamma = \gamma_b \cdot \left(1 - r_w\right) \tag{157}$$

$$\psi = \tan^{-1}\left[\frac{\gamma_{sat} \cdot k_h}{\gamma_b \cdot \left(1 - r_w\right) \cdot \left(1 - k_v\right)}\right] \tag{158}$$

15.6.6 General requirements for retaining walls

15.6.6.1 Support top and bottom

Retaining walls shall be supported top and bottom by the structure. The opening of windows for lighting and ventilation purposes can cause a captive column problem in seismic zones. In those cases, alternatives of lighting and ventilation shall be used.

15.6.6.2 Drainage

Appropriate drainage, such as through filters, drains and weep-holes, to prevent water from building up within the backfill and dissipation of hydrostatic pressure shall be used.

15.6.6.3 Backfill material

The following materials are unsuitable for backfill material and shall not be used:

— organic silts and silt clays;

— low plasticity;

— inorganic clayey silts;

— elastic silts;

— inorganic clays of high plasticity;

— organic clays;

— silty lays.

15.6.6.4 Surcharge

Any expected surcharge, during construction or use of the structure, shall be included in the design of the wall.

15.6.6.5 Minimum thickness

The minimum thickness of retaining walls shall be 0,20 m.

15.6.7 Details of reinforcement

15.6.7.1 General

Retaining walls shall comply with the requirements of 14.4.

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15.6.7.2 Flexural requirements

The required moment strength of the retaining wall shown in Figure 97 shall be used to proportion the reinforcement, using a unit width of wall. The reinforcement area and spacing shall be determined using the procedures of for slabs. The computation of the distance from extreme tension fibre to centroid of tension reinforcement, *dc*, shall consider the concrete cover for members cast and permanently exposed to earth, the bar diameters used, and the perpendicular reinforcement placed between the reinforcement considered and the concrete surface. A value of *dc* = 0,1 m shall be used to compute *d* as $d = h - d_c$.

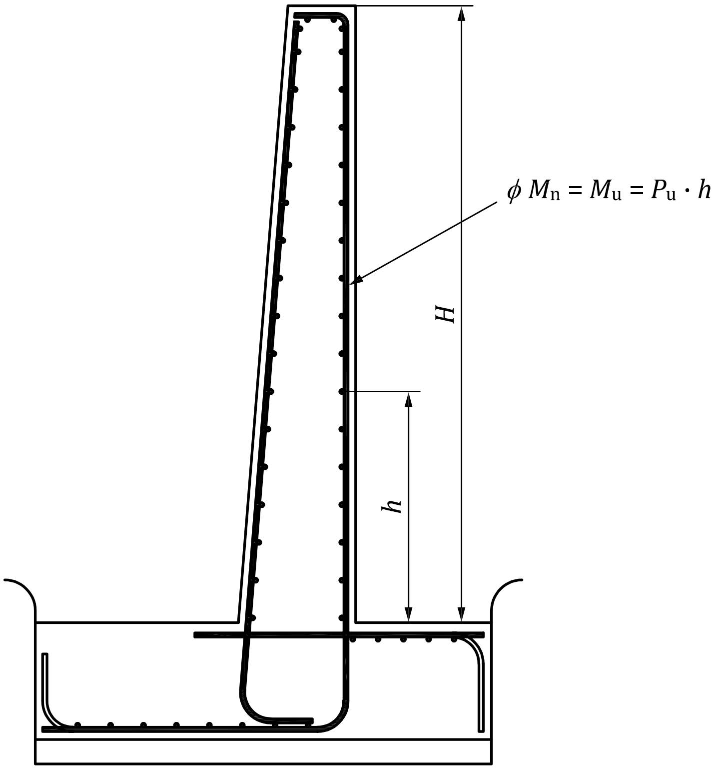

Figure 97 — Design moments and reinforcement for retaining walls

15.6.7.3 Shear requirements

The factored shear for the retaining wall shall be determined from Formula (159):

$$V_u = 1{,}15 \frac{P_u \cdot l}{2} \tag{159}$$

The requirements of 11.3.5 for shear strength of solid slabs shall be met.

16 Lateral load resisting system

16.1 General

The resistance to lateral (horizontal) forces, under the present guidelines, should be evaluated and provided for, following the guides of Clause 16. Wind forces, earthquake forces, and soil lateral pressure are covered.

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16.2 Specified lateral forces

16.2.1 General

The specified lateral forces prescribed in Clause 8 should be employed in design. The simultaneous occurrence of the prescribed lateral forces with other forces and loads should be evaluated employing the load combination guidelines of 8.1.1. When the lateral forces are applied to structural and non-structural, elements a continuous load path from the point of application of the force to the lateral-force resisting structural elements should be devised, and adequate strength should be provided to all elements along the load path.

16.2.2 Lateral forces included

16.2.2.1 Wind forces

Wind forces should be determined employing the guidelines of 8.1.6.

16.2.2.2 Earthquake forces

Earthquake forces should be determined employing the guidelines of 8.1.7.

16.2.2.3 Soil lateral forces

Soil lateral pressure forces should be determined employing the guidelines of 8.1.1.5.

16.2.2.4 Lateral fluid pressure

Auxiliary structures subjected to lateral fluid pressure, such as tanks, should be self-contained and the lateral fluid pressure should be compensated within the auxiliary structure. The main building structure should not be employed to resist any lateral forces derived from the contained liquids.

16.3 Lateral force resisting structural system

16.3.1 General

The lateral force resisting system comprises the structural elements that acting jointly support and transmit to the ground the lateral forces arising from earthquake motions, wind, and lateral earth pressure. See Figure 98.

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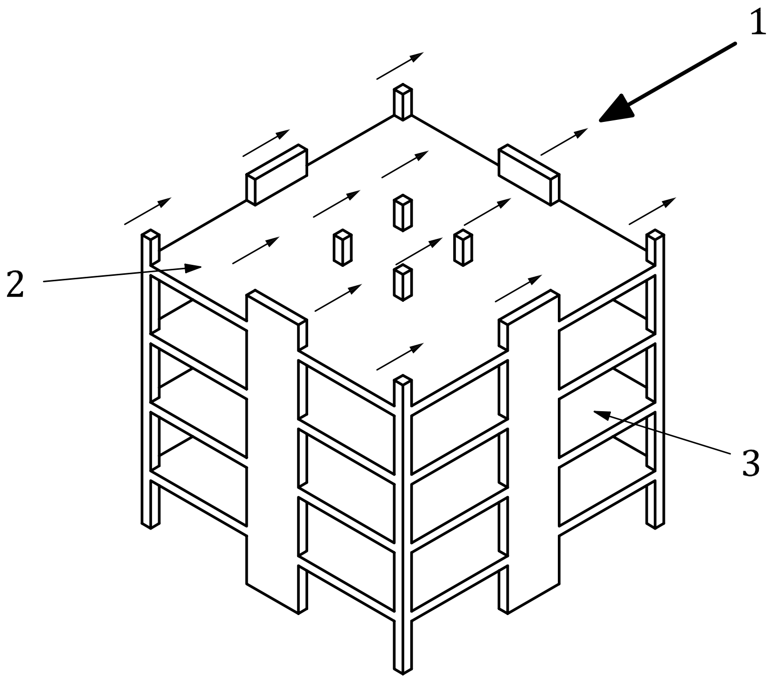

Key

1 prescribed floor lateral forces

2 floor diaphragm

3 lateral load resisting structural walls

Figure 98 — Lateral force structural resisting system

The floor system should act as a diaphragm that carry in its plane the lateral force from the point of application to the vertical elements of the lateral force resisting system. The vertical elements of the lateral force resisting system, in turn, collect the forces arising from all floors affected and carry them down to the foundation, and through the foundation to the underlying soil. Under the guides of this document, the main vertical elements of the lateral force resisting system should be a number of structural concrete walls in both principal directions in plan. These structural concrete walls should have no openings for windows or doors.

16.4 Minimum amount of structural concrete walls

16.4.1 General

A minimum amount of reinforced concrete structural walls should be provided for factored lateral force resistance. These structural walls should comply with the following guides.

a) The structural walls should have rectangular cross-sections. Other cross sections are beyond the scope of this document.

b) The structural walls should be continuous from foundation to roof, except for resistance for unbalanced lateral soil pressure in a basement where the walls for this purpose should be permitted to be suspended at the level of the first story.

c) The structural walls should be aligned vertically, and where reductions of section occur, the cross-section of the wall underneath should be greater than the one of the wall in the floor above, and the centroid of the cross-section of wall above should be in the middle third, in both directions, of the cross-section of the wall below.

d) The structural walls should have no openings for windows or doors.

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e) In both principal direction in plan, there should be at least two groups consisting each one in one or more walls acting in the same plane parallel to the principal direction, and the planes of each group should be as further apart as possible. Priority should be given to place the structural walls in the periphery of the building.

f) The walls should be located as symmetrically as possible with respect to the centers of mass and stiffness of each floor.

Dimensions of the structural walls should comply with the guides of 16.4.2 and 16.4.3.

16.4.2 Wall area guideline for shear strength

At any floor, i, for the two principal directions, x and y, the sum of the cross-section areas ($A_g = l_w \cdot b_w$) for all structural walls acting in the principal direction under study should be obtained from Formula (160):

$$\sum \left(l_w \cdot b_w \right) \geq \frac{V_u}{\frac{1}{9} \cdot \sqrt{f_c'}} \tag{160}$$

In Formula (160), only walls where the horizontal length of wall, l_w, is parallel to the direction under study should be included, b_w corresponds to the wall thickness, and V_u should be obtained from the guides of 14.6. The guideline area should be divided into a minimum of two groups of walls as guideline by 16.4.1 e).

16.4.3 Wall dimensions guideline for lateral stiffness

The slenderness ratio (h_w/l_w) for any individual wall should comply with Formula (161):

$$\frac{h_w}{l_w} \leq 4 \tag{161}$$

In Formula (161), h_w corresponds to the total height of the wall from the foundation to the roof, and l_w corresponds to the horizontal length of wall. The thickness of wall, b_w, should comply with the guidelines of Clause 14.

16.5 Special reinforcement details for seismic zones

16.5.1 General

The following additional guides for the structural elements mentioned should be employed in buildings designed under these guidelines located in seismic zones.

16.5.2 Girders of frames

16.5.2.1 Dimensional guides

The minimum width, b_w, for girder should be 250 mm, and it should comply with the guides of Clause 11.

16.5.2.2 Longitudinal reinforcement

In addition to the guides of 12.3.5.5, the following guides should be met.

a) At least two bars should be provided both top and bottom.

b) At any section, both the ratios of positive and negative reinforcement should be greater than or equal to the minimum guide by 9.5.3.1.

c) At any section, both the ratios of positive and negative reinforcement should not exceed 0,025.

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d) The area of positive reinforcement at the joint faces should not be less than one-half the area provided for negative reinforcement at the same joint face.

e) The area of positive and negative reinforcement at any section should not be less than one-fourth of the maximum negative reinforcement area at the face of either joint.

f) Lap splices should not be used in the zones comprised by the beams-columns joint and the confinement zones defined in 16.5.2.3 a). The full length of the lap splice should have confinement stirrups as defined in 16.5.2.3 b) with a maximum spacing of $d/4$ or 100 mm.

16.5.2.3 Transverse reinforcement

In addition to the guides of 12.3.5.5.3, the following guides should be met.

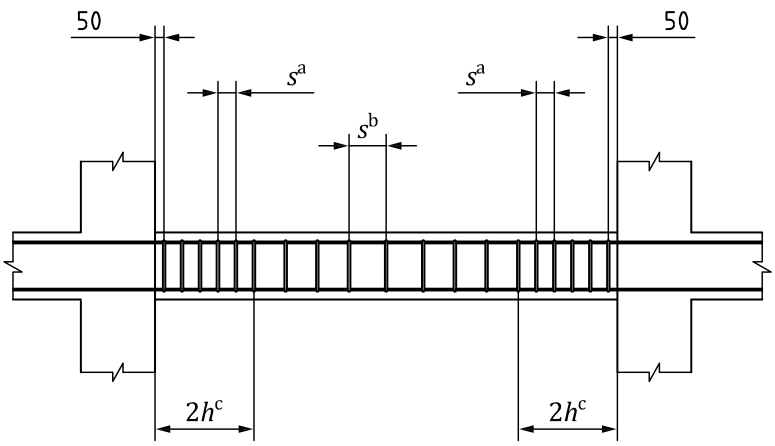
a) Over a distance equal to twice the member depth, h, measured from the face of the supporting element toward midspan, at both ends of the girder, the transverse reinforcement should be confinement stirrups. See Figure 99.

b) Confinement stirrups should be closed stirrups at least 10 mm in diameter, with hooks as defined in 9.3.12 d), and complying with the guides for column ties of 9.6.4.1. Crossties should comply with the guides of 9.3.12 e).

c) First confinement stirrup should be located no more than 50 mm from the face of the supporting element.

d) Maximum spacing of confinement stirrups should not exceed $d/4$ nor 125 mm.

e) For the central part of the girder span, between confinement zones, the transverse reinforcement should be closed stirrups with hooks complying with 9.3.12 d) and the maximum stirrup spacing should not exceed more than $d/2$.



a $s \leq \begin{cases} d/4 \\ 125\ \text{mm} \end{cases}$

b $s \leq d/2$

c Confinement zones.

Figure 99 — Confinement stirrup spacing

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16.5.2.4 Shear strength

In addition to the guides of 12.3.5.5.3, the following guides should be met.

a) The additional factored design shear force, ΔV_e, corresponding to the probable flexural capacity development of the span at the faces of the joints should be obtained as the largest value from Formula (162) and Formula (163). See Figure 100.

$$\Delta V_e = \frac{\left(M_{pr}^{+}\right)_{left} + \left(M_{pr}^{-}\right)_{right}}{l_n} \qquad (162)$$

$$\Delta V_e = \frac{\left(M_{pr}^{-}\right)_{left} + \left(M_{pr}^{+}\right)_{right}}{l_n} \qquad (163)$$

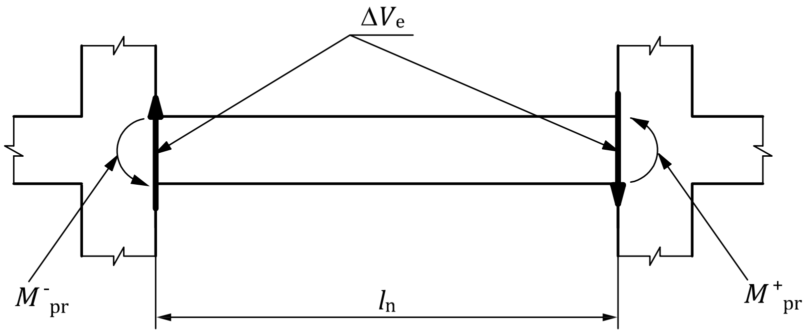

Figure 100 — Calculation of ΔV_e

b) In Formula (162) and Formula (163), M_{pr}^{+} and M_{pr}^{-} correspond, respectively, to the positive and negative flexural probable strength at the joint faces, obtained from Formula 33 using the corresponding longitudinal reinforcement area, 1,25 f_y and a strength reduction factor $\phi = 1$.

c) The largest value of ΔV_e obtained from Formula (162) and Formula (163) should be added to V_u at the faces of the supports and the shear diagram should be recalculated as guided in 12.3.5.5.3. See Figure 101.

d) The guide transverse reinforcement for shear should be obtained as prescribed in 12.3.5.5.3, except that if ΔV_e is greater than V_u for the gravity loads at the face of the support, in computing the shear reinforcement, the contribution of concrete to the shear strength should be taken as ($\phi \cdot V_c = 0$) in the confinement zones guide in 16.5.2.3 a).

e) The confinement stirrups guide by 16.5.2.3 should be permitted to be employed as part of the required shear reinforcement.

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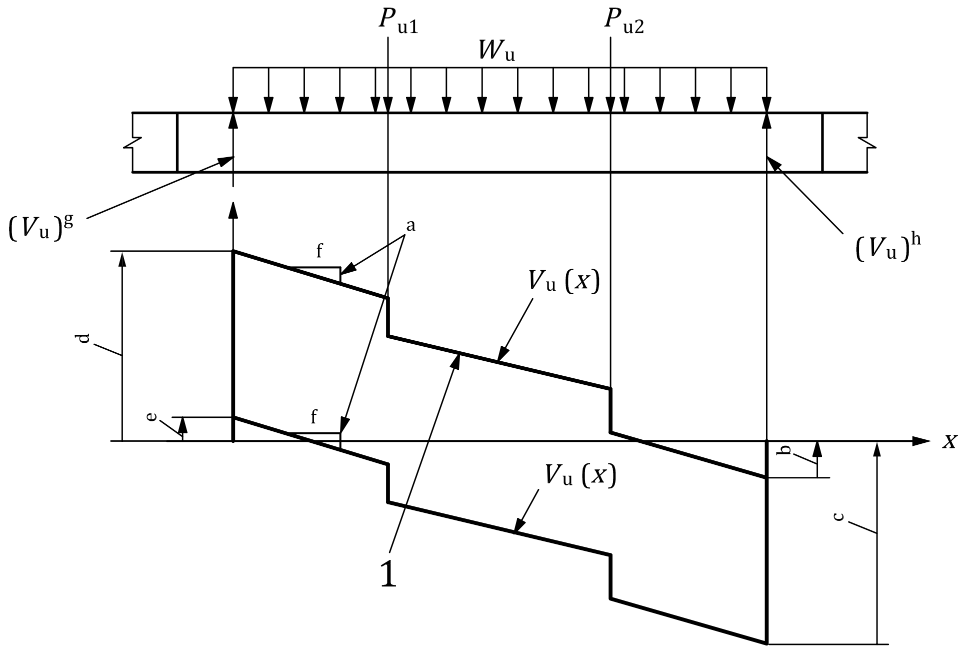

Key

1 shear euvelope

a $[(V_u)2 + (V_u)1 - \Sigma P_u]\ 1/l_u$

b $(V_u)\ 1 - \Delta V_e$

c $(V_u)\ 1 + \Delta V_e$

d $(V_u)\ 2 + \Delta V_e$

e $(V_u)\ 2 - \Delta V_e$

f Right supp.

g Left supp.

Figure 101 — Calculation of the envelope of shear in the girder

16.5.3 Columns

16.5.3.1 Dimensional guides

The guides of 16.3 should be complied with and 13.3.2.1 should change to the following:

a) the shortest cross-sectional dimension should not be less than 300 mm;

b) the ratio of the largest cross-sectional dimension to the perpendicular shortest dimension should not exceed 2,5.

16.5.3.2 Longitudinal reinforcement

The guides of 13.4 should be complied with, and in 13.4.2.6 should change to restrict the location of splices only to the centre half of the member length.

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16.5.3.3 Minimum flexural strength of columns

Unless the full clear length of the column is provided with transverse reinforcement complying with 16.5.3.4, the flexural strengths of the columns should satisfy Formula (164):

$$\sum M_c \geq \frac{6}{5}\sum M_g \tag{164}$$

where

$\sum M_c$ is the sum of the lowest flexural strengths ($\phi \cdot M_n$) of the columns framing into the joint;

$\sum M_g$ is the sum of the flexural strength ($\phi \cdot M_n$) of the girders framing into the joint.

The flexural strengths of the columns should correspond to the lowest flexural strength computed using the appropriate formula for the range of axial loads P_u that act on the column. Flexural strengths should be summed such that the column moments oppose the beam moments. Formula (164) should be satisfied for beam moments acting in both directions in the vertical plane of the frame considered. See Figure 102.

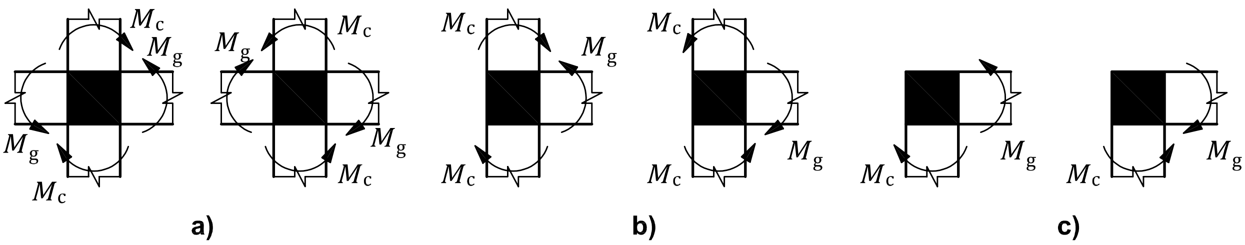

Figure 102 — Minimum flexural strength of columns

16.5.3.4 Columns with transverse reinforcement in the form of ties

When the column transverse reinforcement are ties, in addition to the guides of 9.6.4.1 and 16.4.3, the following guides should also be met.

a) Over a distance l_0 not less than the largest column cross-section dimension, one-sixth of the clear length of the member, or 500 mm, equal to twice the member depth measured from the face of the supporting element toward midspan, at both ends of the girder, the transverse reinforcement should be confinement stirrups. See Figure 103.

b) Confinement ties should be closed single or overlapping ties with hooks as defined in 9.3.12 d) and complying with the guides for column ties of 9.6.4.1.

c) It should be permitted to use crossties complying with the guides of 9.3.12 e) of the same bar diameter and spacing as the confinement ties. Each end of the crosstie should engage a peripheral longitudinal reinforcing bar. Consecutive crossties should be alternated end to end along the longitudinal reinforcement.

d) For each direction parallel to sides of the cross-section, the maximum horizontal distance, measured centre-to-centre, between legs of the peripheral confinement tie and crossties, and between crossties, should not exceed 200 mm or one-half of the smallest cross-section dimension; otherwise, additional crossties should be provided. If the number of legs of confinement ties and crossties exceeds the number of bars located in that face of the cross section, additional longitudinal bars should be provided. See Figure 103.

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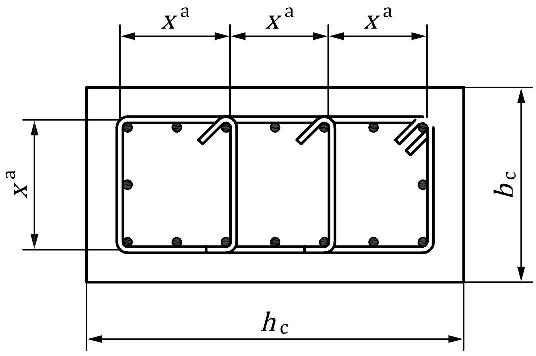

a $x \leq \begin{cases} 200\text{ mm} \\ b_c/2 \end{cases}$

Figure 103 — Separation between legs of confinement ties and crossties

e) Maximum spacing of confinement ties should not exceed 100 mm the value obtained from Formula (165).

$$s \leq \frac{A_b \cdot f_{ys}}{f'_c \cdot 15\text{ mm}} \tag{165}$$

In Formula (165), A_b is the confinement tie and crosstie bar area, and f_{ys} is the nominal yield strength of the confinement tie and crosstie. See Figure 104.

f) The first confinement tie should be located no more than 50 mm from the face of the joint.

g) When reinforcement as indicated above is not placed in all the column clear length, in the central part of the column clear length between confinement zones, the transverse reinforcement should be confinement ties of the same diameter, strength, f_{ys}, and number of crossties employed in the confinement zones, and the maximum centre-to-centre spacing should not exceed the smaller of six times the diameter, d_b, of the longitudinal column bars or 150 mm. See Figure 104.

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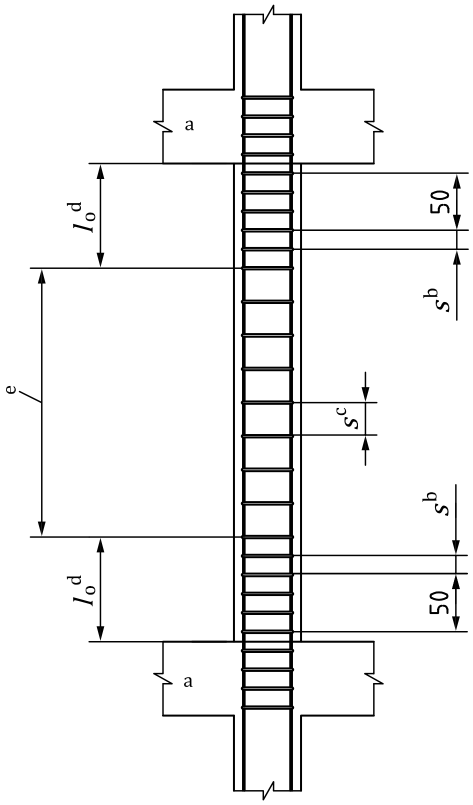

a Join transverse reinforcement as guided by 16.5.4.3.

b $s \le \begin{cases} 100\ \text{mm} \\ \dfrac{A_b \cdot f_{ys}}{f_c' \cdot 15\ \text{mm}} \end{cases}$

c $s \le \begin{cases} 6d_b \\ 150\ \text{mm} \end{cases}$

d $l_o \ge \begin{cases} h_c \\ h_n/6 \\ 500\ \text{mm} \end{cases}$

e Long. reinforcement lap splices may be made in the central zone.

Figure 104 — Confinement tie spacing in columns

16.5.3.5 Columns with transverse reinforcement in the form of spiral

When the column transverse reinforcement are spirals, in addition to the guides of 9.6.4.2 and 13.4.3, the following guides should also be met.

a) Over a distance not less than the largest column cross-section dimension, one-sixth of the clear length of the member, or 500 mm, equal to twice the member depth measured from the face of the supporting element toward midspan, at both ends of the girder, the transverse reinforcement should be spiral complying with the other guides of 16.5.3.5.

b) The volumetric ratio of the spiral should not be less than indicated by Formula (29) and by Formula (166):

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$$\rho_s = \frac{A_b \cdot \pi \cdot d_c}{A_c \cdot s} \geq 0,12 \cdot \frac{f'_c}{f_{ys}} \tag{166}$$

c) When spiral reinforcement as indicated above is not placed in all the column clear length, in the central part of the column clear length between confinement zones, the transverse reinforcement should be spiral of the same diameter and yield strength, f_{ys}, employed in the confinement zones and the maximum centre-to-centre spacing should not exceed the smaller of six times the diameter, d_b, of the longitudinal column bars or 150 mm.

16.5.3.6 Shear strength

In addition to the guides of 13.6, the following guides should be met.

a) The factored design shear force, ΔV_e, corresponding to the probable flexural capacity development of the column at the faces of the joints should be obtained employing Formula (167) for both principal directions in plan. See Figure 105.

$$\Delta V_e = \frac{(M_{pr})_{top} + (M_{pr})_{bottom}}{h_n} \tag{167}$$

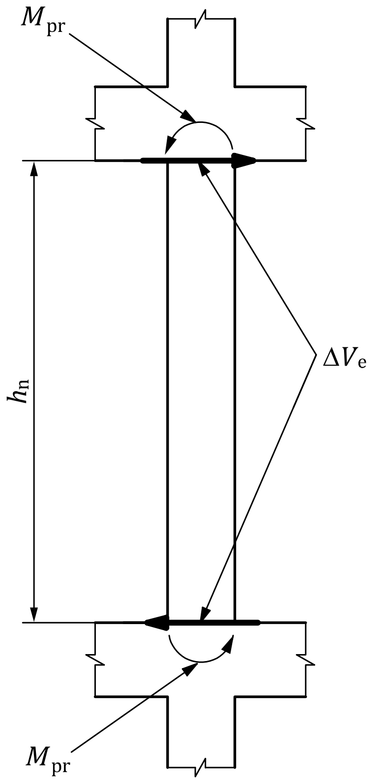

Figure 105 — Calculation of ΔV_e for columns

b) In Formula (167), M_{pr} corresponds to flexural probable strength at the joint faces, obtained using 1,25 f_y and a strength reduction factor ϕ = 1. The flexural strengths of the columns should correspond to the lowest probable flexural strength computed using the appropriate formula, for the range of axial loads, P_u, that act on the column. The factored design shear force for the column, ΔV_e, does not need to exceed the value determined from the joint strength based on probable moment strength M_{pr} of the girders framing into the joint obtained in 16.5.2.4. See Figure 106.

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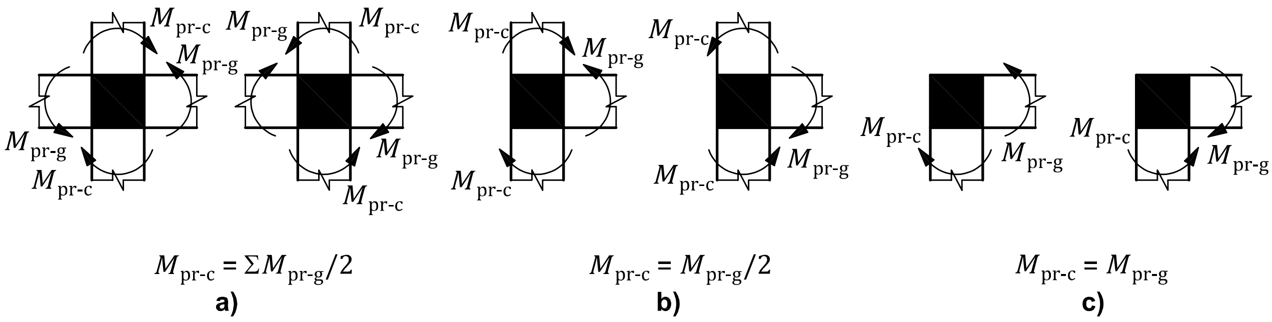

Figure 106 — Maximum M_{pr} for the columns needed to obtain column shear ΔV_e

c) The required transverse reinforcement for shear should be obtained as prescribed in 13.4.3, except that in computing the shear reinforcement, the contribution of concrete to the shear strength should be taken as ($\phi \cdot V_c = 0$) in the confinement zones guide in 16.5.3.5 a) and 16.5.3.6 a).

d) The confinement ties or spiral guide by 16.5.3.4 and 16.5.3.5 should be permitted to be employed as the guide shear reinforcement.

16.5.4 Joints

16.5.4.1 General

The guides of 16.5.4 should be complied with at joints of frames located in seismic zones, instead of the guides of 9.6.4.3.

16.5.4.2 Limit on column dimensions at the joint based of girder longitudinal reinforcement

Where longitudinal reinforcement extends through the column-girder joint, the column dimension parallel to the beam should not be less than 20 times the diameter d_b of the largest longitudinal girder bar.

16.5.4.3 Transverse reinforcement within the joint

The following amounts of transverse reinforcement within the joint should be provided within the column-girder joint.

a) Horizontal transverse confinement reinforcement in the same amounts and spacing as guided by 16.5.3.4 should be provided within the column-girder joint. If girders having a width greater than or equal to 3/4 of the column width, frame in all four lateral sides of the joint the confinement ties spacing can be two times as guide by 16.5.4.3, without exceeding 150 mm.

b) When the longitudinal girder reinforcement is located outside the joint confined core provided by the reinforcement required by a), vertical confinement stirrups as guide for girders by 16.5.2.3 should be provided to confine it.

16.5.4.4 Joint shear strength

The horizontal shear strength within the joint should be verified for the shear forces that develop due to the probable moment strength of columns and girders that frame into the joint. See Figure 107. The following guides should be employed for the verification.

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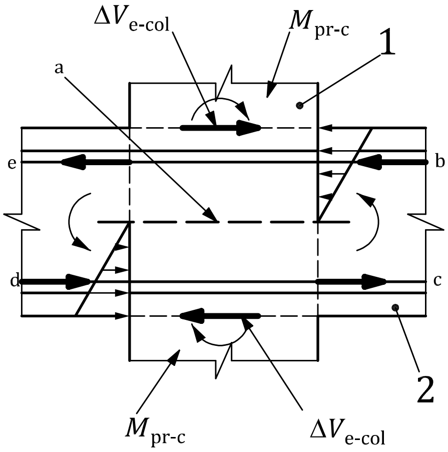

Key

1 column

2 girder

a Plane where shear V_u is evaluated.

b $C_c = T_s = 1{,}25 f_y \cdot A_s$

c $T_s = 1{,}25 f_y \cdot A_s$

d $C'_c = T_s = 1{,}25 f_y \cdot A_s$

e $T'_s = 1{,}25 f_y \cdot A_s$

Figure 107 — Joint shear determination

a) The factored shear at the joint, V_u, should be obtained for both principal directions using Formula (168) for joints where girders frame in both sides and using Formula (169) where girders frame in only one side.

$$V_u = 1{,}25\, f_y \cdot \left(A_s + A'_s\right)_{\text{girder}} - \left(\Delta V_e\right)_{\text{column}} \tag{168}$$

$$V_u \geq \begin{cases} 1{,}25\, f_y \cdot \left(A_s\right)_{\text{girder}} - \left(\Delta V_e\right)_{\text{column}} \\ 1{,}25\, f_y \cdot \left(A'_s\right)_{\text{girder}} - \left(\Delta V_e\right)_{\text{column}} \end{cases} \tag{169}$$

In Formula (168) and Formula (169), the reinforcement area corresponds to the girder longitudinal reinforcement. The shear from development of strength at the column should be determined from 16.5.3.6.

b) The nominal shear strength at the critical plane in the joint should be as follows (for A_j, see Figure 108 and Figure 109):

For joints confined on all four faces $\phi \cdot V_n = \phi \cdot 1{,}70 \cdot \sqrt{f'_c} \cdot A_j$

For joints confined on three faces or on opposite faces $\phi \cdot V_n = \phi \cdot 1{,}25 \cdot \sqrt{f'_c} \cdot A_j$

For other joints $\phi \cdot V_n = \phi \cdot 1{,}00 \cdot \sqrt{f'_c} \cdot A_j$

c) A member that frames into a face of a joint is considered to provide confinement to the joint if at least 3/4 of the face of the joint is covered by the framing member.

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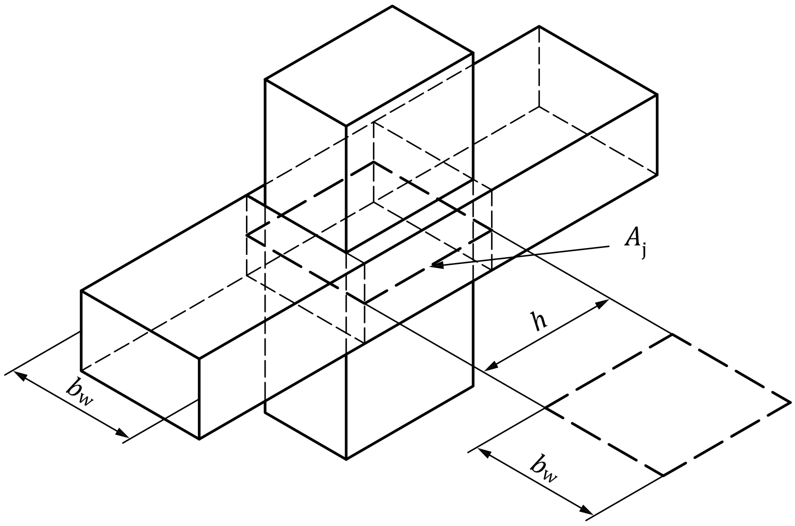

Figure 108 — Definition of A_j for girder wider than column

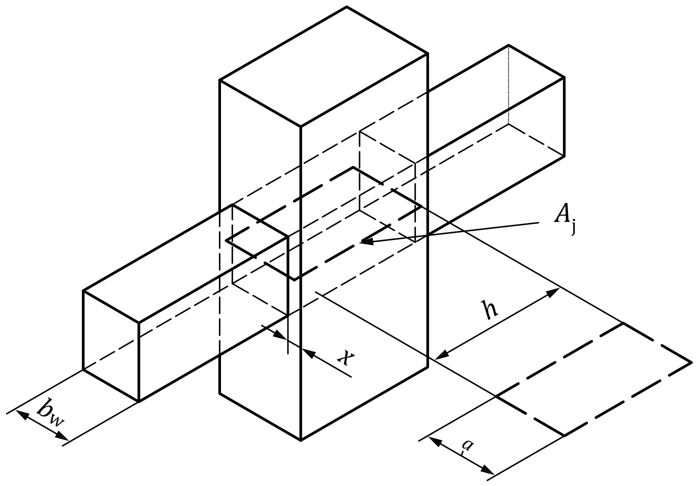

Figure 109 — Definition of A_j for girder narrower than column

A_j corresponds to the effective cross-sectional area within the joint in a plane parallel to the plane of the reinforcement generating the shear and is equal to the product of joint depth by the effective width of the joint. The joint depth corresponds to the dimension of the column parallel to the direction of the girders. The joint effective width is equal to the girder width for girders larger or equal to the column width. See Figure 108. For girders narrower than the column width, the joint effective width is equal to the smaller of the girder width plus the joint depth or the girder width plus twice the smaller perpendicular distance from the longitudinal axis of the girder to the column side, without exceeding the column width. See Figure 109.

16.5.4.1 Anchorage of girder reinforcement at the joint

Longitudinal reinforcement terminating at the joint should end with a 90° standard hook located within the confined core of the column. The anchorage distance should comply with 9.4.3.

16.5.4.2 Girder longitudinal reinforcement straight bars

Straight longitudinal girder reinforcing bars should comply with the development length of 9.4.1.

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16.5.5 Walls

16.5.5.1 General

Structural concrete walls located in seismic zones should comply with the guides of Clause 14, plus the additional guides of 16.5.5.

16.5.5.2 Boundary elements

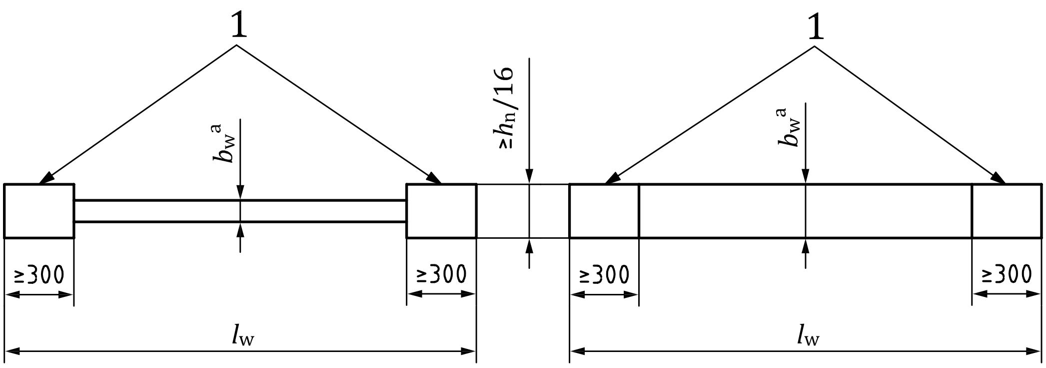
Boundary elements in structural walls should be governed by the following guides.

a) Boundary element should be provided at both edges of structural walls, when maximum factored compressive extreme fibre stress, f_{cu}, evaluated employing Formula (170), corresponding to factored forces, P_u and M_u, from the load combinations that include earthquake effects, exceeds $\left(0{,}2 \cdot f_c'\right)$ unless the entire wall is reinforced to comply with the guides of confinement transverse reinforcement for columns of 16.5.3.4.

$$f_{cu} = \frac{P_u}{A_g} + \frac{6 \cdot M_u}{l_w^2 \cdot b_w} \tag{170}$$

b) The boundary elements should be permitted to be discontinued where the calculated factored compressive stress, f_{cu}, is less than $\left(0{,}15 \cdot f_c'\right)$.

c) The thickness of the boundary elements, where required, should not be less than $h_n/16$ nor b_w, and should have a length not less than 300 mm at each edge. See 14.3.2.1 and Figure 110.



Key

1 boundary elements

a $b_w \geq \begin{cases} 150 \text{ mm} \\ h_n/20 \\ l_w/25 \end{cases}$

Figure 110 — Dimensions of boundary elements

d) Boundary elements, where required, should have transverse reinforcement as specified for columns in 16.5.3.4.

e) Boundary elements should be proportioned to resist all factored gravity loads on the wall, including tributary loads and selfweight, as well as the vertical force required to resist overturning moment calculated from factored related earthquake effect following the guides of 14.2.3. The factored axial load in compression on the boundary elements, P_{cu}, should be obtained from Formula (171) and the factored axial load in tension, P_{tu}, on the boundary elements from Formula (172).

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$$P_{cu} = \frac{P_u}{2} + \frac{M_u}{\left(l_w - 300\text{ mm}\right)} \quad (171)$$

$$P_{tu} = \frac{P_u}{A_g} - \frac{M_u}{\left(l_w - 300\text{ mm}\right)} \leq 0 \quad (172)$$

f) The longitudinal reinforcement of the boundary elements should be proportioned for the factored compression axial load, P_{cu}, employing Formula (109) and Formula (110). In Formula (109), A_g should be substituted by the area of the boundary element. The reinforcement ratio of the boundary element should not exceed the amount prescribed by 14.4.3.2. If the longitudinal reinforcement obtained from Formula (109) and Formula (110) exceeds these limits, the size of the boundary element should be increased until the limits are met. If the size of the boundary elements is increased, the value of P_{cu} and P_{tu} should be corrected, changing the 300 mm of Formula (171) and Formula (172) for the new value.

g) The longitudinal reinforcement of the boundary element should be verified for the absolute value of the factored tension axial load, P_{tu}, if it is present, employing Formula (120).

h) Where boundary elements are columns part of frames; in addition, they should be verified as columns, employing the guides of Clause 16.

16.5.5.3 Shear strength

Shear strength of structural walls should comply with 9.6.5.

17 Nonstructural walls

The walls on the floors that are not part of the main load bearing structural system are called nonstructural walls. Those elements depend on their own structural characteristics to resist lateral forces.

Regardless, the values of A_a and the soil profile, the value of site coefficient F_a will be equal to 1,2.

The value of the ordinate of the elastic design response spectrum is as follows:

$$S_a = 2{,}5A_aF_a = 3A_a$$

The nonstructural walls shall be dimensioned to resist an horizontal uniformly distributed force equal to the following:

$$F_s = \frac{a_m \cdot a_x \cdot m_w}{R_m}$$

$$a_m = \begin{Bmatrix} 1{,}0 \text{ for walls ancored at top and bottom} \\ 2{,}5 \text{ for cantiliver walls} \end{Bmatrix}$$

$$a_x = \begin{Bmatrix} h_n < 0{,}75\ H \quad A_a + \dfrac{8A_a h_n}{3H} \\ h_n < 0{,}75\ H \quad \dfrac{4A_a h_n}{3H} \end{Bmatrix}$$

$$R_m = \begin{Bmatrix} 1{,}0 \text{ for antiliver walls and unreinforced masonry walls} \\ 2{,}5 \text{ for reinforced masonry walls ancored at top and bottom} \end{Bmatrix}$$

The non-structural walls shall be attached to the structure in accordance with the following requirements:

a) nonstructural walls may not be built with reinforced concrete;

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b) nonstructural walls made using masonry shall be separated from all surrounding structural elements by the following:

   1) vertical space of minimum 0,012 h_n;

   2) horizontal space of minimum l_b/150.

c) nonstructural walls made using masonry shall be anchored and reinforced to prevent toppling during earthquakes.

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Annex A
(informative)

Equivalent formulae for material factors

In the limit state design procedure, structural safety is achieved, in part, by the use of factors to magnify the loads and, simultaneously, factors to reduce the materials strength. In many countries, the set of reducing factors depends on the type of stress being considered in the design, regardless of the material used to build the structural element, while in others, these factors vary according to the type of material used. The latter are known as the *material factors*, while the former are known as the ϕ *factors* and are used in the body of these guidelines.

This annex includes the equivalent equations needed when material factors are to be used in place of the ϕ factors. In such a case, ultimate resistant force is not obtained by reducing a nominal force with a factor, but rather the ultimate resistant force is obtained by reducing the specified yield strength for steel or reducing the specificed compressive strength for concrete, or both, by means of dividing these values by the corresponding material factors. Thus, the reduced strength values are as given in Formula (A.1) and Formula (A.2):

$$f_{\mathrm{yd}} = \frac{f_{\mathrm{y}}}{\gamma_{\mathrm{ms}}} \tag{A.1}$$

$$f_{\mathrm{cd}} = \frac{f'_{\mathrm{c}}}{\gamma_{\mathrm{mc}}} \tag{A.2}$$

The material factor, γ_{mc}, vary according to the material used as follows.

MATERIAL	γ_{mc}	γ_{ms}
Cast in place concrete	[1,5]	[1,15]
Standard control not available	[1,7]	[1,25]

The resistant force is then identified by the subindex *r* and no reference to ***nominal*** forces is needed.

Each formula in terms of ϕ factors is tabulated together with its corresponding formula in terms of material factor. Although the results using either formula, in each case, are different, the material factor equations always result in safe values, as compared to the ϕ factors formula.

Formula	In terms of ϕ factors	In terms of material factors
(2)	$\phi \cdot R_{\mathrm{n}} \geq \gamma_1 \cdot S_1 + \gamma_2 \cdot S_2 + \cdots$	$R = f\left(\frac{f_{\mathrm{cd}}}{\gamma_{\mathrm{mc}}}, \frac{f_{\mathrm{yd}}}{\gamma_{\mathrm{ms}}}\right) \geq \gamma_1 \cdot S_1 + \gamma_2 \cdot S_2 + \cdots$
(4)	$\phi \cdot$ (nominal strength) $\geq U$	$R = f\left(\frac{f_{\mathrm{cd}}}{\gamma_{\mathrm{mc}}}, \frac{f_{\mathrm{yd}}}{\gamma_{\mathrm{ms}}}\right) \geq U$
(31)	$\phi \cdot M_{\mathrm{n}} = \phi \cdot A_{\mathrm{s}} \cdot f_{\mathrm{y}} \left(d - \frac{a}{2}\right)$	$M_{\mathrm{r}} = A_{\mathrm{s}} \cdot f_{\mathrm{yd}} \left(d - \frac{a}{2}\right)$

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(32)	$a = \dfrac{A_s \cdot f_y}{0,85 \cdot f'_c \cdot b}$	$a = \dfrac{A_s \cdot f_{yd}}{0,85 \cdot f_{cd} \cdot b}$
(33)	$\phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot d \cdot \left(1 - 0,59 \cdot \dfrac{A_s \cdot f_y}{b \cdot d \cdot f'_c}\right)$	$M_r = A_s \cdot f_{yd} \cdot d \left(1 - 0,59 \cdot \dfrac{A_s \cdot f_{yd}}{b \cdot d \cdot f_{cd}}\right)$
(34)	$\phi \cdot M_n \approx \phi \cdot A_s \cdot f_y \cdot 0,85 \cdot d$	$M_r \approx A_s \cdot f_{yd} \cdot 0,85 \cdot d$
(35)	$\rho = \dfrac{A_s}{b \cdot d} = \alpha - \sqrt{\alpha^2 - \left(\dfrac{M_u}{\phi \cdot b \cdot d^2} \cdot \dfrac{2 \cdot \alpha}{f_y}\right)}$ where $\alpha = \dfrac{f'_c}{1,18 \cdot f_y}$	$\rho = \dfrac{A_s}{b \cdot d} = \alpha - \sqrt{\alpha^2 - \left(\dfrac{M_u}{b \cdot d^2} \cdot \dfrac{2 \cdot \alpha}{f_{yd}}\right)}$ where $\alpha = \dfrac{f_{cd}}{1,18 \cdot f_{yd}}$
(36)	$\rho = \dfrac{A_s}{b \cdot d} \approx \dfrac{M_u}{\phi \cdot b \cdot d^2 \cdot 0,85 \cdot f_y}$	$\rho = \dfrac{A_s}{b \cdot d} \approx \dfrac{M_u}{b \cdot d^2 \cdot 0,85 \cdot f_{yd}}$
(37)	$\phi \cdot M_n = \phi \cdot \left[\begin{matrix}\left(A_s - A'_s\right) \cdot f_y \left(d - \dfrac{a}{2}\right) + \\ A'_s \cdot f_y \cdot \left(d - d'\right)\end{matrix}\right]$	$M_r = \left(A_s - A'_s\right) \cdot f_{yd} \left(d - \dfrac{a}{2}\right) +$ $A'_s \cdot f_{yd} \cdot \left(d - d'\right)$
(38)	$a = \dfrac{\left(A_s - A'_s\right) \cdot f_y}{0,85 \cdot f'_c \cdot b}$	$a = \dfrac{\left(A_s - A'_s\right) \cdot f_{yd}}{0,85 \cdot f_{cd} \cdot b}$
(39)	$A'_s = \dfrac{M_u}{\phi \cdot f_y \cdot \left(d - d'\right)} - \left[b \cdot d^2 \cdot \rho_{max} \cdot f_y \cdot 0,8\right]$	$A'_s = \dfrac{M_u}{f_{yd} \cdot \left(d - d'\right)} - \left[b \cdot d^2 \cdot \rho_{max} \cdot f_{yd} \cdot 0,8\right]$
(41)	$h_f \geq a$ and $a = \dfrac{A_s \cdot f_y}{0,85 \cdot f'_c \cdot b}$	$h_f \geq a$ and $a = \dfrac{A_s \cdot f_{yd}}{0,85 \cdot f_{cd} \cdot b}$
(42)	$\rho \leq \dfrac{0,85 \cdot f'_c \cdot h_f}{f_y \cdot d}$	$\rho \leq \dfrac{0,85 \cdot f_{cd} \cdot h_f}{f_{yd} \cdot d}$
(43)	$\phi \cdot V_n \geq V_u$	$V_r \geq V_u$
(44)	$\phi \cdot V_n = \phi \cdot \left(V_c + V_s\right)$	$V_r = V_c + V_s$
(45)	$\phi \cdot V_c = \phi \cdot 2 \cdot \left[\dfrac{\sqrt{f'_c}}{6}\right] \cdot b_w \cdot d$	$V_c = 0,60\, f_{ctd}\, b_w\, d$ where $f_{ctd} = \left(0,35\sqrt{f'_c}\right) \Big/ \gamma_{mc}$

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(46)	$\phi \cdot V_s = \phi \cdot \left[\dfrac{A_v \cdot f_{ys} \cdot d}{s}\right]$		$V_s = \dfrac{A_v \times f_{ys} \times d}{1,15 \cdot s}$	
(47)	$\phi \cdot V_s \leq \phi \cdot \left[\dfrac{2}{3} \cdot \sqrt{f_c'} \cdot b_w \cdot d\right] = 4 \cdot \phi \cdot V_c$		$V_s \leq 0,20\, f_{cd}\, b_w\, d$	
(48)	$A_v = \dfrac{1}{16}\sqrt{f_c'}\,\dfrac{b_w \cdot s}{f_{ys}} \geq \dfrac{b_w \cdot s}{3 \cdot f_{ys}}$		$A_v = 0,30\dfrac{f_{ctd}}{f_{yds}}\,b_w\,s$ where $f_{ctd} = \left(0,35\sqrt{f_c'}\right)\Big/\gamma_{mc}$ $f_{ctd} = \left(0,35\sqrt{f_{ck}'}\right)\Big/\gamma_{mc}$	
Table 8 $V_c > V_u \geq \dfrac{V_c}{2}$ $\left[\left(\phi V_c\right) > V_u \geq \dfrac{\left(\phi V_c\right)}{2}\right]$		$A_v = \dfrac{1}{16}\sqrt{f_c'}\,\dfrac{b_w \cdot s}{f_{ys}} \geq \dfrac{b_w \cdot s}{3 \cdot f_{ys}}$	$A_v = 0,30\dfrac{f_{ctd}}{f_{yds}}\,b_w\,s$ where $f_{ctd} = \left(0,35\sqrt{f_c'}\right)\Big/\gamma_{mc}$	
Table 8 $V_u \geq V_c$ $[V_u \geq (\phi V_c)]$	$2 \cdot \phi \cdot V_c > \phi \cdot V_s$	$A_v = \dfrac{\left(V_u - \phi \cdot V_c\right) \cdot s}{\phi \cdot f_{ys} \cdot d}$	$2 \cdot V_c > V_s$	$A_v = \dfrac{1,15\left(V_u - V_c\right) \cdot s}{\phi \cdot f_{ysd} \cdot d}$
	$4 \cdot \phi \cdot V_c > \phi \cdot V_s \geq 2 \cdot \phi \cdot V_c$	$A_v = \dfrac{\left(V_u - \phi \cdot V_c\right) \cdot s}{\phi \cdot f_{ys} \cdot d}$	$4 \cdot V_c > V_s \geq 2 \cdot V_c$	$A_v = \dfrac{1,15\left(V_u - V_c\right) \cdot s}{\phi \cdot f_{ysd} \cdot d}$
	$\phi \cdot V_s \geq 4 \cdot \phi \cdot V_c$	not permitted	$V_s \geq 4 \cdot V_c$	not permitted
(49)	$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[1 + \dfrac{2}{\beta_c}\right] \cdot \left[\dfrac{\sqrt{f_c'}}{6}\right] \cdot b_0 \cdot d$		$V_r = 0,60\left[1 + \dfrac{2}{\beta_c}\right] f_{ctd}\, b_0\, d$ where $f_{ctd} = \left(0,35\sqrt{f_c'}\right)\Big/\gamma_{mc}$	
(50)	$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[2 + \dfrac{\alpha_s \cdot d}{b_0}\right] \cdot \left[\dfrac{\sqrt{f_c'}}{12}\right] \cdot b_0 \cdot d$		$V_r = 0,3\left(2 + \dfrac{\alpha_s\, d}{b_0}\right) f_{ctd}\, b_0\, d$ where $f_{ctd} = \left(0,35\sqrt{f_c'}\right)\Big/\gamma_{mc}$	

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(51)	$\phi \cdot V_{n} = \phi \cdot V_{c} = \phi \cdot \left[\dfrac{\sqrt{f_{c}'}}{3} \right] \cdot b_{0} \cdot d$	$V_{r} = 1,20 f_{ctd} b_{0} d$ where $f_{ctd} = \left(0,35 \sqrt{f_{c}'} \right) / \gamma_{mc}$
(74)	$d \geq \dfrac{3 \cdot q_{u} \cdot \alpha_{a} \cdot l_{a}}{\phi \cdot \sqrt{f_{c}'}}$	$d \geq 0,8 \dfrac{q_{u} \alpha_{a} \ell_{a}}{f_{ctd}}$ where $f_{ctd} = \left(0,35 \sqrt{f_{c}'} \right) / \gamma_{mc}$
(75)	$d \geq \dfrac{3 \cdot q_{u} \cdot \alpha_{b} \cdot l_{b}}{\phi \cdot \sqrt{f_{c}'}}$	$d \geq 0,8 \dfrac{q_{u} \alpha_{b} \ell_{b}}{f_{ctd}}$ where $f_{ctd} = \left(0,35 \sqrt{f_{c}'} \right) / \gamma_{mc}$
(76)	$d \geq \dfrac{3 \cdot q_{u} \cdot l_{a}}{2 \cdot \phi \cdot \sqrt{f_{c}'}}$	$d \geq 0,4 \dfrac{q_{u} l_{a}}{f_{ctd}}$ where $f_{ctd} = \left(0,35 \sqrt{f_{c}'} \right) / \gamma_{mc}$
12.3.5.5.4 a)	$\phi \cdot \dfrac{\sqrt{f_{c}'}}{4} \cdot b_{w} \cdot d$	$\left(0,90 f_{ctd} b_{w} d \right)$ where $f_{ctd} = \left(0,35 \sqrt{f_{c}'} \right) / \gamma_{mc}$
(96)	$A_{i} \geq \dfrac{\left(1 - h_{b} / h_{s} \right) V_{u}}{\phi \cdot f_{y}}$	$A_{i} \geq \dfrac{\left(1 - h_{b} / h_{f} \right) V_{u}}{f_{yd}}$
(108)	$\sqrt{\left[\dfrac{\left(V_{u} \right)_{x}}{\left(\phi \cdot V_{n} \right)_{x}} \right]^{2} + \left[\dfrac{\left(V_{u} \right)_{y}}{\left(\phi \cdot V_{n} \right)_{y}} \right]^{2}} \leq 1,0$	$\sqrt{\left(\dfrac{V_{ux}}{V_{rx}} \right)^{2} + \left(\dfrac{V_{uy}}{V_{uy}} \right)^{2}} \leq 1,0$
(109)	$\phi \cdot P_{0n} = \phi \cdot \left[0,85 \cdot f_{c}' \cdot \left(A_{g} - A_{st} \right) + A_{st} \cdot f_{y} \right]$	$P_{r0} = 0,85 \cdot f_{cd} \cdot \left(A_{g} - A_{st} \right) + A_{st} \cdot f_{yd}$
(110)	$\phi \cdot P_{n(max)} \leq 0,80 \cdot \phi \cdot P_{0n}$	$P_{r(max)} \leq 0,80 \cdot P_{r0}$
(111)	$\phi \cdot P_{n(max)} \leq 0,85 \cdot \phi \cdot P_{0n}$	$P_{r(max)} \leq 0,85 \cdot P_{r0}$
(112)	$\phi \cdot P_{bn} = \phi \cdot 0,40 \cdot f_{c}' \cdot h \cdot b$	$P_{br} = 0,42 \cdot f_{cd} \cdot h \cdot b$
(113)	$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0,25 \cdot h + \phi \cdot$ $\left(0,95 A_{se} + 0,16 A_{si} \right) \cdot f_{y} \cdot \left(\dfrac{h}{2} - d' \right)$	$M_{br} = P_{bn} \cdot 0,25 \cdot h + (0,95 A_{se} + 0,16 A_{si}) \cdot$ $f_{y} \cdot \left(\dfrac{h}{2} - d' \right)$
(114)	$\phi \cdot P_{bn} = \phi \cdot 0,4 \cdot f_{c}' \cdot A_{c}$	$P_{br} = 0,4 \cdot f_{cd} \cdot A_{c}$

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(115)	$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0,2 \cdot h + \phi \cdot 0,5 \cdot A_{st} \cdot f_y \cdot \left(\frac{h}{2} - d'\right)$	$M_{br} = P_{br} \cdot 0,2 \cdot h + 0,48 \cdot A_{st} \cdot f_{yd} \cdot \left(\frac{h}{2} - d'\right)$
(120)	$\phi \cdot P_{tn} = \phi \cdot A_{st} \cdot f_y$	$P_{tr} = A_{st} \cdot f_{yd}$
(121)	$\phi \cdot M_n \geq M_u$	$M_r \geq M_u$
(122)	$P_u \leq \phi \cdot P_{n(max)}$	$P_u \leq P_{r(max)}$
(123)	$P_u \geq -\left(\phi \cdot P_{tn}\right)$	$P_u \geq -\left(P_{tr}\right)$
(124)	$M_u \leq \phi \cdot M_n = \frac{\left(\phi \cdot P_{0n}\right) - P_u}{\left(\phi \cdot P_{0n}\right) - \left(\phi \cdot P_{bn}\right)} \cdot \left(\phi \cdot M_{bn}\right)$	$M_u \leq M_n = \frac{\left(P_{r0}\right) - P_u}{\left(P_{r0}\right) - \left(P_{br}\right)} \cdot \left(M_{br}\right)$
(126)	$M_u \leq \phi \cdot M_n = \frac{P_u + \left(\phi \cdot P_{tn}\right)}{\left(\phi \cdot P_{tcn}\right) + \left(\phi \cdot P_{tn}\right)} \cdot \left(\phi \cdot M_{tcn}\right)$	$M_u \leq M_n = \frac{P_u + \left(P_{tr}\right)}{\left(P_{br}\right) + \left(P_{tr}\right)} \cdot \left(M_{br}\right)$
(127)	$\frac{\left(M_u\right)_x}{\left(\phi \cdot M_n\right)_x} + \frac{\left(M_u\right)_y}{\left(\phi \cdot M_n\right)_y} \leq 1,0$	$\frac{\left(M_u\right)_x}{\left(M_r\right)_x} + \frac{\left(M_u\right)_y}{\left(M_r\right)_y} \leq 1,0$
(160)	$\sum\left(l_w \cdot b_w\right) \geq \frac{V_u}{\frac{1}{9} \cdot \sqrt{f_c'}}$	$\sum\left(l_w b_w\right) \geq \frac{V_u}{0,48 f_{ctd}}$ where $f_{ctd} = \left(0,35\sqrt{f_c'}\right) / \gamma_{mc}$
(165)	$s \leq \frac{A_b \cdot f_{ys}}{f_c' \cdot 15\ \text{mm}}$	$s \leq \frac{A_b f_{yds}}{f_{cd} \times 20\ \text{mm}} \leq 100\ \text{mm}$
(166)	$\rho_s = \frac{A_b \cdot \pi \cdot d_c}{A_c \cdot s} \geq 0,12 \cdot \frac{f_c'}{f_{ys}}$	$\rho_s = \frac{A_b\ \pi\ d_c}{A_c\ s} \geq 0,16 \frac{f_{cd}}{f_{yds}}$
16.5.4.4 all four faces three or opposite faces other joints	$\phi \cdot V_n = \phi \cdot 1,70 \cdot \sqrt{f_c'} \cdot A_j$ $\phi \cdot V_n = \phi \cdot 1,25 \cdot \sqrt{f_c'} \cdot A_j$ $\phi \cdot V_n = \phi \cdot 1,00 \cdot \sqrt{f_c'} \cdot A_j$	$V_r = 6,35\ f_{ctd}\ A_j$ $V_r = 4,65\ f_{ctd}\ A_j$ $V_r = 3,72\ f_{ctd}\ A_j$ where $f_{ctd} = \left(0,35\sqrt{f_c'}\right) / \gamma_{mc}$

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ICS 91.080.40
Price based on 182 pages

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