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Guidelines for simplified seismic assessment and rehabilitation of concrete buildings

Lignes directrices pour l'évaluation sismique simplifiée et la réhabilitation des structures en béton

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Foreword

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

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

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

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

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

Introduction

The aim of this International Standard is to provide rules for the earthquake resistant assessment and rehabilitation design and execution for existing structural concrete buildings for which simplified procedures may be applied instead of more sophisticated and thorough analyses, in light of the simplicity, symmetry, and other characteristics of the structure under study. This International Standard is developed for countries that do not have existing national standards on this subject and to offer, to local regulatory authorities anywhere, an alternative for the study of relatively small and simple buildings that abound in both rural and urban environments. The analysis and design rules are based in simplified worldwide-accepted strength models. This International Standard is self-contained; therefore actions (loads), simplified analysis procedures and design specifications are included, as well as minimum acceptable construction practice guidelines.

The minimum dimensional guidelines contained in this International Standard are intended to account for undesirable side effects that will otherwise 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 for rehabilitation of concrete buildings in the numerous regions of the world which lie in earthquake prone areas. The earthquake resistance of rehabilitated buildings 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 International Standard contains guidelines that can be modified by the national standards body due to local design and construction requirements and practices. These guidelines that can be modified are included using ["boxed values"]. The authorities in each member country are expected to review the "boxed values" and may substitute alternative definitive values for these elements for use in the national application of this International Standard. Changes to boxed values shall not be made without thorough analyses and sound supporting studies.

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

Guidelines for simplified seismic assessment and rehabilitation of concrete buildings

1 Scope

This International Standard can be used as an alternative to the development of a building code, or equivalent document in countries where no national design codes are available by themselves, or as an alternative to the building code in countries where specifically considered and accepted by the national standards body or other appropriate regulatory organization, and applies to the assessment of earthquake resistance capability and to the seismic rehabilitation design and construction for existing structural concrete buildings.

The purpose of these guidelines is to provide sufficient information to perform the seismic assessment and rehabilitation of the structural concrete building that complies with the limitations established in Clause 5, for both undamaged structures that are deemed not to comply with the required characteristics for an adequate response at a specified performance level, and for structures that have undergone damage under seismic loadings. The rules of design as set forth in this International Standard are simplifications of more elaborate requirements.

Although the guidelines contained in this International Standard were drawn to produce, when properly employed, a reasonable assessment of the seismic vulnerability of an undamaged structure, a reasonable assessment of a structure damaged by a seismic event and a structural rehabilitation of the assessed concrete structure with an appropriate margin of safety, these guidelines are not a replacement for sound and experienced engineering. In order to attain the intended results on assessment and rehabilitation design, this International Standard must be used as a whole, and alternative procedures should be employed only when explicitly permitted by the guidelines. The minimum dimensioning guides as prescribed in this International Standard replace, in most cases, more elaborate procedures such as those prescribed in the national code or, if no national code exists, in internationally recognized full fledged codes, and the possible economic impact is compensated for by the simplicity of the procedures prescribed here.

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

While buildings rehabilitated in accordance with these guidelines are expected to perform within the selected performance levels for the applicable design earthquakes, compliance with these guidelines is necessary but may not guarantee the sought for performance, as current knowledge of structural behavior under seismic loads, and of the loads themselves, is still incomplete.

2 Normative references

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

ISO 15673:2005, *Guidelines for the simplified design of structural reinforced concrete for buildings*

3 Terms and definitions

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

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

NOTE For the application of these guidelines its value can be approximated to 10 m/s^2.

3.2
adherence
force acting on the interface of two solid materials

3.3
admixture
material other than water, aggregate, or hydraulic cement, added to concrete before or during its mixing to modify its properties

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

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

3.6
bar diameter, nominal
approximate diameter of a steel reinforcing bar, often used as a class designation

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

3.7
beam
structural member for which ratio of axial load to axial gross capacity is equal to or less than 0,1.

3.8
bearing capacity of the soil
maximum permissible stress on the foundation soil that provides adequate safety against bearing failure of the soil

NOTE Its value is defined at the working stress level.

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

3.10
boundary elements
structural elements embedded at the ends of structural walls strengthened by transverse reinforcement to confine the longitudinal reinforcement

NOTE Boundary elements may require an increase in thickness of the wall.

3.11
caisson
foundation pile of large diameter, built partly or totally above ground and sunk below ground usually by digging out the soil inside

3.12
carbonation
process of conversion of calcium hydroxide in hardened cementitious material into calcium carbonate due to reaction with carbon dioxide diffused into the cement paste from the atmosphere

3.13
cement
material as specified in the corresponding referenced ISO standards, which, when mixed with water, has hardening properties

3.14
center of mass
geometric plan location of the resultant force due to the action of gravity on the mass of the floor is located, supposing the floor diaphragm as an infinite rigid body in its own plane

3.15
center of rigidity
geometric plan location of the resultant of the resistance forces due to structural vertical elements stiffness, calculated, supposing that the floor diaphragm is an infinite rigid body in its own plane in such a way that when applying a horizontal force in any direction, rotation of the diaphragm takes place with no distortion of the original shape of the floor

3.16
corrosion
process of disintegration of the reinforcing steel bars due to chemical or electromechanical change caused in presence of moisture

3.17
column
structural member in which the ratio of axial compressive loads to axial gross capacity is more than 0,1

3.18
collector elements
structural elements that carry the forces within a horizontal diaphragm to the lateral-force resisting system

3.19
combined footing
footing that transmits to the supporting soil the load carried by several columns or structural concrete walls

3.20
compression reinforcement
reinforcement provided to resist compression stresses in the member section

3.21
concrete
mixture of cementitious materials with fine aggregate, coarse aggregate, and water, with or without admixtures, to form a hardened material with specific strength properties

3.22
concrete mix design
choice and proportioning of the ingredients of concrete

3.23
concrete specified compressive strength, f_c'
compressive strength of cylindrical concrete specimens used in design and evaluated in accordance with the appropriate ISO standard, expressed in megapascals (MPa)

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

3.24
confinement hook
hook at the ends of a stirrup, hoop, or crosstie having a bend of not less than 135° with a six-diameter (but not less than 75 mm) extension that engages the longitudinal reinforcement and projects into the interior of the stirrup, hoop or crosstie

3.25
confinement stirrup or tie
closed stirrup, tie or continuously wound spiral

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

3.26
cover, concrete
thickness of concrete between the surface of any reinforcing bar and the nearest face of the concrete member

3.27
crack
break, with or without quite separating in two parts, of concrete, usually near or at the surface

3.28
creep
unrecoverable strain caused to a material subjected to constant stress for a long duration

3.29
crosstie
continuous reinforcing bar having a 135° hook at one end and a hook not less than 90° at least a six-diameter extension at the other end

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

3.30
curing
process in which concrete is kept damp for a period of several days, starting from the moment it is cast, in order to prevent evaporation of water within the cementitious paste to ensure that the hardening process attains the intended strength

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

3.31
dead load
permanent load
load in which variations over time are rare or of small magnitude

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

3.32
deformed reinforcement
steel reinforcement that has deformations in its surface to increase its bond to the concrete

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

3.33
depth of member, h
vertical dimension of a cross section of a horizontal structural element or cross section dimension parallel to the direction of transversally applied loads to vertical structural elements

3.34
design load combinations
combinations of factored loads and forces as specified in these guidelines

3.35
design strength
product of the nominal strength and a strength reduction factor ϕ

3.36
development length
length of embedded reinforcement required to develop the design strength of reinforcement at a critical section

3.37
development length for a bar with a standard hook
minimum length to be provided 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

3.38
differential settlement
non-uniform vertical displacement of the foundation

3.39
drift
difference between the horizontal displacements of two floor levels

3.40
durability
characteristics of a structure to resist gradual degradation of its serviceability in a given environment for the design service life

3.41
effective depth of section, d
distance measured from extreme compression fiber to centroid of tension reinforcement

3.42
embedment length
length of embedded reinforcement provided beyond a critical section

3.43
fatigue
weakening of a material by load cycles, with or without load reversals

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

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

NOTE The concrete cover is a function of specified fire resistance, measured in hours.

3.46
flange
top or bottom part of an "I" or "T" shaped section

3.47
flexural
pertaining to the effect of flexure

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

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

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

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

3.51
foundation
part of the structure that transmits loads to the underlying soil

NOTE A retaining wall is a special kind of foundation that resists the pressure exerted by the soil on sloping or vertical excavations

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

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

3.54
frame system
system in which seismic shear forces are resisted by shear and flexure in members and joints of a frame

3.55
girder
main horizontal support beam, usually supporting other beams

3.56
gravity loads
loads that act downward and are caused by the acceleration of gravity, g, acting on the mass of the elements

3.57
hook
bend at the end of a reinforcing bar

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

3.58
joist
beam used in parallel series directly supporting floor and ceiling loads, and supported in turn by larger girders, beams, or bearing structural concrete walls

3.59
lap splice
splice between two reinforcing bars obtained by overlapping them through a specified length

3.60
lateral-force resisting system
portion of the structure composed of members proportioned to resist forces related to earthquake or wind effects

3.61
lightweight aggregate concrete
concrete made with coarse granular material that weighs less than the granular material used in normal weight concrete

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

3.62
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 loads
all forces that are variable within the structure's normal operation, not including construction or environmental loads

3.64
load effects
forces and deformations produced in structural members by the applied loads

3.65
load factor
factor that accounts for deviations of the actual load from the nominal load due to uncertainties both in load calculation and load effect analysis, and to account for the probability that more than one extreme load will occur simultaneously

3.66
loads
forces or other actions that result from the weight of all building materials, pedestrians, environmental effects, differential movement, and restrained dimensional changes

3.67
longitudinal reinforcement
reinforcement 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
modulus of elasticity
ratio of normal stress to corresponding strain for tensile or compressive stresses below proportional limit of material

3.70
negative moment
flexural moment that produces tension stresses at the upper part of the section of a horizontal or nearly horizontal element, and that requires placing negative flexural reinforcement in the upper part of the element section

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

3.72
nominal loads
magnitudes of the loads specified in these guidelines (dead, live, soil, wind, snow, rain, flood, and earthquake), before applying magnifying or reducing factors

3.73
nominal strength
capacity of a structure or member to resist the effects of loads, as determined by computations using specified material strengths and dimensions and the formulas set forth by these guidelines

NOTE Specified material strengths are derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modelling effects and differences between laboratory and field conditions

3.74
overturning
action resulting when the moment produced at the base of vertical lateral force resisting elements is larger than the resistance provided by the building weight and foundation resistance to uplift

3.75
pedestal
upright compression member with a ratio of unsupported height to average least lateral dimension of less than 3

3.76
performance level
limiting damage state for a building, considering structural and nonstructural components

3.77
pile
slender timber, concrete or structural steel element embedded in the ground to support loads

3.78
plain reinforcement
smooth surfaced steel reinforcement, or reinforcement that does not conform to the definition of deformed reinforcement

3.79
positive moment
flexural moment that produces tension stresses at the lower part of the section of a horizontal, or nearly horizontal element, and that requires placing positive flexural reinforcement in the lower part of the element section

3.80
positive reinforcement
in horizontal or nearly horizontal elements, the flexural reinforcement required for positive moment and that is placed in the lower part of the section of the element

3.81
reaction
resistance to a force or load, or upward resistance of a support such as a structural concrete wall or column against the downward pressure of a loaded member such as a beam

3.82
rehabilitation
process of repairing or reinforcing a structure to achieve a desired useful condition

NOTE Also known as retrofit.

3.83
reinforcement
steel bars, wire, or wire mesh, used for reinforcing the concrete to improve its performance when tensile stresses are expected, due either to the applied loads, or to environmental effects such as variation of temperature

3.84
required factored strength
strength of a member or cross section required to resist factored loads or related internal moments and forces in such combinations as are stipulated by these guidelines

3.85
retaining wall
wall built to support lateral earth or liquid pressure

3.86
seismic hazard
level of seismic actions at the building site quantified in terms of the horizontal effective acceleration of the ground, which has a given probability of being exceeded in a predetermined lapse of time

3.87
selfweight
weight of the structural element, caused by the mass of the materials in elements and structure

3.88
service load
unfactored loads specified by these guidelines

3.89
settlement
downward movement of the supporting soil

3.90
shear force
internal force, resulting from shear stresses acting parallel to the plane of a cross section of an element

3.91
shear reinforcement
reinforcement proportioned to resist shear forces

3.92
shores
temporary structural members designed to support the weight of the formwork, concrete, and construction loads

3.93
shrinkage and temperature reinforcement
reinforcement provided for shrinkage and temperature stresses in structural solid slabs and footings

3.94
skew
part of a structure that deviates from a straight line or from a right angle

3.95
slab
solid flat part of a reinforced concrete deck carried by supporting joists or beams or girders

3.96
solid slab
deck or floor system comprised of a slab of uniform thickness that does not have voids or that has voids with size less than 0,6 times the slab thickness

3.97
spalling
loss of material at the surface of a concrete element due to breakage or splitting off in chips or bits, usually due to expansive force within the mass of concrete

3.98
span length
distance between supports of a horizontal structural element

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 force prescribed by these guidelines, for earthquake-resistant design

3.101
specified wind forces
nominal pressure of wind to be used in design performed in accordance with these guidelines

3.102
spiral reinforcement
continuously wound reinforcement in the form of a cylindrical helix

3.103
spread footing
isolated footing that transmits to the supporting soil the load carried by a single column

3.104
stirrup
reinforcement used to resist shear and torsion stresses in a structural member

NOTE Typically bars, wires, or welded wire fabric (plain or deformed) either single leg or bent into L, U, or rectangular shapes and located perpendicular to or at an angle to longitudinal reinforcement. (The term "stirrups" is usually applied to lateral reinforcement in girders, beams, and joists; the term "ties" to those in columns and walls, perhaps because they are intended also as confinement for the longitudinal reinforcement.).

3.105
stiffness
measure of difficulty to bend or flex a structural element or a structure expressed as the necessary force to cause a unitary deformation

NOTE When applied to qualify a building story, it is expressed as the ratio between the story total shear force and the story drift, under lateral loading.

3.106
strength
maximum axial force, shear force or moment that can be resisted by a component

3.107
strengthening
measures taken for a deteriorated structure or any of its structural members to restore its design load carrying capacity

3.108
strength reduction factor, ϕ
coefficient that accounts for deviations of the actual strength from the nominal strength, according to the manner and consequences of failure

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

3.109
stress
intensity of force per unit area

3.110
structural concrete
all concrete used for structural purposes including reinforced and prestressed concrete

3.111
structural concrete walls
walls proportioned to resist combinations of shear, moments, and axial forces including but not limited to selfweight and forces applied directly to it

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

3.112
structural diaphragms
structural systems, such as floor and roof decks, which transmit forces induced by lateral motions

3.113
support
structural element that provides bearing and movement restriction to other structural or non-structural elements

3.114
tie
loop of reinforcing bar or wire enclosing longitudinal reinforcement

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

3.115
tie elements
elements which serve to transmit inertia forces and prevent separation of components such as footings and walls

3.116
transverse reinforcement
reinforcement located perpendicular to the longitudinal axis of the element, such as stirrups, ties, spiral reinforcement, among others

3.117
vulnerability level
quantification of the potential of a building for inadequate performance under any set of loads including environmental actions

3.118
wall
member, usually vertical and whose length and height are much larger than its thickness

3.119
web
thin vertical portion of an "I" or "T" shaped section that connects the flanges

3.120
weight
vertical downward force exerted by a mass when subjected to the acceleration of gravity

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

3.121
wire
slender stringlike steel filament, usually of circular section, of small diameter

3.122
wire mesh
welded-wire fabric reinforcement

3.123
working stress
allowable stress to be used with unfactored loads

3.124
yield strength, f_y
specified minimum stress at which steel initiates yielding

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

NOTE 2 Applicable international standards specify that the yield strength be determined for steel subjected to tensile forces.

4 Symbols and abbreviated terms

For the purposes of this document, the following symbols and definitions apply[2].

Symbol	Explanation	Unit
A	Depth of equivalent uniform compressive stress block	mm
A_b	Area of an individual reinforcement bar or wire	mm²
A_c	Loaded area of bearing on concrete or the area of the confined column core	mm²
A_g	Gross area of section of element	mm²
A_j	Effective cross-sectional area within a joint for shear evaluation or area of additional hanger reinforcement	mm²
A_s	Area of longitudinal tension reinforcement	mm²
A'_s	Area of longitudinal compression reinforcement	mm²
$A_{s,\ min}$	Minimum area of longitudinal tension reinforcement	mm²
A_{se}	Total extreme steel area in a column or structural concrete wall for computation of the balanced moment strength	mm²
A_{ss}	Total side steel area in a column or structural concrete wall for computation of the balanced moment strength	mm²
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	mm²
a	Depth of equivalent uniform compressive stress block	mm
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	-
B	Width of compression face of member, or width of the section of the member	mm
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
β	Ratio of clear spans in long to short direction of two-way slabs	-
D	Dead loads, or related internal moments and forces	-
d	Effective depth, should be taken as the distance from extreme compression fiber to centroid of tension reinforcement	mm
d'	Distance from extreme compression fiber to centroid of compression reinforcement	mm
d_b	Nominal diameter of reinforcing bar or wire	mm
d_c	Distance from extreme tension fiber to centroid of tension reinforcement or diameter of the confined core of column with spiral reinforcement	mm
d_P	Diameter of pile, at footing base	mm
E	Load effects of earthquake, or related internal moments and forces	-

Symbol	Explanation	Unit
E_c	Modulus of elasticity of concrete	MPa
F	Loads due to weight and pressures of fluids with well-defined densities and controllable maximum heights, or related internal moments and forces	-
F_i, F_x	Design wind or seismic force applied at level i or x, respectively	kN
F_a	Soil coefficient	-
F_{iu}, F_{xu}	Factored design lateral force applied to the wall at level i or x, respectively	N
f'_c	Specified compressive strength of concrete	MPa
$\sqrt{f'_c}$	Positive square root of specified compressive strength of concrete	MPa
f_{cd}	Compressive strength of concrete reduced by the material factor	MPa
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, (f_{ypr} = 1,25 $\cdot f_y$).	MPa
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
γ_{mc}	Material factor for concrete.	-
$\gamma_{\mu\sigma}$	Material factor for steel.	-
ϕ	Strength reduction factor.	-
H	Loads due to the weight and pressure of soil, water in soil, or other materials, or related internal moments and forces.	-
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 deck I, measured from deck finish of the story to deck finish of the story immediately below.	mm
h_s	Total height of the supporting girder.	mm
h_w	Height of entire structural concrete wall from base to top.	mm
l	Span of structural element or length of span measured center-to-center of beams or other supports	-
I_c	Moment of inertia of the column section.	m^4
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

Symbol	Explanation	Unit
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_o	Column confinement length	-
l_w	Horizontal length of structural concrete wall.	mm
L	Live loads, or related internal moments and forces	-
L_r	Sloping roof live load, or related internal moments and forces	-
M_{bn}	Nominal flexural moment strength at section at balanced conditions.	N.mm
M_{br}	Flexural moment strength at section at balanced conditions.	mm
M_{cs}	Flexure strength of beam sections	N.mm
M_{iu}, M_{xu}	Factored story moment caused by lateral loads at story i or x, respectively.	N
M_n	Nominal flexural moment strength at section.	mm
M_r	Flexural moment strength at section.	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
ΣM_c	Sum of lowest flexural strengths (ϕ Mn) of columns framing into a joint.	N.m
ΣM_g	Sum of flexural strengths (ϕ Mn) 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
max.	Maximum	-
min.	Minimum	-
P_{bn}	Nominal compression axial load strength at section at balanced conditions	N
P_{br}	Nominal compression axial load strength at section at balanced conditions.	N
P_{cs}	Design axial strength for axial compression without flexure	N
P_{cu}	Factored compression load on wall boundary element, including earthquake effects	-
P_d	Non factored dead load axial force at section or non factored concentrated dead load applied directly to the element.	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_{tcs}	Design strength for axial tension without flexure	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

Symbol	Explanation	Unit
P_{0n}	Axial compressive strength at section.	N
ΣP_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	Energy dissipation Level	-
R_a	Rain load, or related internal moments and forces	-
R_0	Basic coefficient of energy dissipation capacity	-
r_u	Factored uniformly distributed reaction from the slab on the supporting girder, beam or structural concrete wall.	N/m
R_u	Total factored concentrated reaction from a supported structural element.	N
ΣR_u	Sum of all factored reactions from supported structural elements at the same story.	N
$\rho = \frac{A_s}{b \cdot d}$	Ratio of longitudinal tension reinforcement.	-
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.	-
$\rho = \frac{A_{st}}{b \cdot d}$	Ratio of total longitudinal reinforcement area to gross concrete section area,	-
ρ_v	Ratio of vertical reinforcement in structural concrete walls	-
S	Snow load, or related internal moments and forces	-
s	Center-to-center spacing of transverse reinforcement measured along the axis of the element or spacing between stirrups or vertical spacing between bars of skin reinforcement or spacing of longitudinal or transverse reinforcement or clear distance between webs.	mm
T	Cumulative effect of temperature, creep, shrinkage, or differential settlement, or related internal moments and forces	-
T_u	Factored torsional moment at section.	N.mm
U	Required factored strength to resist factored loads or related internal moments and forces	-
V_c	Contribution of the concrete to the nominal shear strength at section.	N
V_{iu}, V_{xu}	Factored story shear caused by lateral loads at story i or x, respectively.	N
V_n	Nominal shear strength at section.	N
V_s	Contribution of the horizontal reinforcement to the nominal shear strength at section.	N
V_u	Factored shear force at section.	N
ΔV_e	Factored design shear force from the development of the probable flexural capacity of the element at the faces of the joints.	N
W	Wind loads, or related internal moments and forces	-
W_u	Total factored uniformly distributed design load per unit element length.	kN/m

Symbol	Explanation	Unit
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

5 Limitations

These guidelines should be employed only when the building being studied complies with all the limitations set forth in 5.1 to 5.10.

5.1 Occupancy

5.1.1 Permitted occupancy

The study of the building and the design of its rehabilitation should be permitted depending on the occupancy subgroup as presented in Table 1.

Table 1 — Permitted occupancies

Occupancy Group		Occupancy Subgroup		Permitted
A	Assembly	A-1	Churches, theatres, stadiums, coliseums, gymnasiums	NO
		A-2	Building having an assembly room with capacity less of 100 persons and not having a stage	YES
B	Business	B	Building for use as offices, professional services, containing eating and drinking establishments with less than 50 occupants	YES
E	Educational	E-1	Classrooms for schools up to high-school	YES
		E-2	Classrooms for universities	YES
F	Industrial	F-1	Light industries not employing heavy machinery	YES
		F-2	Heavy industries employing heavy machinery	NO
G	Garages	G-1	Garages for vehicles with carrying capacity up to 20 kN	YES
		G-2	Garages for vehicles of more than 20 kN carrying capacity	NO
H	Health	H-1	Nurseries for day care of infants	YES
		H-2	Health care centers for ambulatory patients	YES
		H-3	Hospitals	NO
M	Mercantile	M	Display and sale of merchandise	YES
R	Residential	R-1	Hotels	NO
		R-2	Houses and apartment buildings	YES
S	Storage	S-1	Storage of light materials	YES
		S-2	Storage of heavy or hazardous materials	NO
U	Utility	U	Utilities, water supply systems, power generating plants	NO

5.1.2 Mixed occupancy

Study and rehabilitation design of buildings of mixed occupancy should be permitted using these guidelines when all the types of occupancy in the building are permitted by Table 1.

5.2 Maximum number of stories

The maximum number of stories for a building studied using these guidelines should be ten [10]. This number of stories should include the floor at the level of the ground, and should not include either the basement or the roof. The number of basements should not exceed two.

5.3 Maximum aspect ratios

The maximum plan aspect ratio should not exceed [1/5]. The maximum height to length ratio should not exceed [3/1].

5.4 Maximum story height

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

5.5 Maximum difference in story height

Height of consecutive stories should be approximately equal; maximum difference shall not exceed [50%].

5.6 Maximum difference in floor area

Area of consecutive floors should be approximately equal; maximum difference shall not exceed [30%].

5.7 Maximum difference in story mass

Mass of consecutive stories should be approximately equal; maximum difference shall not exceed [30%].

5.8 Maximum column offset

Columns should be continuous and as vertically aligned as possible; column offset shall not exceed [25%] of column dimension in the direction of offset.

5.9 Maximum span length

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

5.10 Maximum difference in span length

Span should be approximately equal, with the larger of two adjacent spans not greater than the shorter by more than [20] percent of the larger span. All the spans must be approximately equal.

5.11 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.

5.12 Maximum slope for slabs, girders, beams and joists

The building may have sloping slabs, girders, beams and joists, but the slope of the structural element should not exceed [15°], except in members that are part of stairways.

5.13 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°].

5.14 Distance between center of mass and center of rigidity

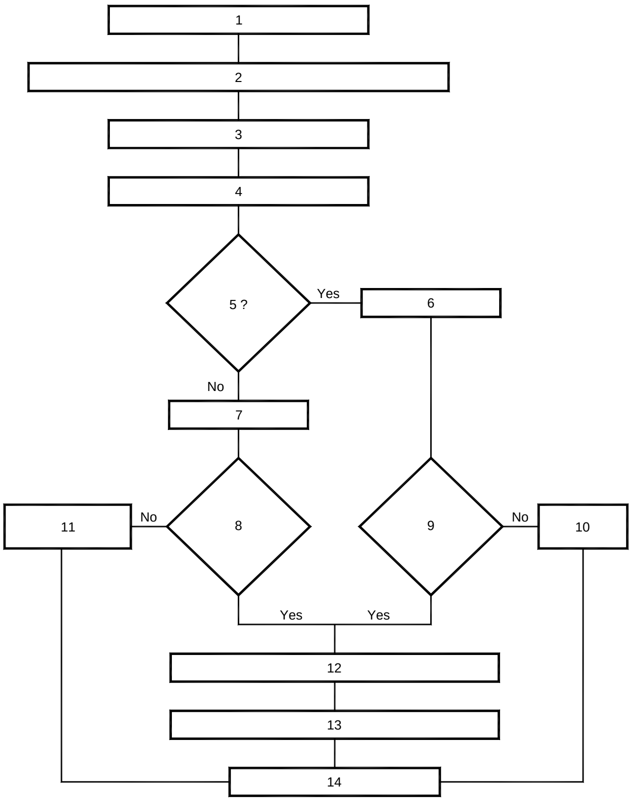
When designing the rehabilitation of a structure that had been assessed using these guidelines the distance between center of mass and center of rigidity shall not be greater than fifteen percent [15 %] of the dimension of the plant of the structure in each direction in order to reduce the risk of global torsion of the structure.

6 Assessment and rehabilitation procedure

6.1 Procedure outline

The suggested procedure for assessment and for rehabilitation of reinforced concrete buildings under seismic loadings is outlined in Figure 1, for both undamaged and damaged structures, i.e., for both structures for which characteristics for an adequate response at a specified performance level are to be evaluated and structures that have undergone damages under seismic loadings.

The rules of design as set forth here are simplifications of the more elaborate requirements in the sense that no detailed structural analysis is required, neither for assessment nor for rehabilitation.



Key

1. data collection (Clause 6.2)
2. lateral load resisting (Clause 6.3)
3. material assesment (Clause 6.4)
4. structure condition assessment (Clause 6.5)
5. structure is damaged?
6. assesment (Chapter .8)
7. assesment (Chapter .9)
8. structure is vulnerable?
9. structure is repairable?
10. Demolition (Clause 12.1)
11. no further action required
12. rehabilitation analysis and design (Chapter 11)
13. rehabilitation construction (Chapter 12)
14. documentation (Clause 6.9)

Figure 1 — Assessment and rehabilitation procedure.

6.2 Data collection

Compiling all the documents of the original building is the first and one of the most important tasks in the procedure. The following list shows the minimum documentation required for the structural assessment:

a) geotechnical report,

b) architectural drawings,

c) structural drawings,

d) site seismicity,

e) structural calculations,

f) construction specifications,

g) foundation reports,

h) structural design codes utilized in the original design and

i) Past remodelling, repair and rehabilitation reports

j) construction year.

6.3 Lateral load resisting system classification

The structure system should be classified into one of the prevalent types of lateral-load-resisting systems for reinforced concrete buildings, as specified in 8.

6.4 Material assessment

Material resistance and soundness should be determined for the existing structure as specified in 9.1.

6.5 Condition assessment

The actual condition of the structure must be evaluated taken into account the soundness of all and every structural and non structural elements and their constituent materials, as per 9.2.

6.6 Structural assessment

An evaluation of the structural capacity of the building should be conducted and as per chapter 9, for structures damaged by a seismic event, except that clause 9.3 is to be used for undamaged structures.

6.7 Rehabilitation design

If the damaged structure is repairable or if the undamaged structure is deemed not to comply with the required characteristics for an adequate response at a specified performance level, the structure has to undergo rehabilitation in order to perform safely during its extended life cycle. It is the responsibility of the owner of the structure to decide whether to repair and rehabilitate the structure or to demolish it. Once the decision to rehabilitate has been made, the structural designer should follow the guidelines contained in chapter 11.

6.8 Rehabilitation construction

Materialization of the rehabilitation process should be developed as per chapter 12.

6.9 Design documentation

The assessment and rehabilitation steps should be fully recorded. The documentation should include, at least, an assessment record, a geotechnical record, a calculation memoir, architectural and structural drawings, and materials and construction specifications.

6.9.1 Assessment record

The structural designer should document all steps followed for the assessment of the building, as per chapter 9, in an assessment record. This record should contain, as a minimum, the following:

- general data on the building,

- a clear justification of the assessment parameters based on the actual state of the building,

- a summary of the procedure followed and the results obtained for the condition assessment,

- a table containing the damage level classification for the building and actions to be taken, if any, such as shoring, partial demolition, etc.

  Drawings showing the distribution and extent of the evidences of damage and deterioration found in the structure.

- a summary of the procedure followed and the results obtained for the structural assessment, and

- a table containing the vulnerability level classification for the building and actions to be taken, if any, such as stiffening, strengthening, etc.

6.9.2 Rehabilitation calculation memoir

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

- a record with all the design and construction documents of the original building,

- a clear justification of the design parameters based on the actual state of the building,

- loads employed in the design,

- calculation memoir of the seismic rehabilitation design,

- presentation of all design computations, and

- sketches of the reinforcement layout for all structural elements.

6.9.3 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 judged relevant by the geotechnical designer.

6.9.4 Structural drawings

All the drawings required for seismic rehabilitation that, at least, should contain the following:

- materials and construction specifications;

- detailed element configuration including, transverse section size, reinforcement dimensions and layout, lap splices, anchorage, etc;

- appropriate markings to differentiate existing from new elements, and

- elevations showing pertinent sections of the structure.

6.9.5 Specifications

The material and construction specifications required should be recorded in a separate report and included in the structural drawings. This record should contain, at least:

- Minimum compressive strength of concrete test cylinders, f'c.

- Minimum yielding stress for all reinforcing steel, fy.

- Maximum aggregate size for concrete mixture.
- Maximum water/cementituos materials ratio.
- Other durability concerns
- Specification for concrete cover, connections, anchorage procedures and required lengths.

7 General Guides

7.1 Limit states

When using these guidelines an undamaged structure is deemed not to comply with the required characteristics for an adequate response at a specified performance level or a structure that has undergone damages under seismic loadings has been assessed for repairs, the design approach of the present guidelines for its rehabilitation is based on limit states, where a limit state is a condition beyond which a structure or member becomes unfit for service and is judged either to be no longer useful for its intended function or to be unsafe.

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

- structural integrity limit state,
- lateral load drift limit state,
- durability limit state, and
- fire limit state.

However, ultimate and serviceability limit states are to be verified through the different stages of design using these guidelines.

7.2 Ultimate limit state design format

7.2.1 General

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

The basic requirement for ultimate limit state should be:

$$\text{Resistances} \geq \text{Load effects} \qquad \text{Equation 1}$$

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

$$\phi \cdot R_n \geq \gamma_1 \cdot S_1 + \gamma_2 \cdot S_2 + \cdots \qquad \text{Equation 2}$$

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

Design Strength ≥ Required Factored Strength Equation 3

or

$\phi \cdot$ (Nominal Strength) $\geq U$ Equation 4

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

7.2.2 Required factored strength

The required factored strength, U, should be computed by multiplying service loads, or forces, by load factors and combinations specified in the standard used for the rehabilitation structural design. When the assessment of the structure has been conducted following these guidelines, rehabilitation specifications required strength may follow ISO 15673.

7.2.3 Design strength

The design strength provided by a member, its connections to other members, and its cross-sections, is then identified by the sub index ***r***, and should be taken as the strength calculated in accordance with the requirements and assumptions for each particular force effect in each of the element types at the critical sections defined by this guidelines, based on the limit stress reduced according to each corresponding material as per Table 2:

Table 2 — Material factor

MATERIAL	γ **mc**	γ **ms**
Cast in place concrete	[1.5]	[1.15]
Standard control not available	[1.7]	[1.25]

7.3 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 this guidelines, should be obtained indirectly thorough the observance of the limiting dimensions, cover, detailing, and construction requirements.

8 Classification of the structure system of the building

Each building shall be classified into one of the following prevalent types of lateral-load-resisting systems, as shown in Table 3.

8.1 Concrete frame systems

Structural systems conformed by a spatial frame, resistant to moments, essentially complete, without diagonal elements or braces, which resists all the vertical and lateral loads.

Concrete frames are composed primarily of horizontal framing components (beams, girders and or or slabs), vertical framing components (columns) and joints connecting horizontal and vertical framing components. These elements resist lateral loads acting alone.

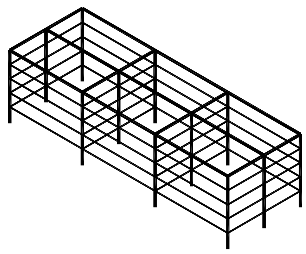
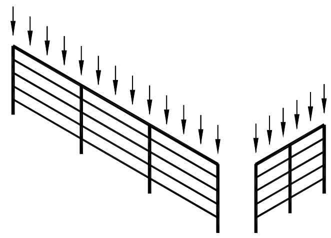
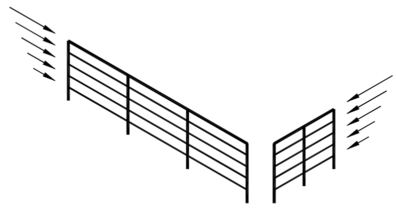
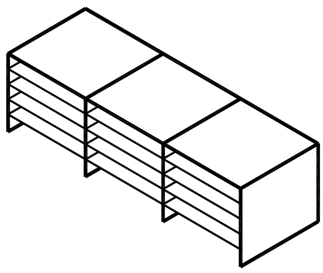
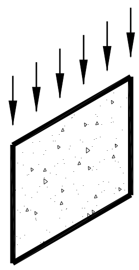
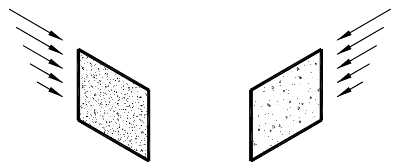
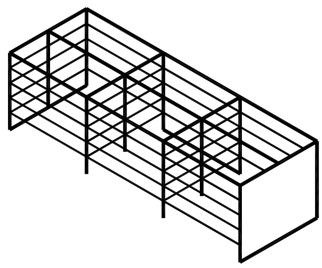
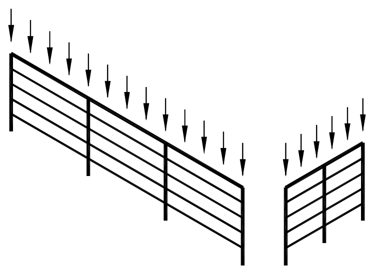
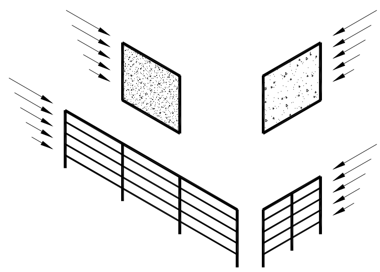
8.2 Concrete wall systems

Structural systems in which both, vertical and lateral, loads are resisted by structural walls or frames with diagonal elements or braces.

8.3 Concrete dual systems

Structural systems comprised of a moment resistant spatial frame without diagonal elements or braces, combined with structural walls or frames with diagonal elements or braces.

Table 3 — Structural systems

STRUCTURAL SYSTEM	LOAD CARRYING SYSTEM	
	VERTICAL LOADS	LATERAL LOADS
FRAMES 		
WALLS 		
DUAL 		

9 Condition assessment of structures damaged by a seismic event

The buildings that have suffered moderate or severe damages after an earthquake occurrence in their structure or in their non structural elements, or in both, and that have not been subjected to mandatory demolition orders issued by the competent authority, must be studied in detail, in agreement with the next requirements and criteria, in order to establish the nature and extent of the damages.

9.1 Material assessment

Material resistance and soundness should be determined for the existing structure. Mechanical properties for concrete materials and components shall be based on available construction documents and as-built conditions for the particular structure. Where such information fails to provide adequate information to quantify material properties or document the condition of the structure, it shall be supplemented by materials tests and assessments of existing conditions.

9.1.1 Material properties

The following component and connection material properties shall be obtained for the as-built structure:

- concrete compressive strength;
- yield and ultimate strength of reinforcing steel and metal connection hardware;

When material testing is required, the test methods to quantify material properties shall comply with the requirements of Section 9.1.3. The frequency of sampling, including the minimum number of tests for property determination shall comply with the requirements of Section 9.1.4

Other material properties that may be of interest for concrete elements and components include:

- tensile strength and modulus of elasticity, which can be derived from the compressive strength, do not warrant the damage associated with the extra coring required;
- ductility, toughness, and fatigue properties of concrete;
- carbon equivalent present in the reinforcing steel;
- presence of any degradation such as corrosion, bond with concrete and chemical composition.

The effort required to determine these properties depends on the availability of accurate updated construction documents and drawings, quality and type of construction (absence of degradation), accessibility, and condition of materials. Generally, mechanical properties for both concrete and reinforcing steel can be established from combined core and specimen sampling at similar locations, followed by laboratory testing. Core drilling should minimize damaging the existing reinforcing steel as much as practicable.

If no testing facilities are available the elements resistance should be evaluated using a maximum concrete compressive strength of [17,5 MPa] and the steel yield strength must be estimated as [240 MPa] for undeformed reinforcing bars or deformed reinforcing bars with diameters equal or less than 10M (10 mm) or No. 3 (3/8 in) or bars manufactured prior to 1971 and as [414 MPa] for other reinforcing bars.

9.1.2 Component properties

The following component properties and as-built conditions shall be established:

1. cross-sectional dimensions of individual components and overall configuration of the structure;
2. configuration of component connections, size of anchor bolts, thickness of connector material, anchorage and interconnection of embedments and the presence of bracing or stiffening components;
3. modifications to components or overall configuration of the structure;
4. current physical condition of components and connections, and the extent of any deterioration present, and
5. presence of conditions that influence building performance.

9.1.3 Test methods

If the determination of material properties is accomplished through removal and testing of samples for laboratory analysis, sampling shall take place in primary gravity- and lateral-force-resisting components in regions with the least stress.

For concrete testing, the sampling program shall consist of the removal of standard cores. Core drilling shall be preceded by de termination of the location of the reinforcing steel by means of nondestructive testing. Core holes shall be filled with comparable strength concrete or grout. If reinforcing steel is tested, sampling shall consist of the removal of local bar segments and installation of replacement spliced material to maintain continuity of the rebar for transfer of bar force.

Removal of core samples and performance of laboratory destructive testing shall be permitted as a method of determining existing concrete strength properties. Removal of bar length samples and performance of laboratory destructive testing shall be permitted as a method of determining existing reinforcing steel strength properties. Properties of connector steels shall be permitted to be determined by wet and dry chemical composition tests, and direct tensile and compressive strength tests.

9.1.4 Minimum number of tests

9.1.4.1 General

If the existing vertical or lateral force-resisting system is being replaced in the rehabilitation process, material testing shall be required only to qualify properties of existing materials at new connection points.

Material testing is not required if material properties are available from original construction documents that include material test records or material test reports. Testing is generally not required on components other than those of the lateral-force-resisting system.

9.1.4.2 Comprehensive testing

Unless specified otherwise, a minimum of three tests shall be conducted to determine any property. If the coefficient of variation exceeds 14 %, additional tests shall be performed until the coefficient of variation is equal to or less than 14 %.

9.1.4.3 Concrete materials

For each concrete element type (such as beams, columns or walls) one test shall be comprised of minimum three (3) core samples subjected to compression loading. A minimum of six tests shall be performed on a building for concrete strength determination, subject to the limitations of this section. If varying concrete classes/grades were employed in the construction of the building, a minimum of three tests shall be performed for each class. The modulus of elasticity shall be permitted to be estimated from the data of strength testing. Samples shall be taken from randomly selected components critical to structural behavior of the building.

The minimum number of tests to determine compressive and tensile strength shall conform to the following criteria.

For concrete elements for which the specified design strength is known and test results are not available, a minimum of three (3) cores/tests shall be conducted for each floor level, [300 m^3], or

[1 000 m^2] of surface area, whichever requires the most frequent testing.

For concrete elements for which the design strength is unknown and test results are not available, a minimum of six (6) cores/tests shall be conducted for each floor level, [300 m^3], or [1000 m^2] of surface area, whichever requires the most frequent testing. Where the results indicate that different classes of concrete were employed, the degree of testing shall be increased to confirm class use.

Quantification of concrete strength via ultrasonic wave propagation or other nondestructive test methods may be used for comparative purposes or to compare strengths of elements not required to be tested but shall not substitute core sampling and laboratory testing.

9.1.4.4 Reinforcing and connector steels

The minimum number of tests required to determine reinforcing and connector steel strength properties shall be as follows. Connector steel shall be defined as additional structural steel or miscellaneous metal used to secure precast and other concrete shapes to the building structure. Tests shall determine both yield and ultimate strengths of reinforcing and connector steel. A minimum of three (3) tensile tests shall be conducted on conventional reinforcing steel samples from a building for strength determination, subject to the following supplemental conditions.

If original construction documents defining properties exist, at least three (3) strength coupons shall be randomly removed from each element or component type and tested.

If original construction documents defining properties do not exist, but the approximate date of construction is known and a common material grade is confirmed, at least three (3) strength coupons shall be randomly removed from each element or component type for every three (3) floors of the building. If the date of construction is unknown, at least six (6) such samples/tests, for every three floors, shall be performed.

All sampled steel shall be replaced with new fully spliced and connected material.

9.1.5 Soundness

The physical condition of components and connectors will affect their performance. The need to accurately identify the physical condition may also dictate the need for certain additional destructive and nondestructive test methods. Such methods may be used to determine the degree of damage or presence of deterioration, and to improve understanding of the internal condition and quality of the concrete. Further guidelines and procedures for destructive and nondestructive tests (NDT) that may be used in the condition assessment are provided in the following paragraphs:

Surface NDT methods include infrared thermography, delamination sounding, surface hardness measurement, and crack mapping. These methods may be used to find surface degradation in components such as service-induced cracks, corrosion, and construction defects.

Volumetric NDT methods, including radiography and ultrasonic wave propagation, may be used to identify the presence of internal discontinuities, as well as to identify loss of section. Impact-echo ultrasonics is particularly useful because of ease of implementation and proven capability in concrete.

Locating, sizing, and initial assessment of the reinforcing steel may be completed using electromagnetic methods or radiography.

Assessment of suspected corrosion activity should use electrical half-cell potential and resistivity measurements.

9.2 Condition Assessment

9.2.1 Layout of structural damages

Nature and extent of damages should be assessed for the structure under study. Damages should be classified by levels as per 9.2.2.

To accurately assess the damage present in the structural elements, it is necessary to distinguish between environment conditions induced cracks and stress induced cracks and, for the latter, to distinguish between flexural cracks and shear cracks; it is also necessary to identify cracks that may indicate lap-splice or anchorage slipping. Crack widths should be determined in order to contribute in assessing the severity of earthquake damage in reinforced concrete components.

A detail registry of all evidences should be drawn including relevant data such as nature of the defect or damage and their location, extent, length, width, thickness, among others, as applicable.

9.2.1.1 Flexural and shear cracks

Flexural cracks are those that develop perpendicular to flexural tension stresses. In beams, flexural cracks run vertically, as they also do in wall spandrels, while in wall piers, flexural cracks run horizontally. Flexural cracks typically initiate at the extreme fiber of a section and propagate towards the section's neutral axis. For components that have undergone cyclic earthquake displacements in both directions, opposing flexural cracks often join with each other to form a relatively straight crack through the entire section.

Shear cracks are those that result from tension stresses corresponding to applied shear forces. The cracks run diagonally, typically at an angle of 35º to 70º from the horizontal. The angle of cracking depends on normal forces (e.g., axial load) and on the geometry of the component. For components that have undergone cyclic earthquake displacements of similar magnitude in both directions, the cracks cross each other, forming X patterns. Flexural cracks often join up with diagonal shear cracks. A typical case is in a wall pier where a horizontal crack at the wall boundary curves downward to become a diagonal shear crack as it approaches the pier centerline. When shear cracks connect to flexural cracks, determine the widths of the flexural portion of the crack and the shear portion of the crack separately. Cracks initially form perpendicular to the direction of the principal tension stresses in a section. At any point of a component, it is possible to relate the orientation of initial cracking to the applied stresses by considering the stress relationships represented by Mohr's Circle. However, after initial cracking, the orientation of principal stresses will change and crack patterns and stress orientations are affected by the reinforcement.

9.2.1.2 Full-thickness versus partial-thickness cracking

In investigating reinforced concrete components, the designer should establish whether critical flexural and shear cracks extend through the thickness of the element. It is assumed that the most significant flexural and shear cracks are full-thickness cracks having a similar crack width on each side of the element. Laboratory tests have invariably used in-plane loading. Therefore, significant cracks observed in these studies are typically full-thickness. In actual buildings, out-of plane forces and deformations may cause cracks to be partial-thickness, or they may result in cracks that remain open to a measurable width on one face of the element, but are completely closed on the opposite face. In such cases, the designer should use judgment in assessing the consequences of the critical cracks. It may be justified to use the average of the measured crack width on each face. More conservatively, the maximum crack width on either face of the element can be used in classifying the observed damage.

9.2.1.3 Cracking as a precursor to spalling

In the compression region of concrete structural components, cracks occur as a precursor to concrete spalling. Such cracks form parallel to the principal compression stresses, and they may develop when compressive strains in the concrete exceed 0,003 to 0,005. Such cracking typically signals an increased damage severity. This type of cracking occurs (1) at the boundary regions of component plastic-hinge zones for flexural behavior, and (2) under a diagonal-compression (web-crushing) type of shear failure.

The cracking in compression regions of flexural members could appear similar to splitting cracks resulting from lap-splice or bond slip of the reinforcement. Both types of cracking tend to occur in the boundary regions of plastic-hinge zones. Some distinguishing features of the two different types of cracks are described below:

a) Cracks as a precursor to spalling in the compression region:

   1) occur under conditions of high compressive strain,

   2) cracks may be relatively short. Sounding with a hammer may reveal incipient spalling, and

   3) cracks occur at the extreme fibers of the section, typically within the cover of the concrete.

b) Bond or lap-splice splitting cracks:

   1) occur at the locations of longitudinal reinforcement that is susceptible to bond or lap-splice slip. (Large bar diameters or inadequate lap-splice length.)

   2) cracks tend to be relatively long and straight, mirroring rebar locations. The cracks originate at the reinforcement and propagate to the concrete surface.

9.2.1.4 Splitting cracks at lap splices

If lap splices are insufficient to develop the required tension forces in the reinforcement, slip occurs at the splices. The visible evidence of lap-splice slip is typically longitudinal cracks (parallel to the splice) that originate at the lap splice and propagate to the concrete surface. Thus, the crack locations reflect the locations of the lap-spliced reinforcement.

9.2.1.5 Crack widths

Crack widths are to be measured according to the investigation procedures outlined in this document. The maximum crack width defines the damage severity. When multiple cracks are present, the widest crack of the type being considered (e.g., shear or flexure) governs the damage severity classification. The maximum crack width may be significantly larger than the average width of a series of parallel cracks. Although average crack width may be a better indicator of average strain in the reinforcement, maximum crack width is judged to be more indicative of maximum reinforcement strain, and, in general, damage severity. A concentration of strain at one or two wide cracks typically indicates an undesirable behavior mode and more serious damage, whereas an even distribution of strain and crack width among numerous parallel cracks indicates better seismic performance.

The crack width criteria are based on a comparison to research results rather than on detailed analyses of crack width versus strain relationships. The criteria recognize that the residual crack width observed after an earthquake may be less than the maximum crack widths occurring during the earthquake.

9.2.2 Damage levels

Damages should be classified in one of five categories: Insignificant, slight, moderate, serious or severe. Classification should be made according to type and intensity of damages, recording the element type, element location, and all relevant data about the damage characteristics.

For instance, Table 4 gives a classification for concrete beams under flexure; Table 5 may be used as guides to assess damages in concrete columns due to local effects or lack of transverse reinforcement; whereas Table 6 may be used as guides for appropriately assessing damages in ends of concrete beams and columns.

Furthermore, the assessment of damages in concrete frame joints is classified in Table 7; and the classification of damage in isolated wall or strong pier with ductile behavior under flexure, with shear stresses due to flexure, with web crushing due to flexure, with slip at the base, with edge compression due to flexure are given by Table 8, Table 9, Table 10, Table 11 and Table 12, respectively.

On the other hand, damage classification for an isolated wall or weak pier with ductile behavior under flexure is given by Table 13 and with incipient shear stresses due to flexure, by Table 14.

Table 4 — Damage classification for concrete beams under flexure

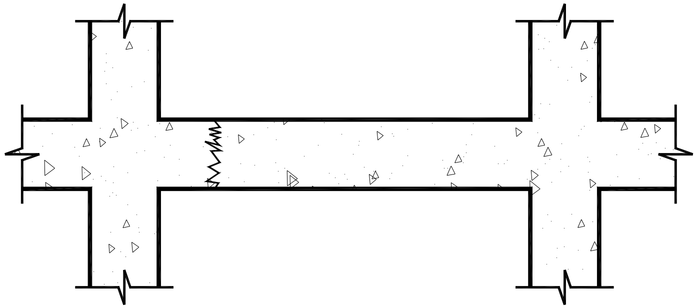
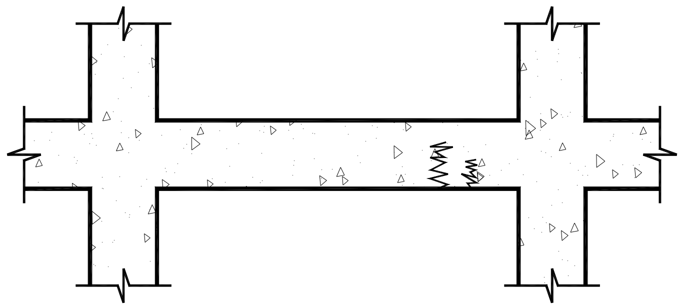
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	No cracks or cracks width less than 0,2 mm Vertical crack generally dividing the whole section Position next to joint	
SLIGHT	Cracks width between 0,2 mm and 1 mm Vertical crack generally dividing the whole section	
MODERATE	Cracks width between 1 mm and 2 mm Vertical crack generally dividing the whole section	
SERIOUS	Cracks width between 2 mm and 6 mm Vertical crack generally dividing the whole section Exposure of reinforcement	
SEVERE	Cracks width bigger than 6 mm Spalling Possible buckling of longitudinal reinforcement	
DAMAGE LEVELS	**DAMAGE DESCRIPTION**	**TYPICAL APPEREANCE**
INSIGNIFICANT	Vertical cracks or with slight inclination At any place along length, except close to joints	
SLIGHT	Cracks width between 0,2 mm and 1 mm Similar to previous classification but with wider cracks	
MODERATE	Cracks width between 1 mm and 2 mm Similar to previous classification but with wider cracks	
SERIOUS	Cracks width between 2 mm and 6 mm Similar to previous classification but with wider cracks	
SEVERE	Cracks width bigger than 6 mm Spalling Possible buckling of longitudinal reinforcement	

Table 5 — Damage classification for concrete columns due to local defects or lack of transverse reinforcement

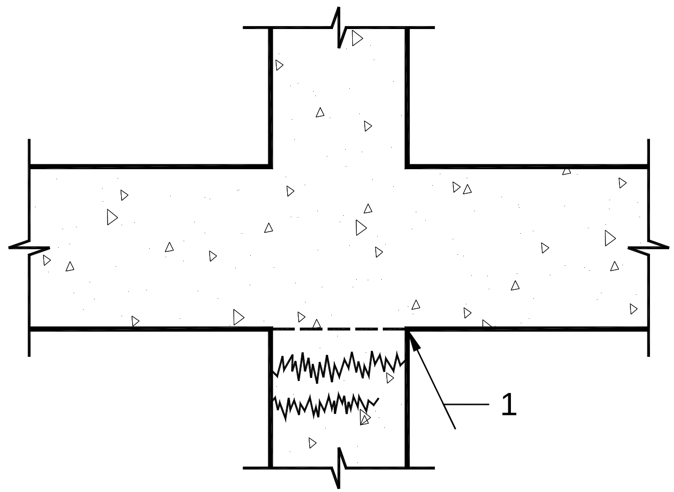
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	Horizontal crack near joint Not necessarily across whole section	 **Key.** 1. Construction Joint.
SLIGHT	Cracks width between 0,2 mm and 1 mm Similar to previous classification but wider cracks	
MODERATE	Cracks width between 1 mm and 2 mm Similar to previous classification but wider cracks	
SERIOUS	Cracks width between 2 mm and 6 mm Similar to previous classification but with wider cracks	
SEVERE	Cracks width bigger than 6 mm Spalling Possible buckling of longitudinal reinforcement	

Table 5 — Damage classification for concrete columns due to local defects or lack of transverse reinforcement (*continued*)

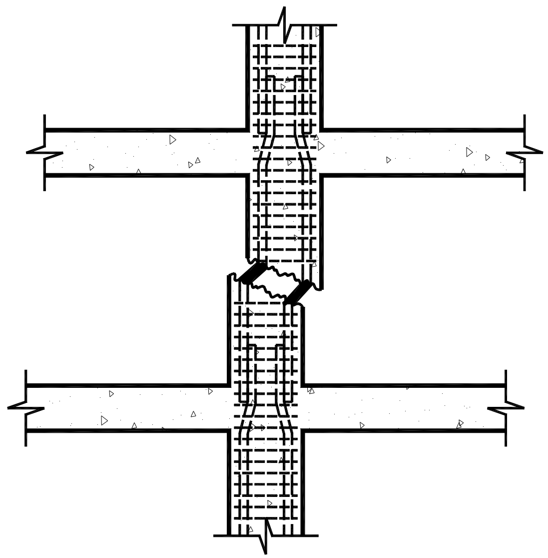
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	Imperceptible cracking	
SLIGHT	Cracks width between 0,2 mm and 1 mm	
MODERATE	Cracks width between 1 mm and 2 mm Similar to previous classification but with wider cracks	
SERIOUS	Cracks width between 2 mm and 6 mm	
SEVERE	Cracks width bigger than 6 mm Spalling Possible buckling of longitudinal reinforcement	

Table 6 — Classification for damage at ends of concrete beams and columns

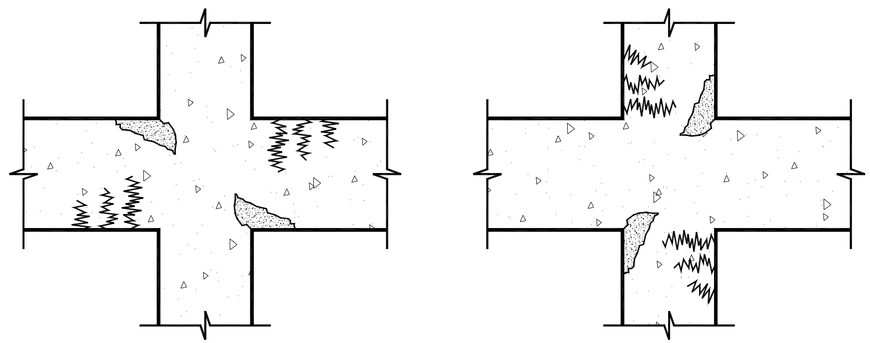
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	Imperceptible cracking Parallel flexure cracks at opposite sides close to joint Spalling at opposite sides close to joint	
SLIGHT	Cracks width between 0,2 mm and 1 mm Similar to previous classification but with wider cracks	
MODERATE	Cracks width between 1 mm and 2 mm Similar to previous but wider cracks	
SERIOUS	Cracks width between 2 mm and 6 mm Similar to previous classification but with wider cracks	
SEVERE	Cracks width bigger than 6 mm Spalling Possible buckling of longitudinal reinforcement	

Table 7 — Damage classification for concrete frame joints

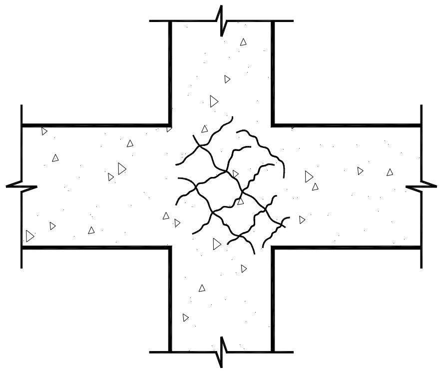
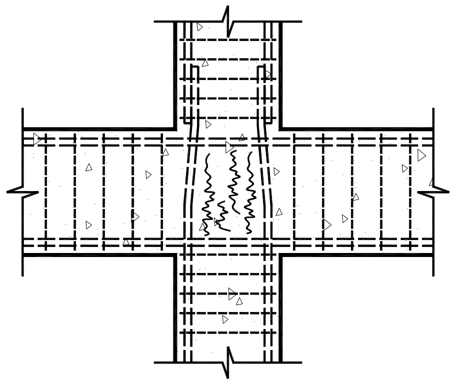
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	Imperceptible cracking Diagonal cracks at joint in both directions	
SLIGHT	Cracks width between 0,2 mm and 1 mm Similar to previous classification but with wider cracks	
MODERATE	Cracks width between 1 mm and 2 mm Similar to previous classification but with wider cracks	
SERIOUS	Cracks width between 2 mm and 6 mm Similar to previous classification but with wider cracks	
SEVERE	Cracks width bigger than 6 mm Material expelling	
DAMAGE LEVELS	**DAMAGE DESCRIPTION**	**TYPICAL APPEREANCE**
INSIGNIFICANT	Imperceptible cracking One or more parallel cracks Parallel direction to the column axis	
SLIGHT	Cracks width between 0,2 mm and 1 mm Similar to previous classification but with wider cracks	
MODERATE	Cracks width between 1 mm and 2 mm Similar to previous classification but with wider cracks	
SERIOUS	Cracks width between 2 mm and 6 mm Similar to previous classification but with wider cracks	
SEVERE	Cracks width bigger than 6 mm Material expelling	

Table 8 — Damage classification for an isolated wall or strong pier with ductile behavior under flexure

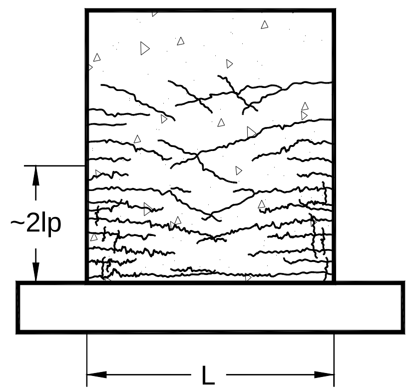
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	Cracks width less than 4 mm shear cracks width less than 3 mm No spalling nor vertical cracking	
SLIGHT	Cracks width less than 4 mm shear cracks width less than 3 mm Similar to previous classification but with wider cracks	
MODERATE	Spalling and vertical cracking on wall's edge base cracks width less than 6 mm	
SERIOUS	Not used	
SEVERE	Fractured reinforcement Wide flexion fissure and concentrated in one single crack Big residual displacements	

Table 9 — Damage classification for an isolated wall or strong pier with shear stresses due to flexure

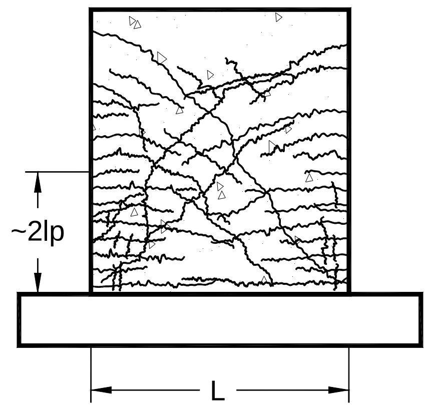
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	Flexion Cracks width less than 5 mm Shear cracks width less than 1,5 mm No spalling nor vertical cracking	
SLIGHT	Not used	
MODERATE	Shear cracks width between 1,5 mm and 3 mm Flexure cracks width less than 6 mm Partial spalling	
SERIOUS	Shear cracks width between 3 mm and 9 mm	
SEVERE	Fractured reinforcement Wide shear fissure and concentrated in one single crack	

Table 10 — Damage classification for an isolated wall or strong pier with web crushing due to flexure

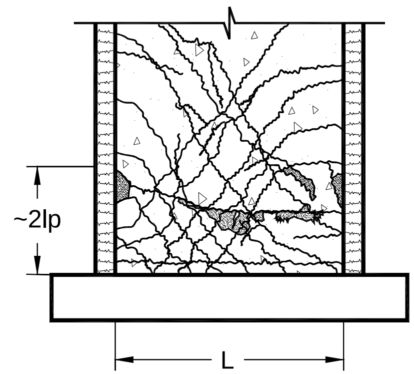
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	Flexion Cracks width less than 5 mm Shear cracks width less than 1,5 mm No spalling nor vertical cracking	
SLIGHT	Shear cracks width less than 3 mm Flexure cracks width less than 6 mm Partial spalling on the edge	
MODERATE	Not used	
SERIOUS	Significant spalling	
SEVERE	Big Spalling and empties in the soul Significant residual displacements	

Table 11 — Damage classification for an isolated wall or strong pier with slip at the base

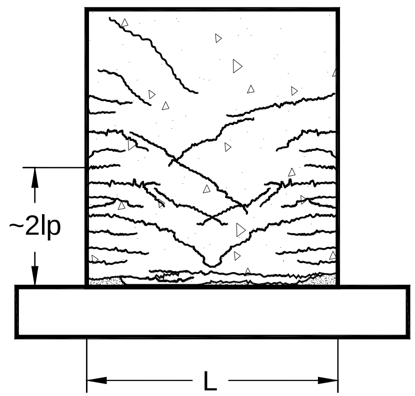
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	Cracks width less than 4 mm Shear cracks width less than 3 mm No spalling nor vertical cracking	
SLIGHT	Cracks width less than 4 mm Shear cracks width less than 3 mm Similar to previous classification but with wider cracks	
MODERATE	Not used	
SERIOUS	Development of Important horizontal crack on the whole wall longitude. With some concrete degradation Possible lateral sliding	
SEVERE	Significant lateral movement out of plane of sliding surface	

Table 12 — Damage classification for an isolated wall or strong pier with edge compression due to flexure

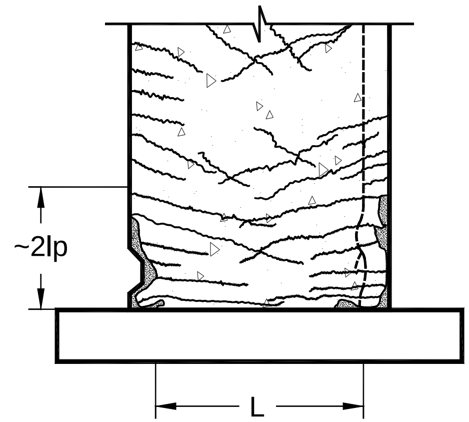
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	Cracks width less than 4 mm Shear cracks width less than 3 mm No spalling nor vertical cracking	
SLIGHT	Cracks width less than 4 mm Shear cracks width less than 3 mm Similar to previous classification but with wider cracks	
MODERATE	Spalling and vertical cracking on wall's edge base Cracks width less than 6 mm	
SERIOUS	Spalling and vertical cracking on plastic zones edges Buckling of edge vertical reinforcement Cracks width less than 9 mm	
SEVERE	Fractured reinforcement Wide flexion fissure and concentrated in one single crack Big residual displacements	

Table 13 — Damage classification for an isolated wall or weak pier with ductile behavior under flexure

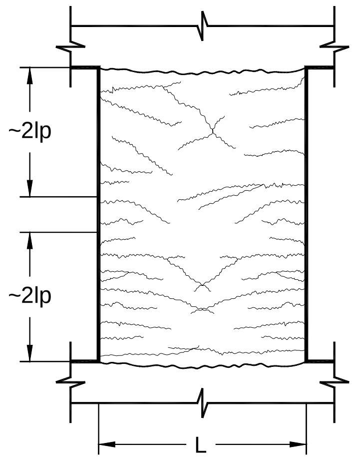
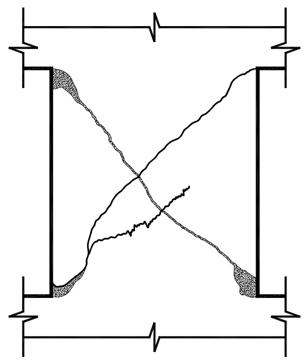
DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	Cracks width less than 4 mm Shear cracks width less than 3 mm No spalling nor vertical cracking	
SLIGHT	Cracks width less than 4 mm Shear cracks width less than 3 mm Similar to previous classification but with wider cracks	
MODERATE	Spalling or vertical cracking on the plastic zone edges Cracks width less than 6 mm	
SERIOUS	Not used	
SEVERE	Fractured reinforcement Wide flexion fissure and concentrated in one single crack Big residual displacements	

Table 14 — Damage classification for an isolated wall or weak pier with incipient shear stresses due to flexure

DAMAGE LEVELS	DAMAGE DESCRIPTION	TYPICAL APPEREANCE
INSIGNIFICANT	flexure cracks width less than 3 mm No shear cracking	
SLIGHT	Not used	
MODERATE	Cracks width less than 3 mm No vertical cracking nor spalling	
SERIOUS	The fissures concentrate in one or more cracks shear cracks width between 3 mm and 9 mm	
SEVERE	Fractured reinforcement Wide shear fissure concentrated in one single crack	

9.2.3 Layout of non structural damages

Non structural items may also be damaged during an earthquake. Non structural elements are elements such as mechanical and electrical equipment, hydraulic and sanitary piping, partition walls, façade cladding, window and façade glass and parapets may fall or tumble down during a strong earthquake. Loose, badly anchored or connected, or poorly configured nonstructural elements may represent a danger to life safety; however, some non structural damage may affect the structural stiffness of the building. In such cases it could represent a hazard to its structural integrity. For the purpose of these guidelines, non structural damage is considered significant when it may affect structural integrity or stability.

9.2.3.1 Mechanical equipment

Mechanical equipment such as elevators, escalators and their motors, air-conditioning fan and compressors, garage doors and motors, door jacks, among others, may represent a hazard to its inhabitants when they are damaged or fail during an earthquake. Special attention should be given to the condition of their anchorage and connections. Damage on mechanical equipment seldom affects the structural integrity and stability of the building.

9.2.3.2 Hydraulic and sanitary installations

Hydraulic and sanitary installations may also represent a life hazard, especially when hydraulic pipes conduct hot liquids or gases. Special attention should be given to their anchorages and splices. Damage on hydraulic piping systems seldom affects the structural integrity and stability of the building.

9.2.3.3 Partition walls

Partition walls are of concern particularly when built with heavy materials such as stone, clay or concrete masonry. Unstable partitions may pose a danger for life safety. Stiffness of frames with masonry infills may be affected by partial degradation of partitions causing additional structural damage due to global torsion.

9.2.3.4 Facade cladding

Heavy cladding cladding elements such as ceramic tiles, precast concrete panels or cut stone, may fall off the building during an earthquake if improperly anchored. The loss of panels may also create major changes to the building stiffness (the elements are considered nonstructural but often contribute substantial stiffness to a building), thus setting up plan irregularities or torsion when only some fall. The existence of heavy cladding is of concern if the connections were not properly designed and installed, i.e., if their design and installation follow specifications with no seismic anchorage requirements or when no design or installation specification were followed. Glass curtain walls are not considered as heavy cladding.

9.2.3.5 Window and facade glass

Glass on windows or on façade glass curtains may break and fall when window or curtain frames are improperly gasketed and fastened. Glass on windows or facades seldom affects the structural integrity and stability of the building.

9.2.3.6 Parapets

Unbraced parapets are difficult to identify from the street as it is sometimes difficult to tell if a facade projects above the roofline. Parapets often exist on three sides of the building, and their height may be visible from the back of the structure. Parapet damage seldom affects the structural integrity and stability of the building.

9.2.3.7 Non structural damage levels

For the purpose of these guidelines, non structural damage should be classified according to their effects on structural integrity or stability, as outlined in Table 15.

Table 15 — Non structural damage classification

EFFECT ON STRUCTURAL INTEGRITY OR STABILITY	DESCRIPTION
INSIGFNIFICANT	Damage on Mechanical equipment Hydraulic piping systems Glass Parapets Cracks of less than 1 mm in masonry walls. Cracks on cladding but no fallouts.
SLIGHT	Cracks of less than 3 mm in masonry walls. Less than 20 % cladding fallout.
MODERATE	Cracks of less than 10 mm in masonry walls. Between 20 % and 40 % cladding fallout.
SERIOUS	Cracks of more than 10 mm in masonry walls. Between 40 % and 60 % of cladding fallout
SEVERE	Out of plane movements of masonry partitions. Partial collapse of masonry partitions. More than 60 % cladding fallout.

9.3 Structural assessment

The structural capacity must be evaluated according to 10.8.

9.4 Final assessment

The damage condition of the building should be qualified by the maximum level of damage sustained by its structural elements or non structural elements which may interact with the stiffness of the structure. A final assessment should be conducted taking into account the building damage condition qualification and its structural assessment, as per clause 10.9.

10 Condition assessment of existing structures

When an existent structure located in seismic hazard regions of the world was built before the adoption by local authorities, or by good engineering practice, of state-of-the-art earthquake resistant design and construction procedures, its owners should evaluate it to determine whether it is vulnerable to lateral loading.

The seismic evaluation of an existent structure, when it has not been yet affected by a seismic event, should be implemented according to the specifications set forth in this chapter.

10.1 Vulnerability level

In order to determinate if a building is potentially seismically hazardous its vulnerability level must be estimated. Vulnerability is defined as the building's susceptibility to sustain damage, on both structural and non structural elements, in case of a strong ground shaking carrying forces similar to those specified by the design earthquake and should be evaluated based on specific element vulnerability as per 10.2 through 10.8.

This susceptibility indicates the building's vulnerability in terms of severe damage, partial or total collapse if ground motions occur that equal or exceed the maximum earthquake ground motions considered to possibly affect the building's site. The vulnerability should be graded in one of four levels, 1 being the least vulnerable, for which no action is needed, and four being the most vulnerable, for which further material and structural assessment is needed. The suggested actions to be taken in each case are shown in Table 28.

Table 16 — Vulnerability levels for undamaged buildings

LEVEL	VULNERABILITY
1	None
2	Low
3	Moderate
4	High

Vulnerability assessment is based on the following considerations:

- actual condition of the structure,
- seismic hazard,
- architectural layout,
- structural configuration,
- quality control at construction time,
- slope of terrain,
- soil type, and
- interaction with non structural elements.

10.2 Actual condition of the structure

When speaking about damages, the main focus of these guidelines is concerned with damages caused by seismic events. However, to evaluate the actual condition of an existing structure, damages due to design shortcomings, usually referred to as *defects*, damages due to gravity loads, usually referred to as *damages*, and damages due to environmental actions, usually referred to as *deterioration*, must be considered.

Some of the most common evidences of defects, damages and deterioration found in existing structures are:

- Corrosion,
- Cracking,
- Deflection,
- Delamination,
- Disintegration,
- Lixiviation,
- Scaling,

- Settlement,
- Spalling, and
- Wear

All evidences of damages should be adequately recorded, graded and documented. Evidence should be graded according to their extent expressed as a percentage of affected elements or areas in relation to the total number of elements or total area, as per Table 17.

Table 17 — Non seismic damage classification as percentage of total area

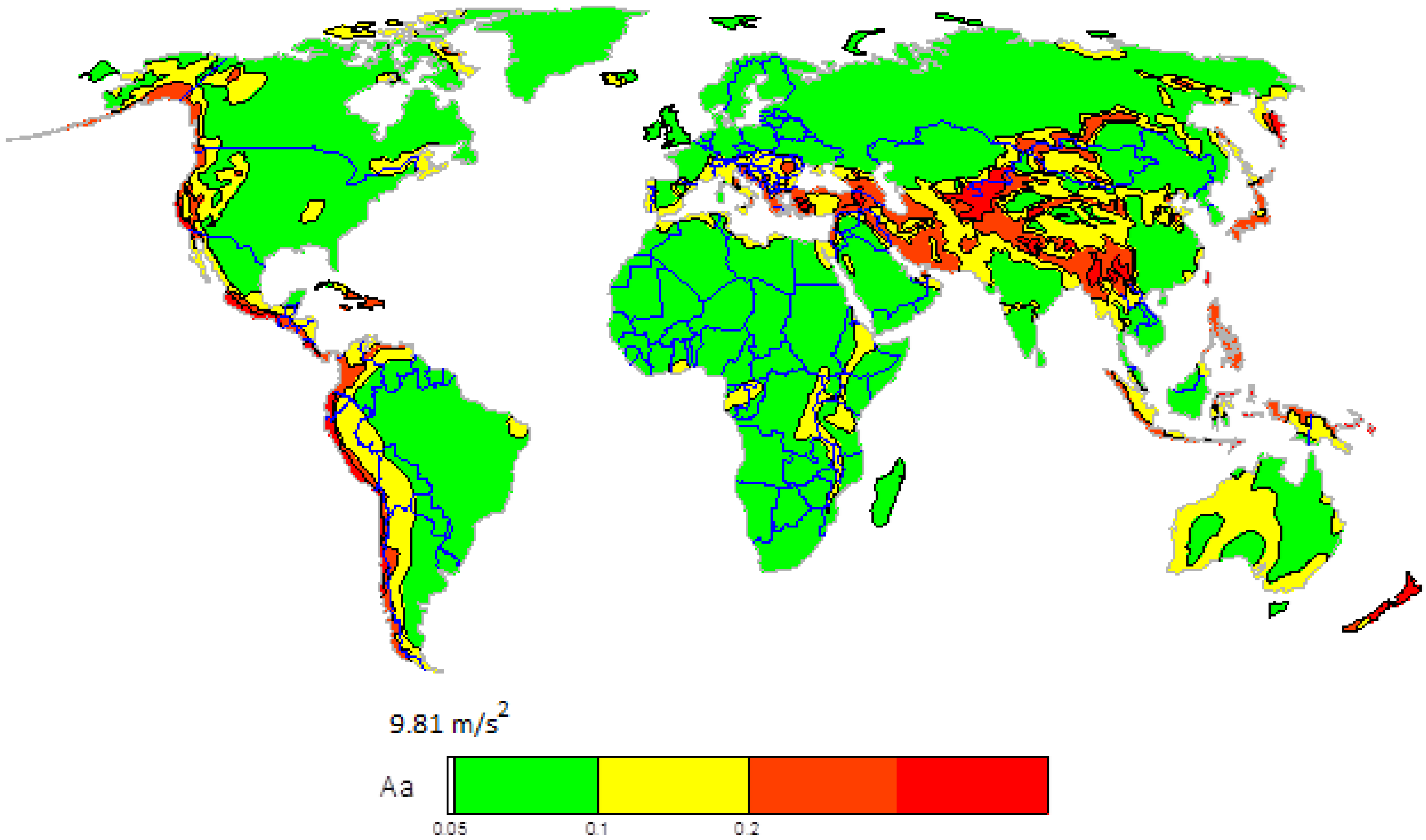
LEVEL	DAMAGE EXTENT	QUALIFICATION
INSIGNIFICANT	< [5] %	0
SLIGHT	[5] % - [15] %	1
MODERATE	[15] % - [25] %	2
SERIOUS	[25] % - [55] %	3
SEVERE	>[55] %	4

10.3 Seismic hazard

The first step in the seismic evaluation of an existing structure is the determination of the hazard level of the site where the building is located.

Seismic hazard should be classified in terms of the intensity of the effective peak ground horizontal acceleration in rock at the different sites for which the seismic hazard is being classified. The peak rock acceleration corresponds to the median spectral acceleration for one degree of freedom systems with short periods of structural vibration, i.e., periods not exceeding 0,15 seconds, and is usually denoted as A_a, expressed as a fraction of the acceleration of gravity (acceleration of gravity ≈ 9,81 $^{m}/_{s^2}$), g.

For the purpose of the scope of these guidelines, the values for A_a must be taken from the national corresponding standard having jurisdiction over the site of the considered existing structure. When the national code defines the maximum seismic ground motion for each considered site based on spectral response accelerations at 5 % of critical damping, S_S, A_a may be estimated as the value of S_S for a period of 0,15 seconds, divided by 375 ($A_a = S_S/375$). When the national code defines the maximum seismic ground motion for each considered site based on a seismic zone factor Z, the value of A_a should be taken equal to Z. When no national code exists for the site of the building being considered, A_a may be estimated from the world seismic hazard map shown in Figure 2.



Key

No hazard 0 < Aa< 0.05
Low Hazard 0.05 < Aa < 0.1
Intermediate Hazard. 0.1 < Aa < 0,2
High Hazard Aa > 0,2

Figure 2 — Global seismic hazard map

10.3.1 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.

10.3.2 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.

10.3.3 Intermediate seismic hazard zones

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

10.3.4 High seismic hazard zones

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

10.4 Architectural layout

The architectural layout of the building determines aspects that are basic for its response characteristics under seismic forces. Evaluation of these aspects includes regularity of building's shape, both in plant and in elevation. Regular shapes are helpful in providing for better seismic response. In contrast, irregular shapes might be an indication of possible inadequate seismic response. Buildings classified as irregular may not be suited for rehabilitation under these guidelines, unless the rehabilitation procedure includes the elimination of the irregularities.

10.4.1 Plan irregularity

The architectural plan layout of the building under study should be classified as regular or irregular. Square, circular, polygonal or low aspect ratio rectangular shapes may be considered as regular, as shown in Figure 3. Plan irregularity can affect all building types. Damage is likely to occur due to stress concentration at the location of abrupt shape changes which, in turn, represent sudden change in horizontal diaphragms stiffness.

Examples of plan irregularity include buildings with corner recesses, buildings with large voids within their horizontal diaphragms, buildings with good lateral-load resistance in one direction but not in the other and buildings with major stiffness eccentricities in the lateral-force-resisting system, which may cause twisting (torsion) around a vertical axis. Plan irregularities causing torsion are especially prevalent among corner buildings, in which the two adjacent street sides of the building are largely windowed and open, whereas the other two sides are generally solid. Wedge-shaped buildings, triangular in plan, on corners of streets not meeting at 90°, are similarly susceptible to damage.

Table 18 provides a guide as to which plan shapes may be deemed irregular.

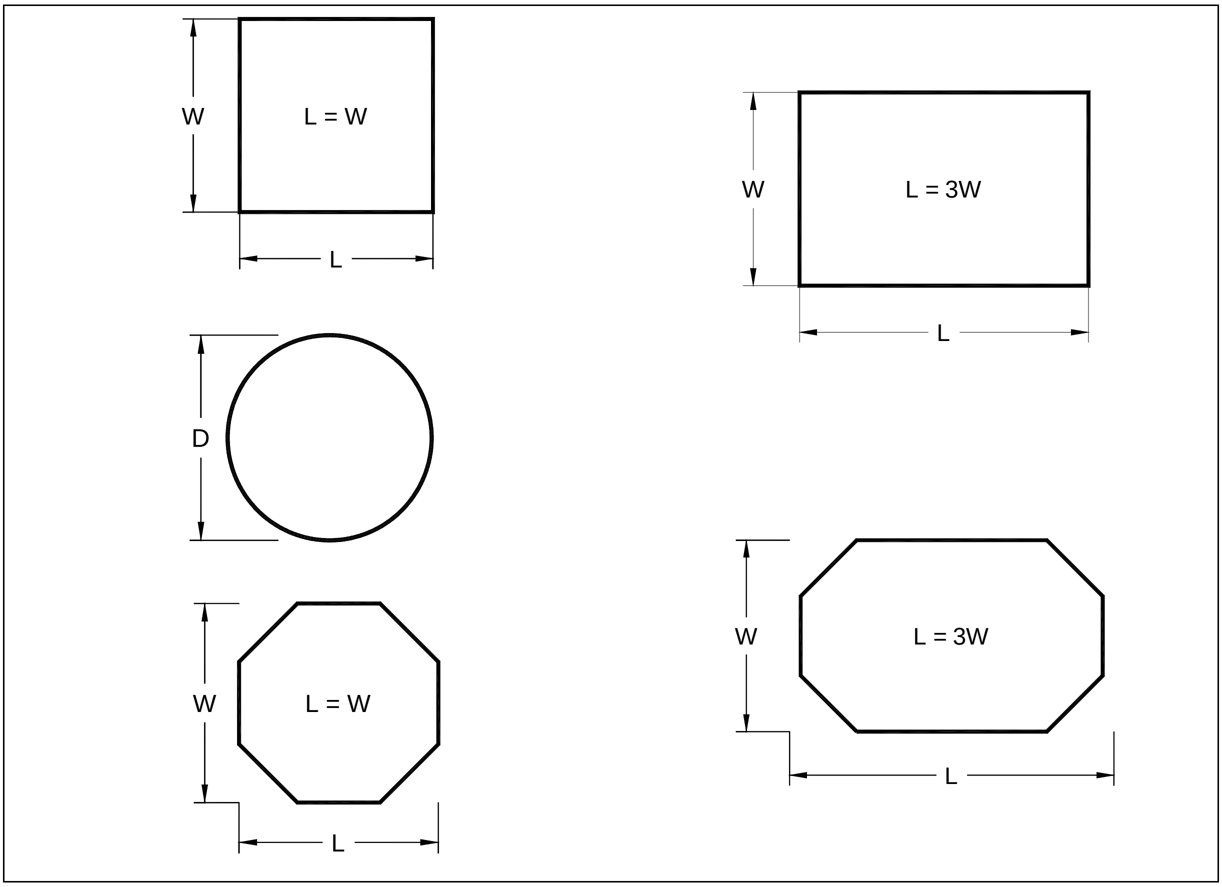
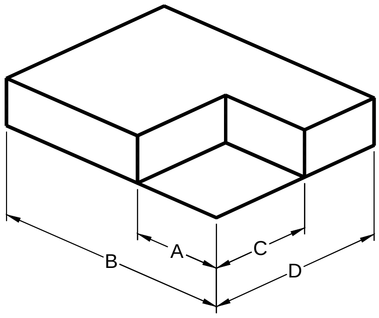
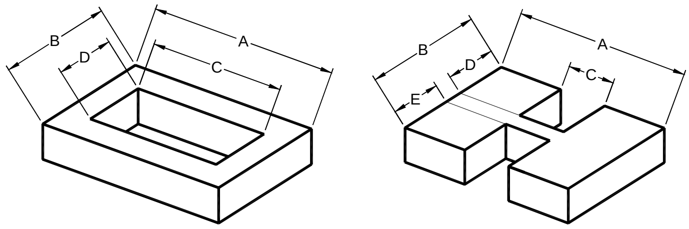
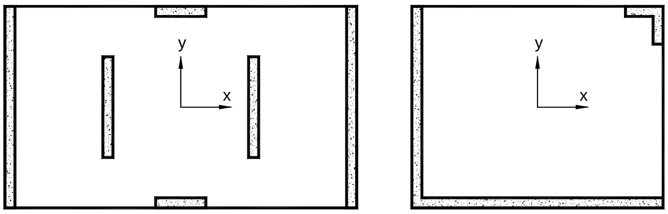
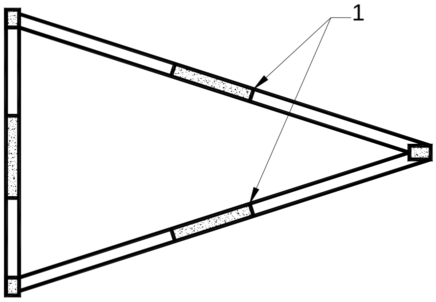


Figure 3 — Regular plan layouts

Table 18 — Common irregularities of building's architectural plan layout

No.	Plan irregularities	Description
I	Excessive recesses of corners: Any corner recess is considered excessive when the projections of the structure, to both sides of the recess, are greater than 15 percent of the plan dimension of the structure in the direction of the recess.	 A > 0.15B C > 0.15D
II	Diaphragm discontinuity: When the diaphragm has substantial discontinuities with areas larger than 50 % of the gross area, the structure is considered as irregular	 CxD < 0.5AxB (CxD+CxE) < 0.5AxB
III	Asymmetric or eccentric stiffness: When the lateral load resisting system in one direction has less than 20 % of the stiffness of the system in the other direction, or when the lateral load resisting system elements are asymmetrically distributed in plan, the structure is irregular.	
IV	Unparallel systems: When the plan projection of the vertical planes of the lateral load resisting system do not result in parallel lines, the structure is irregular	 **Key** 1 unparalel systems

10.4.2 Elevation irregularity

The architectural elevation layout of the building under study should be classified as regular or irregular. Shapes that are square, rectangular, trapezoidal, as well as any other shape that does not contain sudden changes in stiffness, may be considered as regular, as shown in Figure 4. Elevation irregularity can affect all building types. Damage is likely to occur due to stress concentration at the location of abrupt shape changes which, in turn, may represent sudden change in stiffness of the vertical elements of the lateral load resisting system.

Examples of elevation irregularity include buildings with flexible stories, irregular mass distribution between stories, façade recesses, displaced vertical plane of action or weak stories. Table 19 provides a guide as to which elevation shapes may be deemed irregular.

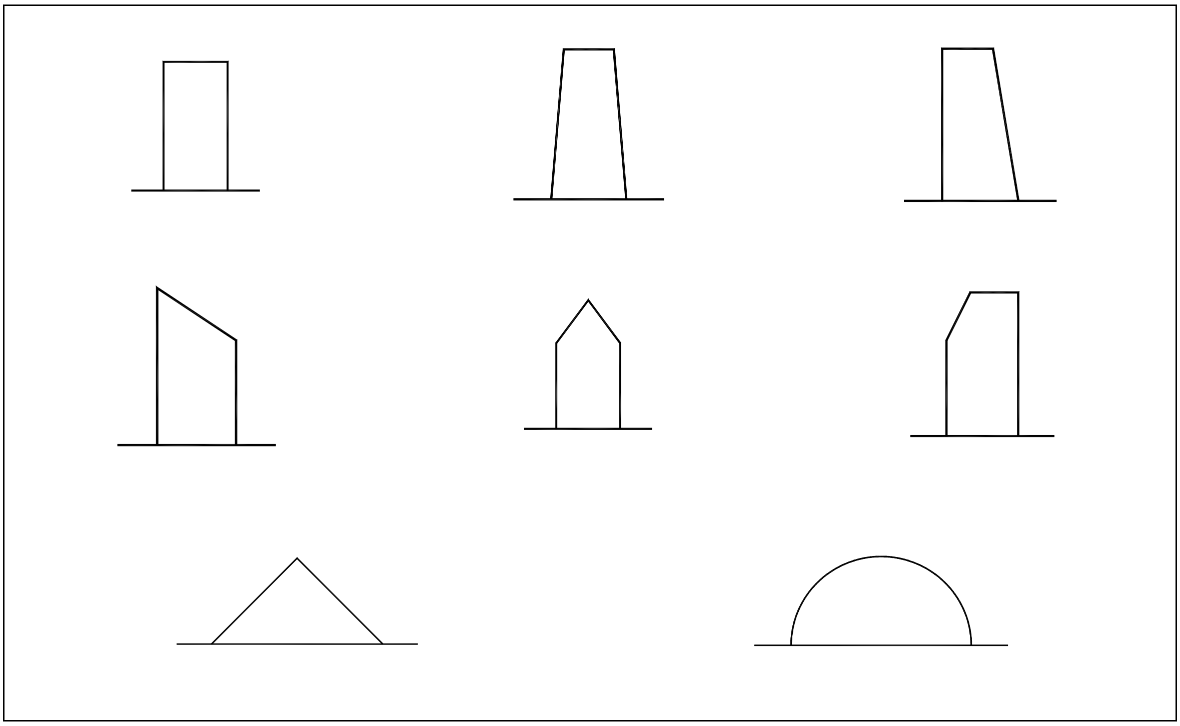


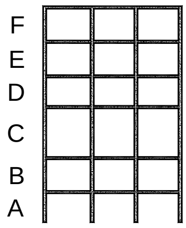
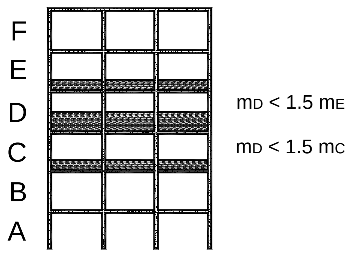
Figure 4 — Regular elevation layouts

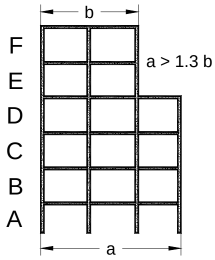
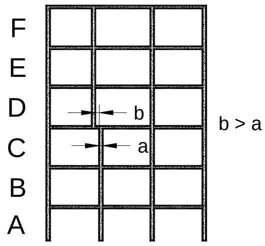
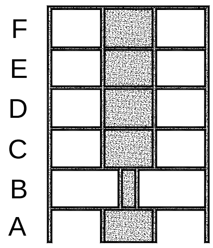
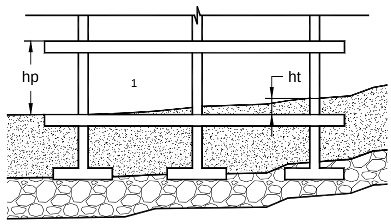
Table 19 — Common irregularities of building's architectural elevation layout

<table>
<tr><th>No.</th><th>Elevation irregularities</th><th>Description</th></tr>
<tr><td>V</td><td>Flexible Floor (Stiffness irregularity): When stiffness against a floor lateral forces is less than 70 % of the next floor stiffness or less than 80 % of the average stiffness of the next three floors, the structure is irregular. For the scope of these guidelines, story stiffness may be calculated as:

$\sum_{i=1}^{N} t_i \left(\frac{L_i}{h_i} \right)^3$ (t_i: thickness; L_i: length; h_i: height)</td><td>Stiffness $k_c < 0{,}7$ Stiffness k_D

Stiffness $k_c < 0{,}8\ (k_D+k_E+k_F)/3$

</td></tr>
<tr><td>VI</td><td>Mass distribution irregularity: When any floor mass, mi, is bigger than 1,5 times the mass of one of the nearest floors, the structure is irregular. In case of ceilings whom are lighter than the under floor is the only exception to this note</td><td></td></tr>
</table>

No.	Elevation irregularities	Description
VII	***Geometric irregularity:*** When the horizontal dimension of the seismic resistance system in any floor is larger than 1,3 times the same dimension in an adjacent floor, the structure is irregular, except for one story attics.	
VIII	***Displacements inside the plane of action:*** When there is displacement of the plane of action of the vertical seismic resistance system larger than one third the element horizontal dimension ($b > {}^{a}/_{3}$), the structure is irregular.	
IX	***Weak floor – Resistance discontinuity:*** When the floor resistance is less than 70 % of the next floor, the structure is irregular. The floor resistance may be calculated as the sum of the shear capacity of all structural vertical elements on the same floor.	 Floor Resistance < 0,7 C Floor Resistance
X	***Sloping terrain*** – If the terrain at ground level slopes in such a way that the difference in unrestrained height between opposite sides of the building exceeds 20 % of the story height, the structure is irregular, as the difference in restraint between sides of the building may cause global torsion and short column effects.	

According to the building classification, building's regularity should be qualified based on Table 20.

Table 20 — Building´s regularity qualification

Irregularity	Classification	Qualification
None	Regular	1
Only one irregularity in plan (I through IV) <u>or</u> one irregularity in elevation (VI through X)	Moderately irregular	2
More than one irregularity in plan (I through IV) <u>or</u> more than one irregularity in elevation (VI through X) <u>or</u> irregularities in both plan and elevation	Highly irregular	3

10.5 Foundation

The foundation system should be qualified according to Table 21

Table 21 — Foundation qualification

Foundation characteristics	Qualification
Structural concrete fully connected with grade beams	1
Structural concrete with some grade beams	2
Structural concrete with no grade beams	3
Plain concrete or unknown	4

10.6 Soil type

Soil type should be identified according the soil study, but is also necessary a visit to the place in order to observe physically the soil and make an objective classification as per Table 22.

Table 22 — Soil qualification

Soil type	Qualification
Hard soil or engineered fills: No settlements around building, no inclined trees or posts, passing trucks do not cause vibration, and, in general, when no significant cracks or damage are evident on walls and floors.	1
Medium bearing strength: Some settlement and vibration by passing trucks. Some cracks and damages in walls and floors.	2
Soft soil or non engineered fills: Settlement of surrounding terrain and relative settlement within building. Significant vibration due to passing trucks. Most neighboring constructions exhibit cracks and damages.	3
Loose sand with possibility of saturation	4

10.7 Quality aspects

The quality of the design and of both the materials and craftsmanship used in construction should be estimated and graded. Quality qualification may affect the overall building's vulnerability.

10.7.1 Quality of design

Design quality should be qualified in terms of the existence of a rational design and of the availability of earthquake resistant design standards at the time of said design, as per Table 23.

Table 23 — Quality of design

Design	Qualification
Design for lateral loads, current code	1
Design for lateral loads, previous codes	2
Design for vertical loads only	3
Unknown or inexistent	4

10.7.2 Quality of materials

Materials quality should be qualified in terms of the actual condition of the structure, as per Table 24

Table 24 — Quality of materials

Level of non seismic damage	Qualification
INSIGNIFICANT OR SLIGHT	1
MODERATE	2
SERIOUS	3
SEVERE	4

10.7.3 Quality of construction

Construction quality should be qualified in terms of workmanship and techniques used during construction, as per Table 25. Data on construction may be available from construction records or may be implied from the structure.

Table 25 — Quality of construction

Construction records	Qualification
Skilled workmanship and industrial techniques.	1
Skilled workmanship and no industrial techniques or unskilled workmanship and industrial tecniques.	2
Unskilled workmanship and no industrial techniques.	3
No records	4

10.7.4 Non structural elements

Loose or poorly connected nonstructural elements can pose life-safety hazards. Although these hazards may be present, the basic lateral-load system for the building may be adequate and require no further review.

Non structural elements vulnerability should be classified as per Table 26.

Table 26 — Non structural elements vulnerability qualification

Non structural element	Qualification
No masonry infills; no rigid fire system or gas system connections; no fragile finishes.	1
- Fragile finishes, such as mortar based plasters, tiles, stucco, etc. - Glass directly in contact with structure or otherwise unprotected from relative movement with respect to structure deformation	2
Masonry infill against structure	3
Rigid fire system or gas system connections;	4

10.8 Structural assessment

An evaluation must be made of the structural capacity of the structure under study. The structural assessment should comprise both aspects of resistance and flexibility of the structure shown in Annex A.

10.9 Final assessment

Final assessment of the structure must take into consideration all findings and qualifications established in chapters 9 or 10, whether the structure has been damaged by a seismic event or has not suffered damages but is being studied to classify its structural vulnerability.

In the first case, structures being assessed due to damages suffered by a seismic event, final assessment should take into consideration all findings and qualifications established by chapter 9. Final assessment should consist in the qualification of the reparability of the damaged elements and in the structure capacity compliance with required loading.

In the second case, final assessment should consist in the qualification of the susceptibility of the structure to suffer damages in the event of an earthquake and its value should be taken as the highest qualification obtained for each individual aspect studied, as per chapter 10.

Final assessment conclusions must be reported as per Table 27, for non damaged structures or as per damage assessment tables in chapter 9.

Table 27 — Final assessment of existing structure

CONDITION	CLASSIFICATION	VULNERABILITY QUALIFICATION
Plan or elevation irregularity	1	1
	2	2
	3	3
Foundation	1	2
	2	3
	3	3
	4	4
Soil	1	1
	2	1
	3	4
	4	4
Quality of design	1	1
	2	2
	3	4
	4	4
Quality of materials	1	1
	2	2
	3	3
	4	4
Quality of construction	1	1
	1	1
	1	2
	1	3
Non structural elements	1	1
	2	3
	3	4
	4	4
Resistance	r_R(1)< 1 for all elements	1
	r_R > 1 for secondary elements	3
	r_R < 1 for primary elements	4
Flexibility	Δ_S < limit	1
	Δ_S > limit	4

(1) : Maximum ratio r_R, obtained for any element, between internal required forces and the effective resistance of the element, for axial, shear, flexion and torsion forces.

Once final assessment is achieved, action to be taken shall be decided as per Table 28

Table 28 — Action for damaged or undamaged buildings depending on damage or vulnerability classification.

LEVEL	DAMAGE	VULNERABILITY	ACTION
1	Insignificant or slight	None	No particular action is required but owner may decide to proceed with repairs or rehabilitation
2	Moderate	Low	Repair or rehabilitation advisable. Warning to residents.
3	Serious	Moderate	Repair or rehabilitation must be conducted. Evacuation of residents may be needed. Temporary bracing may be placed in affected areas
4	Severe	High	Repair or rehabilitation must be conducted unless owner or authorities decide to demolish the structure. Residents must be evacuated. Temprorary bracing may be needed.

11 Rehabilitation analysis and design

Once structure assessment is completed the corresponding action must be taken according to Table 28.

If rehabilitation or repair is to take place, its analysis and design shall conform to the guidelines in this chapter.

11.1 Concrete Frame Systems

Frames that are cast monolithically, including monolithic concrete frames created by the addition of new material, shall meet the provisions of this section. Frames covered under this section include reinforced concrete beam-column moment frames, slab-column moment frames, and concrete frames with masonry in fills.

Concrete Frame Systems shall satisfy the following conditions:

1. framing components shall be beams (with or without slabs), columns, and their connections;

2. beams and columns shall be of monolithic construction that provides for moment transfer between beams and columns or slab and columns;

3. primary lateral load resisting elements shall be structural walls;

4. primary reinforcement in components contributing to lateral load resistance shall be nonprestressed, and

5. rehabilitation design of concrete frames assessed following these guidelines shall be developed as per ISO 15673.

11.2 Concrete wall systems

Monolithic reinforced concrete shear walls shall consist of vertical cast-in-place uncoupled elements. For the purpose of the use of these guidelines, only rectangular wall sections are allowed. These walls shall have continuous cross sections and reinforcement and shall provide mainly lateral force resistance. Walls with axial loads greater than 0,35 P_{CS} shall not be considered effective in resisting seismic forces. Walls shall be permitted to be considered as solid walls if they have openings that do not significantly influence the strength or inelastic behavior of the wall. Perforated walls shall be defined as walls having a regular pattern of openings in both horizontal and vertical directions that creates a series of pier and deep beam elements referred to as wall segments.

Coupling beams, and columns that support discontinuous shear walls are out of the scope of these guidelines.

Walls for use as lateral load resisting systems in the rehabilitation of structures assessed using these guidelines shall be designed as per ISO 15673.

11.3 Concrete frames with concrete infills

Concrete frames with infills are constructed in such a way that the infill and the concrete frame interact when subjected to vertical and lateral loads. When a concrete frame with masonry infils has been assessed using these guidelines, its rehabilitation should not take into account the contribution of the infills and the infills must be separated from the structure. Adequate anchoring of the separated infills must be designed to protect them from toppling down but in such a way that independence of in-plane movement is guaranteed between frame and in fills.

11.4 Foundation rehabilitation

11.4.1.1 Types of foundations

Foundations shall be defined as those elements that serve to transmit loads from the vertical structural subsystems (columns and walls) of a building to the supporting soil or rock. Concrete foundations for buildings shall be classified as either shallow or deep foundations. Requirements of this Section shall apply to shallow foundations that include spread or isolated footing, strip or line footing, combination footing, and concrete mat footing. Deep foundations are out of the scope of this document. The provisions of this Section shall apply to existing foundation elements and to new materials or elements that are required to rehabilitate an existing building.

Existing spread footings, strip footings, and combination footings are reinforced or unreinforced.

Vertical loads are transmitted by these footings to the soil by direct bearing; and lateral loads are transmitted by a combination of friction between the bottom of the footing and the soil, and passive pressure of the soil on the vertical face of the footing. Concrete mat footings shall be reinforced to resist the flexural and shear stresses resulting from the superimposed concentrated and line structural loads and the distributed resisting soil pressure under the footing. Lateral loads shall be resisted by friction between the soil and the bottom of the footing, and by passive pressure developed against foundation walls that are part of the system.

Design of foundation rehabilitation of structures assessed by the use of these guidelines, must be accomplished as per ISO 15673.

11.5 Rehabilitation Measures for the structural system

The decision to repair or replace a structure or its component can be taken only after consideration of likely service life of the structure and is established based on the technical and economic evaluations. Once a decision is taken to carry out the rehabilitation a proper technique and methodology should be developed.

11.5.1 Reinforced concrete jacketing

Increasing the confinement at the wall boundaries by the addition of a reinforced concrete jacket may be an effective measure in improving the flexural deformation capacity of a wall. The longitudinal jacket should not be continuous from story to story unless the jacket is also being used to increase the flexural capacity. The minimum thickness for a concrete jacket should be 7,5 cm.

Reinforced concrete jacketing increases the member size significantly. This has the advantage of increasing the member stiffness and is useful where deformations are to be controlled. If columns in a building are found to be slender, reinforced concrete jacketing provides a better solution for avoiding buckling problems. Design for strengthening/repair work is based on composite action between the old and new work. The new jacket should take the total load of the rehabilitated member when:

- old concrete has reached limiting strain and is not likely to sustain any more significant strain and

- old concrete is weak and porous and started deteriorating due to weathering action and corrosion of reinforcement.

Detailing must be properly designed to ensure transfer of load to the new jacket, if the old concrete fails. It is however, necessary to ensure perfect bond also between the old and new concrete by providing shear keys and effective bond coat with the use of epoxy or polymer modified cement slurry giving strength not less than that of the new concrete.

11.5.2 Shotcreting

Shotcrete is defined as pneumatically applied concrete or mortar placed directly on to a surface. The shotcrete shall be placed by either the dry mix or wet mix process.

The dry mix process shall consist of:

- thoroughly mixing the dry materials,

- feeding of these materials into mechanical feeder or gun,

- carrying the materials by compressed air through a hose to a special nozzle,

- introducing water at nozzle point and intimately mixing it with other ingredients at the nozzle, and

- jetting the mixture from the nozzle at high velocity on to the surface to receive the shotcrete.

The wet mix process shall consist of:

- thoroughly mixing all the ingredients with the exception of the accelerating admixture, if used;

- feeding the mixture into the delivery equipment;

- delivering the mixture by positive displacement or compressed air to the nozzle;

- jetting the mixture from the nozzle at high velocity on to the surface to receive the shotcrete, and

- if specified, fibers of steel, poly propylene or other material, as may be specified, could also be used together with the admixtures to modify the structural properties of the concrete being placed in position.

11.5.3 FRP reinforcements

This technique is a non-intrusive structural strengthening technique that increases the load carrying capacity (shear, flexural, compressive) and ductility of reinforced concrete members without causing any destruction or distress to the existing concrete. The design of the rehabilitation using this technique is out of the scope of these guidelines and out of the scope of ISO 15673, which specifies simplified design of reinforced concrete buildings; however, the use of FRP reinforcement may be permitted for buildings assessed using these guidelines, provided the design is conducted by a designer who is proficient and experienced in the design of FRP materials and elements and that the application of the FRP system is conducted by an experienced contractor, certified specifically for FRP systems application.

Enhancement in lateral drift ductility and horizontal shear carrying capacities of a concrete member can also be obtained by confinement of the member by this method. The flexural, shear and axial load carrying capacities of the structural members can be enhanced by appropriate orientation of primary fibers composites. The resulting cured membrane not only strengths the reinforced concrete member but also acts as an excellent barrier to corrosive agents, which are detrimental to concrete and its the reinforcement. Ingress of water, oxygen and carbon dioxide through the external surface of concrete member is prevented by the application of this kind of jacket.

11.5.3.1 CFRP jacketing

The use of carbon fiber sheets, epoxied to the concrete surface, should also be permitted to increase the shear capacity of a shear wall.

Carbon fiber wrap should be permitted for improving the confinement of concrete in compression.

11.5.3.2 GFRP jacketing

This system comprises of glass fiber wrapped over epoxy primer applied prepared surface of member requiring structural strengthening or surface protection. Subsequent to its wrapping, it is saturated with epoxy using rollers and stamping brushes manually to remove air bubbles, if any and left to cure ambient temperature. The subsequent later unidirectional fiber could be applied after giving the required overlap along the direction of fibers as per design requirements.

11.5.3.3 Steel jacketing

The design of the rehabilitation using steel jacketing is out of the scope of these guidelines and out of the scope of ISO 15673, which specifies simplified design of reinforced concrete buildings; however, the use of this technique may be permitted for buildings assessed using these guidelines, provided the design is conducted by a designer who is proficient and experienced in steel design and that the application of the steel jacketing system is conducted by an experienced contractor, certified specifically for steel construction.

When steel jacketing is used for the structure rehabilitation, the space between the jacket and the member must be filled with an stabilized mortar or a mortar made of resins. If post-tensioned fasteners are to be used, periodical maintenance of the structure should include the application of tension to loose elements due to steel relaxation or reinforced element deformations.

12 Rehabilitation construction

12.1 Demolitions and debris retrieval

Partial demolition can also be an effective corrective measure for irregularities, although this obviously has significant impact on the appearance and utility of the building, and this may not be an appropriate alternative for historic structures. Portions of the structure that create the irregularity, such as setback towers or side wings, can be removed. Expansion joints can be created to transform a single irregular building into multiple regular structures; however, care must be taken to avoid the potential problems associated with pounding.

Mass can be reduced through demolition of upper stories, replacement of heavy cladding and interior partitions, or removal of heavy storage and equipment loads.

12.2 Cover retrieval

Prior to preparation of concrete surfaces, exposed reinforcement should be inspected for access clearance, cross-sectional area and location. Reinforcing bars must be continued to completely expose the bar more than half of reinforcing bar perimeter has been exposed. For completely exposed reinforcing bars, a minimum average clearance of 25 mm or nominal maximum size of aggregate plus 5 mm, whichever is greater, must provided between the reinforcing bar and surrounding concrete.

12.3 Surface preparations

The general procedure in preparing concrete and reinforcement surfaces for optimum bonding is to sandblast the surfaces and the remove dust and debris by air blasting, low-pressure water blasting, or brooming. If the damage is due to corrosion, a suitable coating may be considered after removal of total rust from its surface to protect the exposed reinforcing steel. Final inspection of the prepared area including remedying any deficiencies should be completed just prior to batching the repair material.

12.4 Adherence concerns

In all kind of measures to develop the structure rehabilitation, the adherence is one of the most important subjects because it will avoid that the new material separates from the old one. The adherence can break because of volumetric changes during concrete setting.

The bond strength of repair with the substrate is essential to have a successful repair system. If it is felt that the bond strength of the repair material with the base material is inadequate or less than the strength of the base material, then some other suitable means could be explored to improve bond strength between repair material and substrate. These could be use of:

adhesive,

surface interlocking system and

mechanical bonding.

A variety of adhesives, in the range of epoxies, polymer modified cement slurries including unmodified polymer applications are available. The selection depends upon available open time for bonding.

12.5 Durability concerns

Durability is defined as the continued ability of the structure to withstand the expected wear and deterioration and perform satisfactorily in the normal operating conditions through out its intended life without the need for undue maintenance. What is implied is that the designer should expect certain degree of deterioration during the service life and provide required design inputs to adequately control it.

A simplified approach for this concern include design ensuring durability of construction facilities, addressing the issue under carbonation, chloride ingress, leaching, sulphate attack, alkali-silica reaction and frees thaw.

The requirements on durability are expressed in terms of minimum cement content, maximum water/cement ratio, minimum grade of concrete and minimum cover of reinforcement. These design parameters are related to specific exposure conditions. The general approach is to demand impermeability of concrete as the first line of defense against any of the deterioration process. The parameters mentioned above play a significant part in enhancing the durability, a comprehensive approach to design reinforced concrete structures for durability should give equal attention to the type and quality of component materials, the selection of mix proportions, the control of processing conditions. The design and detailing aspects should aim at minimizing the size and number of joints and cracks due to thermal gradients, drying shrinkage, creep and loading.

Annex A
(normative)

Structural Assessment

A.1 Resistance

The structure capacity to resist all loads should be evaluated as the maximum ratio r_R, obtained for any element, between internal required forces and the effective resistance of the element, for axial, shear, flexion and torsion forces.

The effective resistance is the calculated capacity of the element, affected by reduction factors in terms of design and construction quality, as defined in **Equation A.1**

$$R_{EFF} = \phi_Q \phi_C R_{CS} \qquad \text{Equation A.1}$$

Where:

R_{EFF}: *Effective resistance of an element.*

ϕ_Q: Reduction coefficient that depends on design and construction quality.

ϕ_C: Reduction coefficient that depends on condition of structure.

R_{CS}: Resistance or capacity of critical section of the element.

The values for ϕ_Q and ϕ_C are given in Table A.1.

Table A.1 — Values for ϕ_Q and ϕ_C

	Qualification	**ϕ_Q or ϕ_C**
Condition (10.2)	**Design or Construction (10.7.1 and 10.7.3)**	
0 or 1	1	[1,0]
2	2	[0,9]
3	3	[0,8]
4	4	[0,6]

The element strength or structural capacity, R_{CS}, of structural elements should be calculated based on as built conditions, as per A.1.1, A.1.2, A.1.3, B.1.4 and A.1.5. When no dimensional and reinforcement detailing is available, direct measurements must be conducted. Reinforcement may be estimated by magnetic detecting techniques or by physical exploration through the removal of small amount of concrete cover in key points. Both longitudinal and transverse reinforcement should be estimated.

A.1.1 Design strength for flexure only

Flexure strength of beam sections should be calculated using **Equation A.2** (see Figure A.1).

$$M_{cs} = \phi\left[\left(A_s - A_s'\right)\cdot f_y\left(d - \frac{a}{2}\right) + A_s' \cdot f_y \cdot \left(d - d'\right)\right] \text{ and } a = \frac{\left(A_s - A_s'\right)\cdot f_y}{0{,}85 \cdot f_c' \cdot b} \qquad \text{Equation A.2}$$

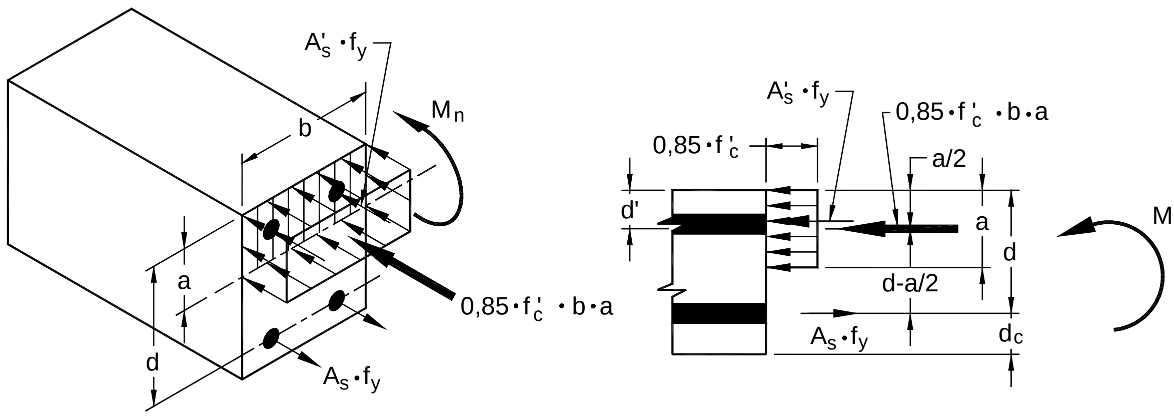


Figure A.1 — Flexural nominal moment strength for doubly reinforced sections

A.1.2 Design strength for axial compression

Equation A.3 and **Equation A.4** should be used to determine the design axial strength for axial compression without flexure, $\phi \cdot P_{0n}$.

For columns with ties and structural concrete walls

$$P_{CS} = 0.56 \cdot \left[0{,}85 \cdot f_c' \cdot \left(A_g - A_{st} \right) + A_{st} \cdot f_y \right] \qquad \text{Equation A.3}$$

For columns with spiral reinforcement:

$$P_{CS} = 0.64 \cdot \left[0{,}85 \cdot f_c' \cdot \left(A_g - A_{st} \right) + A_{st} \cdot f_y \right] \qquad \text{Equation A.4}$$

A.1.3 Balanced strength for axial compression with flexure

A.1.3.1 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 balanced design strength point should be determined using **Equation A.5** and **Equation A.6** respectively. However these equations only apply to rectangular columns with symmetrical reinforcement.

$$\phi \cdot P_{bn} = \phi \cdot 0{,}42 \cdot f_c' \cdot h \cdot b \qquad \text{Equation A.5}$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0{,}32 \cdot h + \phi \cdot \left[0{,}6 \cdot A_{se} + 0{,}15 \cdot A_{ss} \right] \cdot f_y \cdot \left(\frac{h}{2} - d' \right) \qquad \text{Equation A.6}$$

For Equation A.6 the total longitudinal reinforcement area, A_{st}, should be divided into extreme steel, A_{se}, and side steel, A_{ss}, in such a manner that $A_{se} + A_{ss} = A_{st}$. See Figure A.2. In Equation A.5 and Equation A.6, ϕ= [0,70]

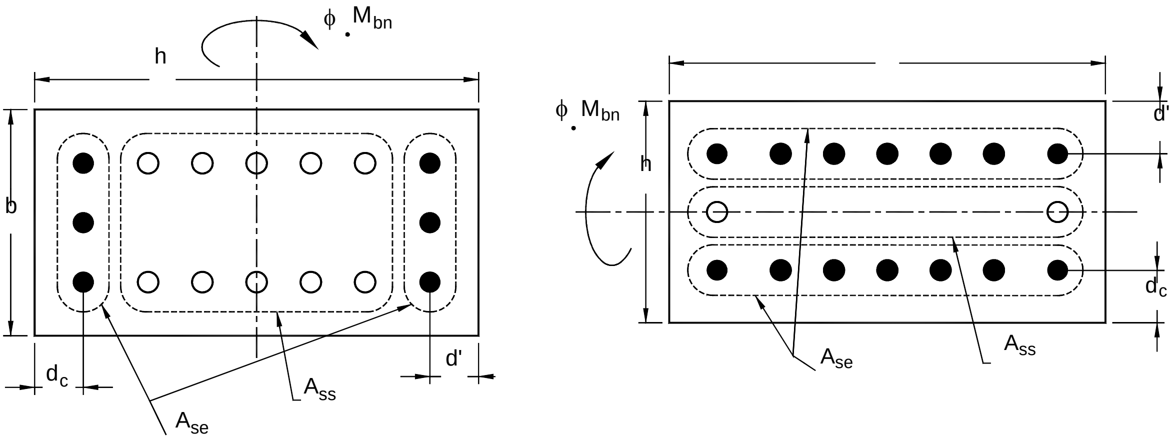


Figure A.2 — Dimensions for calculation of balanced moment design strength

A.1.3.2 Circular section columns with spiral reinforcement

The values for axial force, $\phi \cdot P_{bn}$, and moment, $\phi \cdot M_{bn}$, at the balanced design strength point should be determined using Equation A.7 and Equation A.8 respectively:

$$\phi \cdot P_{bn} = \phi \cdot 0{,}5 \cdot f_c' \cdot A_c \qquad \text{Equation A.7}$$

$$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0{,}2 \cdot h + \phi \cdot 0{,}6 \cdot A_{st} \cdot f_y \cdot \left(\frac{h}{2} - d'\right) \qquad \text{Equation A.8}$$

For Equation A.7, h should be taken as the diameter of the section of the column. In Equation A.7 and Equation A.8, ϕ = [0,75]

A.1.4 Design strength for axial tension without flexure

The design strength for axial tension without flexure, P_{tCS}, should be determined using Equation A.9:

$$P_{TCS} = \phi \cdot A_{st} \cdot f_y \qquad \text{Equation A.9}$$

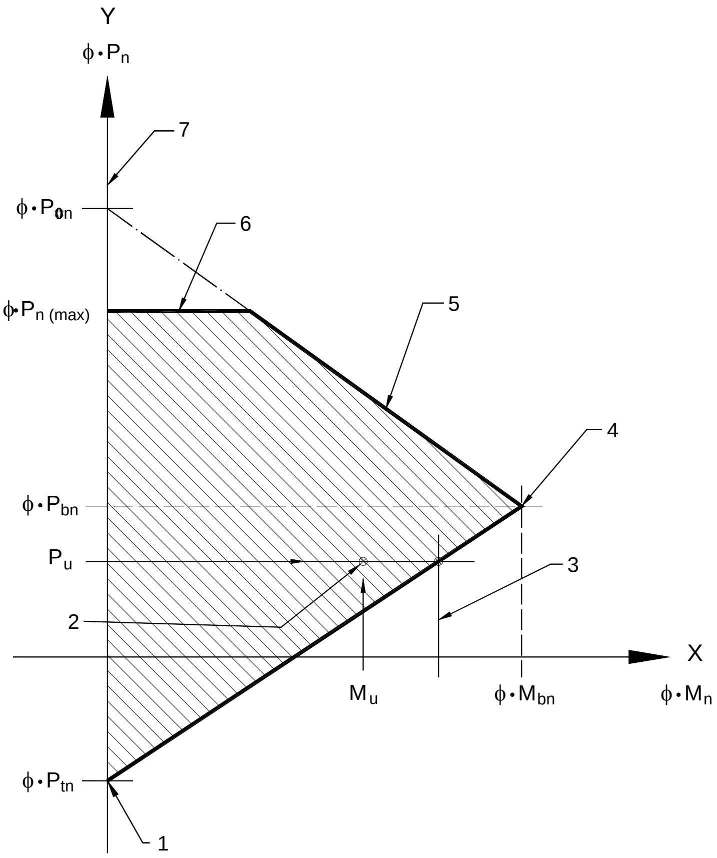
In Equation A.9, ϕ = [0,90]

A.1.5 Minimum design combined axial load and moment strength

The design moment strength at the section of the element, ($\phi \cdot M_n$), at the level of applied factored axial load, P_u, should be greater or equal than the greater required factored strength, M_u, that can accompany the factored axial load, P_u, as shown in Equation A.10.

$$\phi \cdot M_n \geq M_u \qquad \text{Equation A.10}$$

The compliance with Equation A.10 should be accomplished by proving that the coordinates of (M_u, P_u) in a moment vs. axial load interaction diagram relating $\phi \cdot M_n$ and $\phi \cdot P_n$, are inside the interaction design strength surface, shaded portion in Figure A.3.



Key

X Moment

Y Axial Load

1. Design strength for axial tension
2. Required factored axial load and moment
3. Design moment strength at factored axial load level, Pu
4. Balance design strength point
5. Interaction design strength surface
6. Maximum allowable axial compression load
7. Design strength for axial compression

Figure A.3 — Interaction diagram for (ϕ· Mn, ϕ· Pn)

The following conditions should be met for all couples of P_u and M_u that act on the column section:

$$P_u \le \phi \cdot P_{n(max)}$$ Equation A.11

$$P_u \ge -\left(\phi \cdot P_{tn}\right)$$ Equation A.12

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

$$M_u \le \phi \cdot M_n = \frac{\left(\phi \cdot P_{0n}\right) - P_u}{\left(\phi \cdot P_{0n}\right) - \left(\phi \cdot P_{bn}\right)} \cdot \left(\phi \cdot M_{bn}\right)$$ Equation A.13

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

$$M_u \leq \phi \cdot M_n = \frac{P_u + (\phi \cdot P_{tn})}{(\phi \cdot P_{bn}) + (\phi \cdot P_{tn})} \cdot (\phi \cdot M_{bn})$$ Equation A.14

A.2 Story drift

The structure’s story drift must be estimated to define the structure susceptibility to have excessive lateral story drift.

The story drift may be estimated as per Equation A.15.

$$\Delta_S = \frac{S_a F_a C_T^2 \sqrt{h_R}}{1.4 h_S}$$ Equation A.15

Where:

Δ_S : Story drift expressed as a percentage of story height.

S_a : Spectral acceleration corresponding to the period of the structure, T_a. The value for S_a should be obtained from the acceleration spectrum defined by the national code for the building site. If no national code exists, S_a may be estimated as follows:

$S_a = 2.5 A_a F_a$ for $T_a \leq 0.5$

$S_a = 1.2 A_a F_a S$ for $T_a > 0.5$

Where S is defined in Table A.3.

$T_a = 1.2 C_T h_R^{0.75}$

C_T : Coefficient for approximate estimation of period, which value depends on the structural system, as per Table A.2.

h_R : Height of the roof above ground level.

h_S : Height of story.

The story drift equation was developed as follows:

In order to estimate the story drift without requiring specific calculation a common assumption is to estimate a building’s structure fundamental natural vibration period based on the total height of the structure and on two factors accounting for stiffness and the shape of force distribution along the height, respectively. Thus:

$$T = 1{,}2 \cdot C_r \cdot h^{3/4}$$

From here, angular frequency may be inferred as:

$$\omega = \frac{2\pi}{T} = \frac{2\pi}{1{,}2 \cdot C_r \cdot h_R{}^{3/2}}$$

$$\omega^2 = \frac{4\pi^2}{(1{,}2)^2 \cdot (C_r)^2 \cdot h_R{}^{3/2}}$$

With this in mind, the expected spectral displacement for a single degree of freedom (SDOF) system may be obtained as:

$$Sd^{MDOF} = \frac{Sa}{\omega^2} = \frac{Sa \cdot (1{,}2)^2 \cdot (C_r)^2 \cdot h_R{}^{3/2}}{4\pi^2}$$

Now, to extrapolate this value to a various degrees of freedom (VDOF) system, it should be remembered that a reasonable height for the ***equivalent SDOF*** system with all the mass of the real structure being represented is 0.75 of the total height of the structure. Therefore:

$$Sd^{MDOF} = 1{,}33 \cdot Sd^{MDOF} = \frac{1{,}33 \cdot Sa \cdot (1{,}2)^2 \cdot (C_r)^2 \cdot h_R{}^{3/2}}{4\pi^2}$$

It is also common to express the average drift as the total lateral slope, i.e.:

$$\Delta_{average} = \frac{Sd^{MDOF} \cdot g}{h_R} = \frac{1{,}33 \cdot Sa \cdot (1{,}2)^2 \cdot (C_r)^2 \cdot h_R{}^{1/2} \cdot g}{4\pi^2}$$

A reasonable value for the maximum drift is:

$$\Delta_{max} = 1{,}5\ \Delta_{average} = \frac{1{,}5 \cdot 1{,}33 \cdot Sa \cdot (1{,}2)^2 \cdot (C_r)^2 \cdot h_R{}^{1/2} \cdot g}{1{,}4}$$

Thus,

$$\Delta_{max} = \frac{Sa \cdot C_r^2 \cdot h_R{}^{1/2}}{1{,}4}$$

Thereby, the drift story is determined as follows:

$$\Delta_s = \frac{\Delta_{max}}{h_s} = \frac{Sa \cdot C_r^2 \cdot \sqrt{h_R}}{1{,}4 \cdot h_s}$$

And including the soil factor F_a, the drift story is then:

$$\Delta_s = \frac{\Delta_{max}}{h_s} = \frac{Sa \cdot F_a \cdot C_r^2 \cdot \sqrt{h_R}}{1{,}4 \cdot h_s}$$

Table A.2 — Values for C_T

<table>
<tr><th colspan="2">SYSTEM</th><th>C_T</th><th>Drift limit</th></tr>
<tr><td colspan="2">FRAMES</td><td>[0,080]</td><td>[1%]</td></tr>
<tr><td colspan="2">WALLS</td><td>[0,050]</td><td>[0,5%]</td></tr>
<tr><td rowspan="3">DUAL</td><td>Walls total plan area < [1%]</td><td>[1%]</td><td>[1%]</td></tr>
<tr><td>Walls total plan area [1% - 2%]</td><td>[0,70]</td><td>[0,7%]</td></tr>
<tr><td>Walls total plan area > 2%</td><td>[0,50]</td><td>[0,5%]</td></tr>
</table>

Table A.3 — Values of soil coefficient, F_a

Soil qualification	F_a
1	1,0
2	1,5
3	2,0
4	Use not permitted under these guidelines

The calculated drift must not exceed the limiting values specified in Table A.3.

A.3 Energy dissipation level

Structure components shall according to the maximum value of their energy dissipation capacity ratio. This ratio is defined for the whole structure through a parameter R_0. For the purpose of these guidelines, the structure dissipation capacity is classified in three categories: Minimum dissipation capacity (MDC), intermediate dissipation capacity (IDC) and high dissipation capacity (HDC). Table A.4 presents the ranges of energy dissipation according to the value of the parameter R_0.

Table A.4 — Elements classification according to R

R	Energy dissipation Level
< 2	Low
2 to 4	Intermediate
> 4	High

A.3.1 Required energy dissipation level

The energy dissipation level of a reinforced concrete element depends, among other things, on the reinforcement detail, specifically on the transverse reinforcement configuration, as schematically shown in Figure A.4.

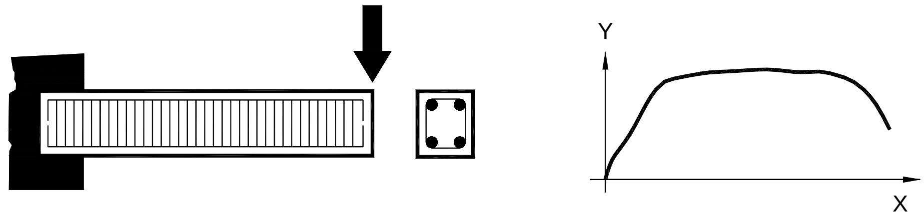


Figure A.4a — HDC High Capacity of energy dissipation

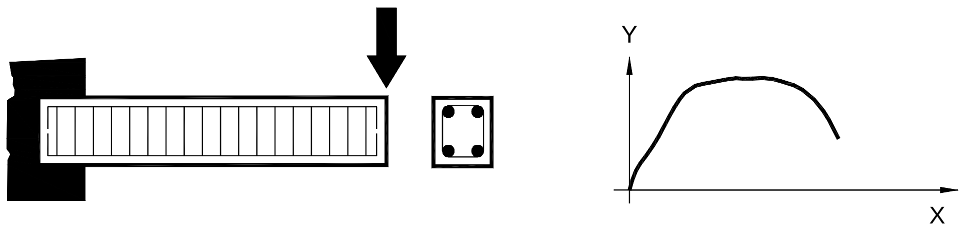


Figure A.4b — IDC Intermediated Capacity of energy dissipation

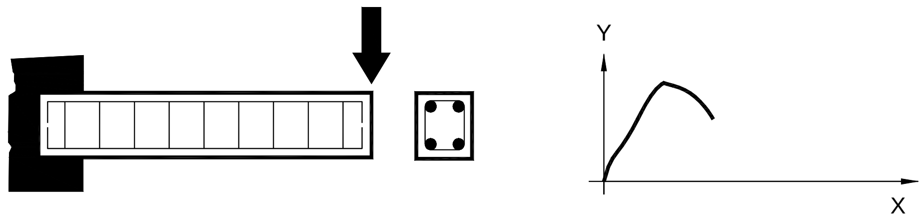


Figure A.4c — MDC Minimum Capacity of energy dissipation

Key:

X. Deflection

Y: Strength

Figure A.4 — Energy Dissipation Capacity Level in the inelastic range

R_0 must be assigned according to the structural system classification, as per Chapter 8 and to the construction and design requirements of the original structure, an energy dissipation coefficient, R_0, must be assigned based on the structural characteristics of the system

The value for R_0 should be determined based on all the information collected, according to chapter 6 from the original design, such as drawings and memories, and on the criterion of the designer conducting the structural assessment, to the best of her or his knowledge.

The selection of the dissipation capacity level at which a structure should respond under seismic loading is a function of the seismic hazard at the site. Table **A.5** shows the energy dissipation levels permitted for use under these guidelines, according to seismic hazard at the site of the building.

Table A.5 — Energy Dissipation Capacity Levels for each seismic hazard region

ENERGY DISSIPATION CAPACITY	SEISMIC HAZARD			
	NO HAZARD	LOW	INTERMEDIATE	HIGH
MINIMUM (MDC)	Acceptable	Acceptable	Not Acceptable	Not Acceptable
INTERMEDIATE (IDC)	Acceptable	Acceptable	Acceptable	Not Acceptable
HIGH (HDC)	Acceptable	Acceptable	Acceptable	Acceptable

This value must be in agreement with the requirements for the material and the structural system specified by these guidelines in Table A.6, Table A.7, Table A.8. Interpolation may be used for systems not lying specifically in one of the structural classification for which R is defined. I no event, the value for R may be larger than the largest R value specified in these guidelines for similar structural systems.

If the collected information is incomplete or there remain unsolved doubts about the reinforcing details of an existing structure, a value of R may be chosen equal to 75 % of the R value for that type of structural system specified in Table A.6, Table A.7, Table A.8.

Table A.6 — Values for Ro on Frame Systems

A. FRAME SYSTEM		R_0
Lateral loads Resisting System	Vertical loads Resisting System	
1. Moment resisting frames		
a. Concrete (HDC)	The same one	7,0
b. Concrete (IDC)	The same one	5,0
c. Concrete (DMI)	The same one	2,5
4. Slab-column Frames (includes reticular cell)		
a. Concrete	The same one	2,5
(IDC)		
b. Concrete (MDC)	The same one	1,5
5. Inverted pendulum Structures		
b. Concrete frames (HDC)	The same one	2,5

Table A.7 — Values for Ro on Wall Systems

B. WALL SYSTEM		R_0
Lateral loads Resisting System	Vertical loads Resisting System	
2. Structural Walls		
a. Concrete walls with special capacity of energy dissipation (HDC)	The same one	7,0
b. Concrete walls with moderated capacity of energy dissipation (IDC)	The same one	4,5
c. Concrete walls with minimum capacity of energy dissipation	The same one	2,5
3. Frames with diagonals (diagonals carry up vertical load)		
b. Frames with concrete diagonals	The same one	3,5
(IDC)		

Table A.8 — Values for Ro on Dual Systems

C. DUAL SYSTEM		R_0
Lateral loads Resisting System	Vertical loads Resisting System	
1. Structural Walls		
a. Concrete walls (HDC)	Concrete frames (HDC)	8,0
b. Concrete walls (HDC)	Moment resisting steel frames (HDC)	8,0
c. Concrete walls	Concrete frames	6,0
(IDC)	(IDC)	
d. Concrete walls	Moment resisting steel frames	6,0
(IDC)	(IDC)	
3. Frames with concentrical diagonals		
d. Concrete IDC)	Concrete frames (IDC)	4,0

When the structure is classified as irregular, the value of the Energy dissipation capacity coefficient R_0 must be reduced by a factor ϕ_P, for plan irregularities, and by a factor ϕ_E for the elevation irregularities, as indicates Equation A.16:

$R = R_0 \cdot \phi_P \cdot \phi_E$ Equation A.16

Where:

R_0 : Basic coefficient of energy dissipation capacity.

The values of ϕ_P and ϕ_E are obtained from Table A.9.

Table A.9 — Values for R_0 reduction factors.

Irregularity	ϕ_P or ϕ_E
I, II, IV, V, VI, VII	0,9
III, VIII, IX, X	0,8

When the buildings has been classified as irregular with more than one plan irregularity or more than one elevation irregularity, the corresponding ϕ_P or ϕ_E should be taken as the largest value for each case.

When assessing the vulnerability of an existing structure, element capacities are compared with required strengths. The required strength calculated under these guidelines is the elastic required strength, while the capacity is calculated at yielding. Therefore, required elastic strengths should be divided by R, according to the energy dissipation level of the structure before comparing these values with capacities.

A.4 Equivalent equations 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 appendix 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:

$$f_{yd} = \frac{f_y}{\gamma_{ms}} \qquad \text{(A-17)}$$

$$f_{cd} = \frac{f'_c}{\gamma_{mc}} \qquad \text{(A-18)}$$

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

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

Equation	In terms of ϕ factors	In terms of material factors
(2)	$\phi \cdot R_n \geq \gamma_1 \cdot S_1 + \gamma_2 \cdot S_2 + \cdots$	$R = f\left(\frac{f_{cd}}{\gamma_{mc}}, \frac{f_{yd}}{\gamma_{ms}}\right) \geq \gamma_1 \cdot S_1 + \gamma_2 \cdot S_2 + \cdots$
(4)	ϕ · (Nominal Strength) ≥ U	$R = f\left(\frac{f_{cd}}{\gamma_{mc}}, \frac{f_{yd}}{\gamma_{ms}}\right) \geq U$
(24)	$\phi \cdot M_n = \phi \cdot A_s \cdot f_y\left(d - \frac{a}{2}\right)$	$M_r = A_s \cdot f_{yd}\left(d - \frac{a}{2}\right)$
(25)	$a = \frac{A_s \cdot f_y}{0{,}85 \cdot f'_c \cdot b}$	$a = \frac{A_s \cdot f_{yd}}{0{,}85 \cdot f_{cd} \cdot b}$
(26)	$\phi \cdot M_n = \phi \cdot A_s \cdot f_y \cdot d \cdot \left(1 - 0{,}59 \cdot \frac{A_s \cdot f_y}{b \cdot d \cdot f'_c}\right)$	$M_r = A_s \cdot f_{yd} \cdot d\left(1 - 0{,}59 \cdot \frac{A_s \cdot f_{yd}}{b \cdot d \cdot f_{cd}}\right)$
(27)	$\phi \cdot M_n \approx \phi \cdot A_s \cdot f_y \cdot 0.85 \cdot d$	$M_r \approx A_s \cdot f_{yd} \cdot 0.85 \cdot d$
(28)	$\rho = \frac{A_s}{b \cdot d} = \alpha - \sqrt{\alpha^2 - \left(\frac{M_u}{\phi \cdot b \cdot d^2} \cdot \frac{2 \cdot \alpha}{f_y}\right)}$ where $\alpha = \frac{f'_c}{1{,}18 \cdot f_y}$	$\rho = \frac{A_s}{b \cdot d} = \alpha - \sqrt{\alpha^2 - \left(\frac{M_u}{b \cdot d^2} \cdot \frac{2 \cdot \alpha}{f_{yd}}\right)}$ where $\alpha = \frac{f_{cd}}{1{,}18 \cdot f_{yd}}$
(29)	$\rho = \frac{A_s}{b \cdot d} \approx \frac{M_u}{\phi \cdot b \cdot d^2 \cdot 0{,}85 \cdot f_y}$	$\rho = \frac{A_s}{b \cdot d} \approx \frac{M_u}{b \cdot d^2 \cdot 0{,}85 \cdot f_{yd}}$
(30)	$\phi \cdot M_n = \phi \cdot \left[\left(A_s - A'_s\right) \cdot f_y\left(d - \frac{a}{2}\right) + A'_s \cdot f_y \cdot \left(d - d'\right)\right]$	$M_r = \left(A_s - A'_s\right) \cdot f_{yd}\left(d - \frac{a}{2}\right) + A'_s \cdot f_{yd} \cdot \left(d - d'\right)$
(31)	$a = \frac{\left(A_s - A'_s\right) \cdot f_y}{0{,}85 \cdot f'_c \cdot b}$	$a = \frac{\left(A_s - A'_s\right) \cdot f_{yd}}{0{,}85 \cdot f_{cd} \cdot b}$
(32)	$A'_s = \frac{M_u}{\phi \cdot f_y \cdot \left(d - d'\right)} - \left[b \cdot d^2 \cdot \rho_{max} \cdot f_y \cdot 0{,}8\right]$	$A'_s = \frac{M_u}{f_{yd} \cdot \left(d - d'\right)} - \left[b \cdot d^2 \cdot \rho_{max} \cdot f_{yd} \cdot 0{,}8\right]$
(34)	$h_f \geq a$ and $a = \frac{A_s \cdot f_y}{0{,}85 \cdot f'_c \cdot b}$	$h_f \geq a$ and $a = \frac{A_s \cdot f_{yd}}{0{,}85 \cdot f_{cd} \cdot b}$

Equation	In terms of ϕ factors	In terms of material factors
(35)	$\rho \le \frac{0{,}85 \cdot f'_c \cdot h_f}{f_y \cdot d}$	$\rho \le \frac{0{,}85 \cdot f_{cd} \cdot h_f}{f_{yd} \cdot d}$
(36)	$\phi \cdot P_{0n} = \phi \cdot \left[0{,}85 \cdot f'_c \cdot \left(A_g - A_{st}\right) + A_{st} \cdot f_y\right]$	$P_{r0} = 0{,}85 \cdot f_{cd} \cdot \left(A_g - A_{st}\right) + A_{st} \cdot f_{yd}$
(37)	$\phi \cdot P_{n(max)} \le 0{,}80 \cdot \phi \cdot P_{0n}$	$P_{r(max)} \le 0{,}80 \cdot P_{r0}$
(38)	$\phi \cdot P_{n(max)} \le 0{,}85 \cdot \phi \cdot P_{0n}$	$P_{r(max)} \le 0{,}85 \cdot P_{r0}$
(39)	$\phi \cdot P_{bn} = \phi \cdot 0{,}42 \cdot f'_c \cdot h \cdot b$	$P_{br} = 0{,}44 \cdot f_{cd} \cdot h \cdot b$
(40)	$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0{,}32 \cdot h +$ $\phi \cdot \left[0{,}6 \cdot A_{se} + 0{,}15 \cdot A_{ss}\right] \cdot f_y \cdot \left(\frac{h}{2} - d'\right)$	$M_{br} = P_{bn} \cdot 0{,}32 \cdot h +$ $\left[0{,}6 \cdot A_{se} + 0{,}15 \cdot A_{ss}\right] \cdot f_{yd} \cdot \left(\frac{h}{2} - d'\right)$
(41)	$\phi \cdot P_{bn} = \phi \cdot 0{,}5 \cdot f'_c \cdot A_c$	$P_{br} = 0{,}52 \cdot f_{cd} \cdot A_c$
(42)	$\phi \cdot M_{bn} = \phi \cdot P_{bn} \cdot 0{,}2 \cdot h + \phi \cdot 0{,}6 \cdot A_{st} \cdot f_y \cdot \left(\frac{h}{2} - d'\right)$	$M_{br} = P_{br} \cdot 0{,}2 \cdot h + 0{,}48 \cdot A_{st} \cdot f_{yd} \cdot \left(\frac{h}{2} - d'\right)$
(43)	$\phi \cdot P_{tn} = \phi \cdot A_{st} \cdot f_y$	$P_{tr} = A_{st} \cdot f_{yd}$
(44)	$\phi \cdot M_n \ge M_u$	$M_r \ge M_u$
(45)	$P_u \le \phi \cdot P_{n(max)}$	$P_u \le P_{r(max)}$
(46)	$P_u \ge -\left(\phi \cdot P_{tn}\right)$	$P_u \ge -\left(P_{tr}\right)$
(47)	$M_u \le \phi \cdot M_n = \frac{\left(\phi \cdot P_{0n}\right) - P_u}{\left(\phi \cdot P_{0n}\right) - \left(\phi \cdot P_{bn}\right)} \cdot \left(\phi \cdot M_{bn}\right)$	$M_u \le M_n = \frac{\left(P_{r0}\right) - P_u}{\left(P_{r0}\right) - \left(P_{br}\right)} \cdot \left(M_{br}\right)$
(48)	$M_u \le \phi \cdot M_n = \frac{P_u + \left(\phi \cdot P_{tn}\right)}{\left(\phi \cdot P_{bn}\right) + \left(\phi \cdot P_{tn}\right)} \cdot \left(\phi \cdot M_{bn}\right)$	$M_u \le M_n = \frac{P_u + \left(P_{tr}\right)}{\left(P_{br}\right) + \left(P_{tr}\right)} \cdot \left(M_{br}\right)$
(49)	$\frac{\left(M_u\right)_x}{\left(\phi \cdot M_n\right)_x} + \frac{\left(M_u\right)_y}{\left(\phi \cdot M_n\right)_y} \le 1{,}0$	$\frac{\left(M_u\right)_x}{\left(M_r\right)_x} + \frac{\left(M_u\right)_y}{\left(M_r\right)_y} \le 1{,}0$
(50)	$\phi \cdot V_n \ge V_u$	$V_r \ge V_u$
(51)	$\phi \cdot V_n = \phi \cdot \left(V_c + V_s\right)$	$V_r = V_c + V_s$
(52)	$\phi \cdot V_c = \phi \cdot 2 \cdot \left[\frac{\sqrt{f'_c}}{6}\right] \cdot b_w \cdot d$	$V_c \le 0.60\ f_{ctd} \cdot b_w \cdot d$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right) / \gamma_{mc}$
(53)	$\phi \cdot V_s = \phi \cdot \left[\frac{A_v \cdot f_{ys} \cdot d}{s}\right]$	$V_s = \frac{A_v \times f_{ys} \times d}{1.15 \cdot s}$
(54)	$\phi \cdot V_s \le \phi \cdot \left[\frac{2}{3} \cdot \sqrt{f'_c} \cdot b_w \cdot d\right] = 4 \cdot \phi \cdot V_c$	$V_s \le 0.20\ f_{cd} \cdot b_w \cdot d$
(55)	$A_v = \frac{1}{16}\sqrt{f'_c}\,\frac{b_w \cdot s}{f_{ys}} \ge \frac{b_w \cdot s}{3 \cdot f_{ys}}$	$A_v = 0.30 \frac{f_{ctd}}{f_{yds}} b_w\, s$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right) / \gamma_{mc}$ $f_{ctd} = \left(0.35\sqrt{f_{ck}}\right) / \gamma_{mc}$

Equation	In terms of ϕ factors		In terms of material factors	
Table 6 $V_c > V_u \ge \frac{V_c}{2}$ $\left(\begin{array}{l}(\phi V_c) > V_u \\ \ge \frac{(\phi V_c)}{2}\end{array}\right)$		$A_v = \frac{1}{16}\sqrt{f'_c}\,\frac{b_w \cdot s}{f_{ys}} \ge \frac{b_w \cdot s}{3 \cdot f_{ys}}$	$A_v = 0.30\,\frac{f_{ctd}}{f_{yds}} b_w\, s$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\,\gamma_{mc}$	
Table 6 $V_u \ge V_c$ $[V_u \ge (\phi V_c)]$	$2 \cdot \phi \cdot V_c > \phi \cdot V_s$	$A_v = \frac{(V_u - \phi \cdot V_c) \cdot s}{\phi \cdot f_{ys} \cdot d}$	$2 \cdot V_c > V_s$	$A_v = \frac{1.15(V_u - V_c) \cdot s}{\phi \cdot f_{ysd} \cdot d}$
	$4 \cdot \phi \cdot V_c > \phi \cdot V_s \ge 2 \cdot \phi \cdot V_c$	$A_v = \frac{(V_u - \phi \cdot V_c) \cdot s}{\phi \cdot f_{ys} \cdot d}$	$4 \cdot V_c > V_s \ge 2 \cdot V_c$	$A_v = \frac{1.15(V_u - V_c) \cdot s}{\phi \cdot f_{ysd} \cdot d}$
	$\phi \cdot V_s \ge 4 \cdot \phi \cdot V_c$	not permitted	$V_s \ge 4 \cdot V_c$	not permitted
(56)	$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[1 + \frac{2}{\beta_c}\right] \cdot \left[\frac{\sqrt{f'_c}}{6}\right] \cdot b_0 \cdot d$		$V_r = 0.60\left[1 + \frac{2}{\beta_c}\right] f_{ctd}\, b_o\, d$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\,\gamma_{mc}$	
(57)	$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[2 + \frac{\alpha_s \cdot d}{b_0}\right] \cdot \left[\frac{\sqrt{f'_c}}{12}\right] \cdot b_0 \cdot d$		$V_r = 0.3\left(2 + \frac{\alpha_s\, d}{b_o}\right) f_{ctd}\, b_o\, d$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\,\gamma_{mc}$	
(58)	$\phi \cdot V_n = \phi \cdot V_c = \phi \cdot \left[\frac{\sqrt{f'_c}}{3}\right] \cdot b_0 \cdot d$		$V_r = 1.20\, f_{ctd}\, b_o\, d$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\,\gamma_{mc}$	
(59)	$\phi V_n = \phi(V_c + V_s)$		$V_r = V_c + V_s$	
(60)	$\phi \cdot V_c = \phi \cdot \left[\frac{\sqrt{f'_c}}{6}\right] \cdot b_w \cdot l_w$		$V_r = 0.6\, f_{ctd}\, b_w\, I_w$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\,\gamma_{mc}$	
(61)	$\phi \cdot V_s = \phi \cdot \lfloor \rho_h \cdot f_y \cdot b_w \cdot l_w \rfloor$		$V_s = \rho_h\, f_{yd}\, b_w\, \ell_w$	
(62)	$\rho_h \ge \frac{V_u - \phi \cdot V_c}{\phi \cdot f_y \cdot b_w \cdot l_w}$		$\rho_h \ge \frac{V_u - V_c}{f_{yd} b_w \ell_w}$	
(63)	$\phi \cdot V_n = \phi \cdot (V_c + V_s) \le \phi \cdot \left[\frac{5}{6}\right] \cdot \sqrt{f'_c} \cdot b_w \cdot l_w$		$V_r = (V_c - V_s) \le 3.0\, f_{ctd}\, b_w\, \ell_w$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\,\gamma_{mc}$	
(64)	$T_u \le \phi \cdot \left[\frac{\sqrt{f'_c}}{24}\right] \cdot \left[\frac{h^2 \cdot b^2}{h + b}\right]$		$T_u \le 0.15\, f_{ctd}\, \frac{h^2\, b}{h + b}$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\,\gamma_{mc}$	
(65)	$\phi \cdot P_n = \phi \cdot 0{,}85 \cdot f'_c \cdot A_c$		$P_r = 0.9\, f_{cd}\, A_c$	
(88)	$d \ge \frac{3 \cdot q_u \cdot \alpha_a \cdot l_a}{\phi \cdot \sqrt{f'_c}}$		$d \ge 0.8\,\frac{q_u\, \alpha_a\, \ell_a}{f_{ctd}}$	

Equation	In terms of ϕ factors	In terms of material factors
		where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\gamma_{mc}$
(89)	$d \geq \frac{3 \cdot q_u \cdot \alpha_b \cdot l_b}{\phi \cdot \sqrt{f'_c}}$	$d \geq 0.8 \frac{q_u\, \alpha_b\, \ell_b}{f_{ctd}}$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\gamma_{mc}$
(90)	$d \geq \frac{3 \cdot q_u \cdot l_a}{2 \cdot \phi \cdot \sqrt{f'_c}}$	$d \geq 0.4 \frac{q_u\, l_a}{f_{ctd}}$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\gamma_{mc}$
7.6.4.5.4 a	$\phi \cdot \frac{\sqrt{f'_c}}{4} \cdot b_w \cdot d$	$\left(0.90\, f_{ctd}\, b_w\, d\right)$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\gamma_{mc}$
(110)	$A_i \geq \frac{\left(1 - \frac{h_b}{h_s}\right) V_u}{\phi \cdot f_y}$	$A_i \geq \frac{\left(1 - {h_b}/{h_f}\right) V_u}{f_{yd}}$
(136)	$\sqrt{\left[\frac{(V_u)_x}{(\phi \cdot V_n)_x}\right]^2 + \left[\frac{(V_u)_y}{(\phi \cdot V_n)_y}\right]^2} \leq 1{,}0$	$\sqrt{\left(\frac{V_{ux}}{V_{rx}}\right)^2 + \left(\frac{V_{uy}}{V_{ry}}\right)^2} \leq 1.0$
(137)	$\sum \left(l_w \cdot b_w\right) \geq \frac{V_u}{\frac{1}{9} \cdot \sqrt{f'_c}}$	$\Sigma\left(\ell_w\, b_w\right) \geq \frac{V_u}{0.48\, f_{ctd}}$ where $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\gamma_{mc}$
(142)	$s \leq \frac{A_b \cdot f_{ys}}{f'_c \cdot 15\, mm} \leq 100\ mm$	$s \leq \frac{A_b\, f_{yds}}{f_{cd} \times 20\, mm} \leq 100\ mm$
(143)	$\rho_s = \frac{A_b \cdot \pi \cdot d_c}{A_c \cdot s} \geq 0{,}12 \cdot \frac{f'_c}{f_{ys}}$	$\rho_s = \frac{A_b\, \pi\, d_c}{A_c\, s} \geq 0.16 \frac{f_{cd}}{f_{yds}}$
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 $f_{ctd} = \left(0.35\sqrt{f'_c}\right)/\gamma_{mc}$

A.5 Equivalent equations for material factors

A.5.1 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 (1500 m/s ≥ v_s> 750 m/s);

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

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

Soil Profile S_E: a soil profile where the measured shear wave velocity v_s ≤ 180 m/s, or the standard penetration test resistance has an average value N 15 in the upper 30 m, or has more than 3.5 m of plastic (PI > 20), high moisture content (w > 40%) and low shear strength (s_u < 25 kPa) clays; and

Seismically vulnerable soils: sites where the soil profile contains soil having one or more of the following characteristics are beyond the scope of these guidelines:

- soils vulnerable to potential failure or collapse under seismic motions, such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soil,
- peats, highly organic clays, or both, with more than 3 m of thickness,
- very high plasticity clays (PI > 75) with more than 8 m of thickness, and
- soft to medium-stiff clays with more than 40 m of thickness.

Soil exploration to obtain the needed values to classsify must always be conducted by a designer familiar with these processes.

A.5.2 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 determined as a function of A_a, and the soil profile type from A.5.1. Linear interpolation can be used between values of A_a .

Site effect of seismically vulnerable soils, as described in A.5.1., is beyond the scope of these guidelines and designs should be made under the National Standard or other applicable standards.

Table A.5-1 – Site soil coefficient

Soil Profile	**Site coefficoent, F_a, for short periods of vibreation**				
	A_a < [0.1]	**A_a < [0.2]**	**A_a < [0.3]**	**A_a < [0.4]**	**A_a < [0.5]**
S_A	[0.80]	[0.80]	[0.80]	[0.80]	[0.80]
S_B	[1.00]	[1.00]	[1.00]	[1.00]	[1.00]
S_C	[1.20]	[1.20]	[1.10]	[1.00]	[1.00]
S_D	[1.60]	[1.40]	[1.20]	[1.10]	[1.00]
S_E	[2.50]	[2.70]	[1.20]	[0.90]	[0.90]

A.5.3 Design response spectral ordinates

For buildings complying with the limitations set forth in Section 5, natural periods of vibration may be assumed to fall within the range of short periods for which response to ground motion is constant.

The ordinates of the elastic design response spectrum, S_a, for a damping ratio of 5% of critical, expressed as a fraction of the acceleration of gravity, shall be calculated in the short periods of vibration range, using Equation A.5-1:

$$S_a = 2.5 A_a F_a \qquad \text{Equation A.5-1}$$

A.5.4 Seismic design base shear

A.5.4.1 Seismic-resistant structural system

The seismic-resistant structural system shall be lateral loads 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 providing a minimum collateral lateral load resistance.

A.5.4.2 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.

A.5.4.3 Computation of the seismic design base shear

The seismic design base shear, Vs, equivalent to the total horizontal inertial effects caused by the seismic ground motions, shall be determined using equation A.5-2.

$$V_s = \frac{S_a \cdot W}{R} \qquad \text{Equation A.5-2}$$

Where Sa shall be determined from equation A.5-1, R is the response modification factor determined from A.5.4.2., 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

A.5.4.4 Vertical distribution of the design seismic forces

The total seismic design base shear shall be distributed over the height of the building using equation A.5-3 and equation A.5-4. At each floor level designated as **x**, $\boldsymbol{F}_{\mathbf{x}}$ shall be applied over the area of the building in accordance with the mass distributions at that level.

$$F_x = C_{vx} \cdot V_s \qquad \text{Equation A.5-3 and}$$

$$C_{vx} = \frac{W_x \cdot V_s}{\sum_{i=1}^{n} (w_i \cdot h_i)} \qquad \text{Equation A.5-4}$$

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