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SANS 10160-1: BASIS OF STRUCTURAL DESIGN AND ACTIONS FOR BUILDINGS AND INDUSTRIAL STRUCTURES — PART 1: BASIS OF STRUCTURAL DESIGN

Remarks:

PLEASE NOTE:

- **The technical committee, SABS SC 59I responsible for the preparation of this standard has reached consensus that the attached document should become a South African standard. It is now made available by way of public enquiry to all interested and affected parties for public comment, and to the technical committee members for record purposes. Any comments should be sent by the indicated closing date, either by mail, or by fax, or by e-mail to**

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SOUTH AFRICAN NATIONAL STANDARD

Basis of structural design and actions for buildings and industrial structures

Part 1: Basis of structural design

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Table of changes

Change No.	Date	Scope

Foreword

This South African standard was approved by National Committee SABS SC 59I, *Construction standards - Basis for the design of structures* in accordance with procedures of the SABS Standards Division, in compliance with annex 3 of the WTO/TBT agreement.

This document supersedes SABS 0160:1989 (edition 2).

SANS 10160 consists of the following eight parts, under the general title *Basis of structural design and actions for buildings and industrial structures*:

Part 1: Basis of structural design.

Part 2: Self-weight and imposed loads.

Part 3: Wind actions.

Part 4: Seismic actions and general requirements for buildings.

Part 5: Basis of geotechnical design and actions.

Part 6: Actions induced by cranes and machinery.

Part 7: Thermal actions.

Part 8: Actions during execution.

Annexes A, B, C, D, E and F are for information only.

Contents

Contents *(concluded)*

0 Introduction

0.1 Background

With the revision of SANS 10160 the scope of application of SABS 0160:1989 (Amended 1993) has generally been maintained in terms of the structures provided for, design procedures to be applied and the associated levels of reliability, and actions to be considered. Similarly the materials-based design standards which are intended to be applied in conjunction with SANS 10160 have generally been maintained. Deviations in scope and contents from that of SABS 0160:1989 derive mainly from the incorporation of improved and additional models and procedures that are in the main implemented internationally.

The general basis of structural design utilises the limit states based partial factor procedures to achieve appropriate levels of reliability for the design of safe and sound structures. The requirements include not only the treatment of actions and their combinations and effects on structures, but also the material-independent requirements for structural resistance. Changes from the general requirements stipulated in SABS 0160:1989 result mainly from extensions of the design situations and the related limit states which are required to be considered. Although this appears to increase the complication of the design procedures, it really clarifies the requirements. The extended basis of design should improve the consistency of the reliability of structural performance (See Annex F), improve the reliability where necessary but also remove some unwarranted conservatism.

The provisions of SANS 10160 updates the procedures for the treatment of actions as stipulated in SABS 0160:1989 by presenting revised and extended requirements, load models and the determination of appropriate values for the actions. The revised procedures apply to self-weight and imposed loads, wind actions, seismic actions and earthquake resistance and crane induced actions.

An important addition to the scope of SANS 10160 is to provide for geotechnical design and actions for situations within the scope of buildings and similar industrial structures. Other additions include the following : Actions induced by stationary rotating machinery is added to the provisions for crane induced actions. New provisions for thermal actions include information on local climatic conditions, as specified in the TMH-7 requirements for bridge design. Requirements and actions on the structure during execution is also added, which represents the situations to which a structure is exposed during construction, prefabrication, erection or reconstruction. These requirements should ensure that proper attention is given to the assignment of responsibilities for the performance of the structure not only ultimately during its use, but also during its execution.

0.2 Relationship with Eurocodes

Although SABS 0160 served as basis for the scope and reference levels of reliability and ISO standards, in particular ISO 2394:1998 *General principles on reliability for structures* (also published as a South African Standard SANS 2394:2004) as basic reference, this standard is primarily based on appropriate parts of the Eurocodes.

Advances were made in the Eurocodes in the treatment of a comprehensive set of structures and structural materials within a consistent reliability framework and providing for an elaborate set of actions related to the function of the civil engineering works and environmental exposure, whilst allowing levels of safety to be set nationally. The comprehensive treatment of the design of civil engineering works in Eurocode results in harmonization and consistency between its various Parts. Equivalent unification is therefore also achieved by the reference of the various procedures incorporated into SANS 10160 to the respective Eurocode Parts.

Adjustments for local environmental conditions, present levels of reliability and a limited degree of providing for existing practice and preferences in SANS 10160 are similar to the adjustments allowed to Eurocode Member States through the Nationally Determined Parameters. SANS 10160 however deviates substantially from Eurocode practice through the compilation into a single standard the general basis of design and actions to be considered in the design of a limited scope of structures. A conscious effort was made to achieve as compact and effective layout of the limited relevant material. However since such a formulation and format can be considered as a *harmonised scaling down* of Eurocode, the benefits from the consistent and unified Eurocode procedures are maintained.

An important practical implication of the high degree of consistency that has been maintained between this standard and the relevant Eurocode Parts is that Eurocode procedures can be applied in design for situations which are outside the scope of this standard. Guidance to this effect is given in SANS 10160. Specialist input will generally be required for these situations.

The reference of this standard to the Eurocodes also implies a recommendation that the future revision of structural materials based design standards, or the introduction of new standards which are not presently available, should do likewise. Such development will improve the consistency between SANS 10160 and all other South African structural design standards. By sharing a common basis of design, the lack of consistency between the present materials based standards will also improve. Such development would also enhance harmonization of South African standards with international practice.

0.3 Outline of Parts

An outline of the most important features of the eight Parts of SANS 10160 and a summary of their contents are given as follows:

a) SANS 10160-1: Basis of structural design. SANS 10160-1 serves as a general standard to specify procedures for determining actions on structures and structural resistance in accordance with the partial factor limit states design approach. The requirements and procedures are formulated to achieve acceptable levels of safety, serviceability and durability of structures within the scope of application of SANS 10160.

Procedures for the basis of structural design include requirements for the specified minimum values for actions on structures presented in SANS 10160-2 to SANS 10160-8; the determination of design values for the effects of combined actions on the structure under a sufficiently severe and varied set of limit states; general requirements for sufficient structural resistance reliability to which the related materials-based design standards should comply.

Provisions are introduced for taking situations and associated actions into account which are not expected during design life, but with such severe consequences that the risks of such situations need to be considered. A proper basis for improved specifications of robustness requirements is also presented.

Improved specification of procedures for design assisted by testing is obtained by requiring an equivalent level of reliability to that achieved by the procedures of this Standard. Guidance is given on testing procedures and the statistical treatment of the results required for compliance.

b) SANS 10160-2 Self-weight and imposed loads. SANS 10160-2 presents procedures for the treatment of self-weight and imposed loads on buildings. Procedures for determining self-weight of structural and non-structural materials as permanent loads are given, including recommended values of material densities. Minimum characteristic values for imposed loads as variable actions are given for loads on floors as a function of the occupancy; an extended range of imposed loads for industrial use of buildings; imposed roof loads; horizontal loads on parapets, railings, balustrades and partitions.

c) SANS 10160-3: Wind actions. SANS 10160-3 covers procedures for the determination of actions on land-based structures due to natural winds. The scope of application is limited to the general type buildings and industrial structures (in line with SANS 10160-1) and is restricted to structures in which wind actions can be treated as quasi-static.

The wind climate given in SABS 0160:1989 is effectively maintained, but its presentation is modified. The basic wind speed is based on an equivalent 10 minute average values, with some updating of its geographical extent. Terrain categories are modified to present a more even distribution of wind exposure conditions. The wide-ranging additional information on pressure and force coefficients represent a substantial update of the procedures for wind actions on structures;

d) SANS 10160-4: Seismic actions and general requirements for buildings. SANS 10160-4 covers earthquake actions on buildings and provides strategies and rules for the design of buildings subject to earthquake actions. Provisions for actions on structures exposed to earthquakes are revised and updated. The specification of seismic design of standard structures is extended, but procedures are restricted to situations where principles of proper layout and detailing are complied with.

e) SANS 10160-5 Basis for geotechnical design and actions. SANS 10160-5 represents an extension of the scope of SANS 10160 to set out the basis for geotechnical design and gives guidance on the determination of geotechnical actions on buildings and industrial structures, including vertical earth loading, earth pressure, ground water and free water pressure and actions caused by ground movement. Procedures are given for determining representative values for geotechnical actions. The design of geotechnical structures such as slopes, embankments or free-standing retaining structures is not covered by the standard.

f) SANS 10160-6: Actions induced by cranes and machinery. SANS 10160-6 specifies imposed loads associated with overhead travelling bridge cranes on runway beams at the same level; and also actions induced by a limited range of stationary machinery causing harmonic loading. The standard includes improved provisions for crane induced actions by the introduction of new load models and proper specification of the combination of actions.

g) SANS 10160-7 Thermal actions. SANS 10160-7 introduces new procedures that cover principles and rules for calculating thermal actions on buildings, as well as their structural elements. Its main features are to introduce provisions for thermal actions based on the South African climate, including the classification and representation of actions, the determination of temperatures and temperature gradients in buildings.

h) SANS 10160-8 Actions during execution. SANS 10160-8 introduces new procedures that cover principles and general rules for the determination of actions which should be taken into account during the execution of buildings. Its main features are to introduce provisions for actions on structures during execution of the construction works, including actions on the partially completed works and temporary structures. It consists of procedures for the identification of design situations and representation of actions and their effects on the incomplete structure, considering all activities carried out for the physical completion of the work, including construction, fabrication and erection.

It should be noted that the responsibility for the performance of the structure during execution is assigned in accordance with contractual conditions and professional appointments. The responsibility for complying with the requirements of SANS 10160-8, and the associated requirements for structural resistance, should be clearly defined in these documents for individual projects.

1 Scope

1.1 SANS 10160 establishes principles and requirements for the safety, serviceability and durability of structures, describes the basis for their design and verification, specifies minimum design values for actions and gives guidelines for related aspects of structural reliability in the structural design of buildings and industrial structures.

1.2 The following structures are provided for:

(a) building structures

(b) industrial structures utilizing structural systems similar to those of building structures.

1.3 It is also applicable for the structural appraisal of existing structures, for developing the design of repairs and alterations or for assessing changes of use.

NOTE : Additional or amended provisions may be necessary where appropriate.

1.4 The general principles and procedures for structural design for the safety, serviceability and durability of structures provided in SANS 10160-1 apply to both the actions on structures and the behaviour of the structure in resisting such actions.

1.5 The general principles for the determination of minimum design values for actions on buildings and industrial structures provided in SANS 10160-1 shall be applied together with the requirements of parts 2 to 8 of SANS 10160 for the respective actions appropriate to the structure under consideration.

1.6 It covers the geotechnical actions directly relevant to buildings and industrial structures. These geotechnical actions are specified in SANS 10160-5, together with the relevant principles of geotechnical design.

1.7 It is intended to be used in conjunction with appropriate standards for the structural design of buildings and industrial structures (see clause 2), including geotechnical aspects, structural fire design, situations involving earthquakes, execution and temporary structures.

1.8 SANS 10160 does not cover the following:

a) actions due to fire;

b) actions on structures subject to internal pressures from the contents (for example bunkers, silos, water tanks) ;

c) actions due to hydrodynamic effects;

d) actions on chimneys, towers and masts;

e) actions on bridges;

f) actions on special industrial structures and

g) actions due to internal or external explosions (for example, from gas or explosive materials).

2 Normative references

The following referenced documents are indispensable for the application of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies. Information on currently valid national and international standards can be obtained from the SABS Standards Division.

SANS 10100-1, *The structural use of concrete – Part 1: Design.*

SANS 10137, *The installation of glazing in buildings.*

SANS 10160-2, *Basis of structural design and actions for buildings and industrial structure – Part 2: Self-weight and imposed loads.*

SANS 10160-3, *Basis of structural design and actions for buildings and industrial structure – Part 3: Wind actions.*

SANS 10160-4, *Basis of structural design and actions for buildings and industrial structure – Part 4: Seismic actions and general requirements for buildings.*

SANS 10160-5, *Basis of structural design and actions for buildings and industrial structure – Part 5: Basis for geotechnical design and actions.*

SANS 10160-6, *Basis of structural design and actions for buildings and industrial structure – Part 6: Actions induced by cranes and machinery.*

SANS 10160-7, *Basis of structural design and actions for buildings and industrial structure – Part 7: Thermal actions.*

SANS 10160-8, *Basis of structural design and actions for buildings and industrial structure – Part 8: Actions during execution.*

SANS 10162-1, *The structural use of steel – Part 1: Limit-state design of hot-rolled steelwork.*

SANS 10162-2, *The structural use of steel – Part 2: Limit-states design of cold-formed steelwork.*

SANS 10162-4, *Structural use of steel – Part 4: The design of cold-formed stainless steel structural members.*

SANS 10163-1 *The structural use of timber – Part 1: Limit-states design.*

SANS 10164-2, *The structural use of masonry – Part 2: Structural design and requirements for reinforced and prestressed masonry.*

ISO 3898, *Bases for design of structures – Notations – General symbols.*

3 Definitions and symbols

3.1 Definitions

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

3.1.1
accidental action
A
action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life

NOTE 1 An accidental action can be expected in many cases to cause severe consequences unless appropriate measures are taken.

NOTE 2 Impact, snow, wind and seismic actions may be variable or accidental actions, depending on the available information on statistical distributions.

3.1.2
accidental design situation
design situation involving exceptional conditions of the structure or its exposure, including fire, explosion, impact or local failure

3.1.3
action (*F*)
(a) set of forces (loads) applied to the structure (direct action);
(b) set of imposed deformations or accelerations caused for example by temperature changes, moisture variation, uneven settlement or earth quakes (indirect action)

3.1.4
basic variable
part of a specified set of variables representing physical quantities which characterise actions and environmental influences, geometrical quantities and material properties including soil properties

3.1.5
characteristic value (X_k or R_k)
value of a material or product property having a prescribed probability of not being attained in a hypothetical unlimited test series. This value generally corresponds to a specified fractile of the assumed statistical distribution of the particular property of the material or product. A nominal value is used as the characteristic value in some circumstances

3.1.6
characteristic value of a geometrical property (a_k)
value usually corresponding to the dimensions specified in the design. Where relevant, values of geometrical quantities may correspond to some prescribed fractiles of the statistical distribution

3.1.7
characteristic value of an action (F_k)
principal representative value of an action

NOTE In so far as a characteristic value can be fixed on statistical bases, it is chosen so as to correspond to a prescribed probability of not being exceeded on the unfavourable side during a "reference period" taking into account the design working life of the structure and the duration of the design situation

3.1.8
design situations
sets of physical conditions representing the real conditions occurring during a certain time interval for which the design will demonstrate that relevant limit states are not exceeded

3.1.9
design value of a geometrical property (a_d)
generally a nominal value. Where relevant, values of geometrical properties may correspond to some prescribed fractile of the statistical distribution

NOTE The design value of a geometrical property is generally equal to the characteristic value. However, it may be treated differently in cases where the limit state under consideration is very sensitive to the value of the geometrical property, for example when considering the effect of geometrical imperfections on buckling. In such cases, the design value will normally be established as a value specified directly. Alternatively, it can be established from a statistical basis, with a value corresponding to a more appropriate fractile (e.g. a rarer value) than applies to the characteristic value

3.1.10
design value of a material or product property (X_d or R_d)
value obtained by dividing the characteristic value by a partial factor γ_m or γ_M or, in special circumstances, by direct determination

3.1.11
design value of an action (F_d)
value obtained by multiplying the representative value by the partial factor γ_f

NOTE The product of the representative value multiplied by the partial factor $\gamma_F = \gamma_{S,d} \times \gamma_f$ may also be designated as the design value of the action.

3.1.12
design working life
assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary

3.1.13
execution
all activities carried out for the physical completion of the work including procurement, the inspection and the documentation thereof

NOTE the term covers work on site; it may also signify the fabrication of components off site and their subsequent erection on site.

3.1.14
irreversible serviceability limit states
serviceability limit states where some consequences of actions exceeding the specified service requirements will remain when the actions are removed

3.1.15
limit states
a state beyond which the structure no longer satisfies the design performance requirements.

NOTE Limit states separate desired states (no failure) from undesired states (failure).

3.1.16
nominal value
value fixed on non-statistical bases, for instance on acquired experience or on physical conditions

3.1.17
permanent action (*G*)
action that is likely to act throughout a given reference period and for which the variation is always in the same direction (monotonic) until the action attains a certain limit value

3.1.18
persistent design situation
design situation that is relevant during a period of the same order as the design working life of the structure

NOTE Generally refers to conditions of normal use.

3.1.19
reliability
reliability of a structure or a structural member to fulfil the specified requirements, including the design working life, for which it has been designed; reliability is usually expressed in probabilistic terms

NOTE Reliability covers safety, serviceability and durability of a structure.

3.1.20
reliability differentiation
measures intended for the socio-economic optimisation of the resources to be used to build construction works, taking into account all the expected consequences of failures and the cost of the construction works

3.1.21
representative value of an action (F_{rep})
A value used for the verification of a limit state.

NOTE Representative values consist of characteristic values, combination values, frequent values and quasi-permanent values, but may also consist of other values.

3.1.22
resistance (*R*)
capacity of a member or component, or a cross-section of a member or component of a structure, to withstand actions without mechanical failure, for example bending resistance, buckling resistance, tension resistance etc.

3.1.23
reversible serviceability limit states
serviceability limit states where no consequences of actions exceeding the specified service requirements will remain when the actions are removed

3.1.24
seismic action (A_E)
action that arises due to earthquake ground motions

3.1.25
serviceability limit states
states that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met

3.1.26
transient design situation
design situation that is relevant during a period much shorter than the design working life of the structure and which has a high probability of occurrence

NOTE A transient design situation refers to temporary conditions of the structure, of use, or exposure, for example during construction or repair.

3.1.27
ultimate limit states
states associated with collapse or with other similar forms of structural failure

NOTE These states generally correspond to the maximum load-carrying resistance of a structure or structural member.

3.1.28
variable action (*Q*)
action for which the variation in magnitude with time is neither negligible in relation to the mean value nor monotonic

3.2 Symbols

NOTE The notation used is based on ISO 3898.

Latin upper case letters

A is the accidental action

A_d is the design value of an accidental action

A_E is the seismic actions

A_{Ed} is the design value of seismic action $A_{Ed} = \gamma_I \times A_{Ek}$

C_d is the limiting design value of the relevant serviceability criterion

E_d is the design value of effect of actions

$E\{-\}$ is the a function defining the effect of actions

$E_{d,dst}$ is the design value of the effect of destabilising actions

$E_{d,stb}$ is the design value of the effect of stabilising actions

F is an action

F_d is the design value of an action

F_k is the characteristic value of an action

$F_{k,i}$ is the characteristic value of action i

F_{rep} is the representative value of an action

F_t is the limiting tie load of 60 kN/m or (20+4 n_s) kN/m, whichever is less

G is the permanent action

$G_{k,j}$ is the characteristic value of permanent action j

H is the building height

H_i is the storey height

H_c is the clear storey height

K_F is the multiplication factor

L is the span of horizontal ties

P is the relevant representative value of a prestressing action

Q is the variable actions

Q_{CI} is the variable crane induced actions

Q_{EX} is the variable actions during execution of structure

Q_G is the variable geotechnical actions

Q_I is the imposed loads, for example on building floors and roofs

Q_k is the characteristic value of a variable action

Q_{MI} is the variable actions induced by machinery

Q_T is the variable thermal actions

Q_W is the wind actions

$Q_{k,1}$ is the characteristic value of the leading variable action

$Q_{k,i}$ is the characteristic value of the accompanying variable action i

R is the resistance

R_d is the design value of the resistance

R_k is the characteristic value of the resistance

$R\{-\}$ is the a function defining the resistance for a particular limit state

T is the force that should be resisted by vertical tie

T_i is the design tensile load for accidental limit state for effective horizontal internal ties

T_p is the design tensile load for accidental limit state for effective horizontal perimeter ties

X_d is the design value of a material property

X_k is the characteristic value of a material property

Latin lower case letters

a_d is the design value of geometrical data

a_k is the characteristic values of geometrical data

p_f is the notional probability of failure

g_k is the characteristic floor self-weight

q_k is the characteristic imposed floor load

n_s is the number of storeys

s is the spacing of horizontal ties

t is the wall thickness

u is the overall horizontal displacement over the building height H

u_i is the horizontal displacement over the storey height H_i for storey i

u_1 is the initial part of the deflection under structural self-weight

u_2 is the initial part of the deflection under non-structural self-weight

u_3 is the additional part of the deflection due to the variable actions (short-term)

u_4 is the long-term part of the deflection under permanent and quasi-permanent leads (creep-deflection)

w is the vertical displacement of a structural member

w_c is the pre-camber in the unloaded structural member

w_1 is the initial part of the deflection under structural self-weight

w_2 is the initial part of the deflection under non-structural self-weight

w_3 is the additional part of the deflection due to the variable actions (short-term)

w_4 is the long-term part of the deflection under permanent and quasi-permanent leads (creep-deflection)

w_{tot} is the total deflection as sum of w_1, w_2, w_3 and w_4

w_{max} is the deviation of the respective middle or end point of the member from a reference position

$x_{k,i}$ is the characteristic value of material property i

Greek upper case letters

Δ_a is the change made to nominal geometrical data for particular design purposes for example assessment of effects of imperfections

$\sum$ implies "the combined effect of"

Greek lower case letters

"+" implies "o be combined with"

β_t is the target safety index

λ_f is the partial factor for actions, which takes account of the possibility of unfavourable deviations of the action values from the representative values

λ_F is the partial factor for actions, also accounting for model uncertainties and dimensional variations

$\lambda_{F,i}$ is the partial factor which allows for the variability in the action, the uncertainty in modelling the action and in some cases the modelling of the action effect

λ_g is the partial factor for permanent actions, which takes account of the possibility of unfavourable deviations of the action values from the representative values

γ_G is the partial factor for permanent actions, also accounting for model uncertainties and dimensional variations

$\lambda_{G,j}$ is the partial factors for the permanent action j

λ_m is the partial material factor which allows for uncertainty in the material property

λ_M is the partial factor for material property, also accounting for model uncertainties and dimensional variations

λ_Q is the partial factor for variable action, also accounting for model uncertainties and dimensional variations

$\lambda_{Q,1}$ is the partial factors for the leading variable action

$\lambda_{Q,i}$ is the partial factor for variable action i

λ_R is the partial factor covering uncertainty in the resistance model, plus geometric deviations if these are not modelled explicitly

$\lambda_{S,d}$ is the partial factor associated with the uncertainty of the action and/or action effect model

Φ is the cumulative normal distribution function

ψ is the combination factor for variable action

ψ_i is the combination factor for an accompanying variable action that accounts for the probability of simultaneous occurrence of this accompanying action with the corresponding leading action; if the combination factor does not apply, $\psi_i = 1$

$\psi_{geotechnical}$ is the combination factor for variable geotechnical actions

ψ_{crane} is the combination factor for variable crane induced actions

4 Requirements

4.1 General pre-requisites

The general pre-requisites for the application of SANS 10160 are:

a) the choice of the structural system and the design of the structure are made by appropriately qualified and experienced personnel;

b) execution is carried out by personnel having the appropriate skill and experience;

c) adequate supervision and quality control are provided during the execution of the work, i.e. in the design offices, factories, plants, and on site;

d) the construction materials and products are used as specified in SANS 10160, in the relevant standards (see clause 2), the relevant execution standards, reference material or product specifications;

e) the structure will be adequately maintained; and

f) the structure will be used in accordance with the design assumptions.

NOTE 1 There may be cases when the assumptions above need to be supplemented.

NOTE 2 See 4.7 and annex A for quality management procedures relevant to ensuring compliance with the general assumptions of SANS 10160.

4.2 Basic requirements

4.2.1 A structure shall be designed and executed in accordance with the limit states procedures of SANS 10160 and the materials-based standards (see clause 2) which deem to satisfy the basic requirement that the structure will, during its intended life, with appropriate degrees of reliability and in an economic way the following:

a) sustain all actions and influences likely to occur during execution and use; and

b) remain fit for the use for which it is intended.

4.2.2 A structure shall be designed to have adequate:

a) structural resistance;

b) serviceability and

c) durability.

4.2.3 The basic requirements shall be met:

a) by the choice of suitable materials;

b) by appropriate design and detailing; and

c) by specifying control procedures for design, production, execution, and use relevant to the particular project.

4.3 Requirements for structural integrity and robustness

4.3.1 A structure shall be designed and executed in accordance with the limit states procedures of SANS 10160 and the materials-based standards (see clause 2) for unidentified and identified accidental design situations and actions in order to provide compliance with the basic requirements that it will not be damaged to an extent disproportionate to the original cause by abnormal events, and has the ability to withstand local damage without it causing or initiating widespread collapse.

NOTE Abnormal events involve exceptional conditions of the structure or its exposure to fire, explosions, earthquakes, impact or local failure or the consequences of human error.

4.3.2 Sufficient integrity and robustness of the structure should be provided for unidentified accidental design situations and actions.

NOTE 1 Examples of unidentified accidental design situations and actions include explosions, adjacent excavation or flooding causing severe local foundation failure, very high winds such as cyclones and tornadoes and the consequences of gross human error.

NOTE 2 Strategies for the provision of sufficient integrity and robustness against unidentified design situations and actions are given in 7.4.2.

4.3.3 Sufficient integrity and robustness of the structure should be provided for identified accidental design situations and actions. Values for identified accidental actions should be determined in accordance with the requirements of the applicable parts of SANS 10160.

NOTE Strategies for the provision of sufficient integrity and robustness against identified design situations and actions are given in 7.4.3.

4.3.4 Optional identified accidental design situations related to specific structures may be provided for in accordance with the procedures as specified in this part of SANS 10160. Although the level of performance may be decided on by the owner, identified events and the resulting situations and actions should be considered to ensure that the risks to which occupants and the public are exposed, are lower than the general risk implied by SANS 10160.

4.3.5 Potential damage shall be avoided or limited by appropriate choice of one or more of the following :

a) avoiding, eliminating or reducing the hazards to which the structure can be subjected;

b) selecting a structural form which has low sensitivity to the hazards considered;

c) selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localised damage;

e) avoiding as far as possible structural systems that can collapse without warning and

f) tying the structural members together.

4.4 Reliability Management

4.4.1 Principles

4.4.1.1 The reliability required for structures within the scope of SANS 10160 shall be achieved:

a) by design in accordance with SANS 10160 and the structural materials-based design standards (see clause 2).

b) by appropriate execution and quality management measures.

NOTE See annex A for additional guidance on the management of reliability for structures.

4.4.1.2 Different levels of reliability may be adopted inter alia for:

a) structural resistance

b) serviceability

4.4.1.3 The choice of the levels of reliability for a particular structure should take account of the relevant factors, including:

a) the possible cause or mode (or both) of attaining a limit state;

b) the possible consequences of failure in terms of risk to life or serious injury;

c) potential economic losses;

d) public aversion to failure and

e) the expense and procedures necessary to reduce the risk of failure.

4.4.1.4 The levels of reliability that apply to a particular structure may be specified in one or both of the following ways:

a) by the classification of the structure as a whole

b) by the classification of its components.

NOTE See annex A for guidance on the reliability classification of structures.

4.4.1.5 The levels of reliability relating to structural resistance and serviceability can be achieved by suitable combinations of:

a) preventive and protective measures (for example implementation of safety barriers, active and passive protection measures, protection against risks of corrosion such as painting or cathodic protection);

b) measures relating to the following design calculations:

1) representative values of design variables; and

2) the choice of partial factors;

c) measures relating to quality management;

d) measures aimed to reduce errors in design and execution of the structure, and gross human errors;

e) other measures relating to the following other design matters:

1) the basic requirements;

2) the degree of robustness (structural integrity);

3) durability, including the choice of the design working life;

4) the extent and quality of preliminary investigation of soils and possible environmental influences;

5) the accuracy of the mechanical models used; and

6) the detailing.

f) efficient execution, for example in accordance with the structural materials-based design standards specified in clause 2; and

g) adequate inspection and maintenance according to procedures specified in the project documentation.

4.4.1.6 The measures to prevent potential causes of failure and/or reduce their consequences may, in appropriate circumstances, be interchanged to a limited extent provided that the required reliability levels are maintained.

4.4.2 Implementation

4.4.2.1 Correct application of the requirements in SANS 10160 together with the structural materials-based standards (see clause 2) will be deemed to achieve the intended levels of reliability for the safety, serviceability and durability performance of structures, based on current knowledge of structural reliability.

4.4.2.2 The required level of reliability shall be achieved by the application of the principles of limit states methods in the design of structures. The limit states are divided into the following two categories:

a) the ultimate limit states, which are those limits concerning the safety of people, the structure, or any part of the structure.

b) the serviceability limit states, which are those limits concerning the functioning of the structure under normal use.

4.4.2.3 Reliability differentiation to take account of differences in performance of the structure, consequences of its failure and the nature of failure should be applied by assigning different reliability classes to the structure. Supervision during design and inspection during execution should also be taken into account. See annex A.

NOTE 1 Reliability differentiation can be applied by adjustment of provisions for the reference reliability class for which design parameters are generally provided.

NOTE 2 The level of reliability used as reference is expressed in terms of the notional probability of not being achieved as $p_{\mathrm{f}} = \Phi(-\beta_{\mathrm{t}})$, where Φ is the cumulative normal distribution function. A value of $\beta_{\mathrm{t}} = 3{,}0$ used for the ultimate limit state corresponds to probability of failure $p_{\mathrm{f}} = \Phi(-3{,}0) \approx 0{,}001$.

4.4.2.4 The structural resistance achieved by applying the structural materials-based design standards specified (see clause 2) is deemed to satisfy the required level of resistance.

NOTE In accordance with the principle of considering the reliability of existing practice to be acceptable, levels of resistance reliability for design according the present materials-based structural design standards (see clause 2) are regarded to be sufficient.

4.5 Design working life

The design working life shall be specified. The design working life should reflect both the intended service life and the influence of the consequence of structural failure on the appropriate level of reliability.

NOTE Indicative categories are given in Table 1. It may also be used for determining time-dependent performance (for example fatigue-related calculations and durability).

Table 1 — Indicative design working life

1	2	3
Design working life category	**Indicative design working life years**	**Description of Structures**
1	10	Temporary structures[1,2]
2	25	Replaceable structural parts, for example bearings, Agricultural structures and similar structures with low consequences of failure.
3	50	Building structures and other common structures[3]
4	100	Buildings structures designated as essential facilities such as having post-disaster functions (hospitals and communication centres, fire and rescue centres), having high consequences of failure[4] or having another reason for an extended design working life.

[1] Structures or parts of structures that can be dismantled with a view to being re-used should not be considered as temporary.
[2] Refer to SANS 10160-8 for the assessment of temporary structures during execution.
[3] The design working life category applies to the reference reliability class referred to in 4.4.2.3.
[4] Consequences of structural failure could be determined according to annex A

4.6 Durability

4.6.1 The structure shall be designed such that deterioration over its design working life does not impair the performance of the structure below that intended, having due regard to its environment and the anticipated level of maintenance.

4.6.2 In order to achieve an adequately durable structure, the following should be taken into account:

a) the intended or foreseeable use of the structure;

b) the required design criteria;

c) the expected environmental conditions;

d) the composition, properties and performance of the materials and products;

e) the properties of the ground and geotechnical conditions;

f) the choice of the structural system;

g) the shape of members and the structural detailing;

h) the quality of workmanship, and the level of control;

i) the particular protective measures;

j) the intended maintenance during the design working life.

NOTE The materials structural design standards specified in Clause 2 stipulate appropriate measures to reduce deterioration.

4.6.3 The environmental conditions should be identified at the design stage and their significance assessed in relation to durability so that adequate provisions can be made for protection of the materials used in the structure in accordance with the requirements of the materials-based standards (see clause 2).

4.6.4 The degree of any deterioration may be estimated on the basis of calculations, experimental investigation, experience from earlier constructions, or a combination of these considerations.

4.7 Quality management

In order to provide a structure that corresponds to the requirements and to the assumptions made in the design, appropriate quality measures should be in place. These measures comprise:

a) **Quality management** calls for the identification of the reliability aspects of quality and management of the activities related to achieving quality requirements.

b) **Quality assurance** concerns the specific actions taken, such as through a quality plan and documentation, to achieve adequate confidence that the design fulfils the specified requirements for quality.

(c) **Quality control** measures consist of the collection of information applied to judge and decide on compliance according to pre-set control criteria and acceptance rules.

NOTE ISO 9001 is an acceptable basis for quality management measures, where relevant. See also SANS 2394.

5 Principles of limit states design

5.1 General

5.1.1 The relevant design situations shall be selected taking into account the circumstances under which the structure is required to fulfil its function.

5.1.2 The selected design situations shall be sufficiently severe and varied so as to encompass all conditions that can reasonably be foreseen to occur during the execution and use of the structure.

5.1.3 A distinction shall be made between ultimate limit states and serviceability limit states (see 4.4.2.2).

5.1.4 Verification of one of the two categories of limit states may be omitted provided that sufficient information is available to prove that it is satisfied by the other.

5.1.5 Limit states shall be related to design situations, taking account of the time dependent nature of both actions and the response of the structure where necessary.

5.1.6 Verification of limit states that are concerned with time dependent effects (for example fatigue) should be related to the design working life of the construction.

5.2 Application of limit states design

5.2.1 Design for limit states shall be based on the use of structural and load models for relevant limit states.

5.2.2 It shall be verified that no limit state is exceeded when relevant design values are used in these models for:

a) actions;

b) material properties;

c) product properties and

d) geometrical data.

5.2.3 All design situations and load cases shall be considered and the relevant critical design situations and load cases shall be identified and verified.

5.2.4 The requirements for 5.2.1 should be achieved by the partial factor method, described in clause 7 for the ultimate limit state and clause 8 for the serviceability limit state.

5.2.5 For a particular verification, load cases should be selected, identifying compatible load arrangements, sets of deformations and imperfections.

5.2.6 Possible deviations from the assumed directions and positions of actions shall be taken into account.

5.2.7 Structural and load models can be either physical models or mathematical models.

5.2.8 As an alternative, a design directly based on probabilistic methods may be used.

NOTE 1 Specific conditions for the use of probabilistic design methods may be required.

NOTE 2 For a basis of probabilistic methods the designer could consult EN 1990, SANS 2394, the JCSS Model Code for Reliability Based Structural Design or specialist literature

5.3 Ultimate limit state

5.3.1 Ultimate limit states concern the following:

a) the safety of people, and

b) the safety of the structure

5.3.2 In some circumstances, the limit states that concern the protection of the contents should be classified as ultimate limit states.

NOTE The circumstances are those agreed for a particular project with the client or the relevant authority.

5.3.3 States prior to structural collapse, which, for simplicity, are considered in place of the collapse itself, may be treated as ultimate limit states.

5.3.4 The following ultimate limit states shall be verified where they are relevant:

a) loss of equilibrium of the structure or any part of it, considered as a rigid body;

b) failure by excessive deformation, transformation of the structure or any part of it into a mechanism, rupture, loss of equilibrium of the structure or any part of it, including supports and foundations;

c) failure caused by fatigue or other time dependent effects.

NOTE Different sets of partial factors are associated with the various ultimate limit states (see clause 7).

5.3.5 Design situations for the ultimate limit state shall be classified in accordance with the time related nature of the application of the action as :

a) **persistent design situations**, which refer to the conditions of normal use;

b) **transient design situations**, which refer to temporary conditions applicable to the structure, for example during execution or repair;

c) **accidental design situations**, which refer to exceptional conditions applicable to the structure or to its exposure, for example the consequences of localised failure, fire, explosion, or impact,

d) **seismic design situations**, which refer to conditions applicable to the structure when subjected to seismic events and regarded as an accidental situation.

e) **failure caused by fatigue** of the structural material.

5.4 Serviceability limit state

5.4.1 Serviceability limit states apply to the following requirements for the structure under normal use:

a) the functioning of the structure or structural members;

b) the acceptability of the structure by users in terms of perceived safety and wellbeing and

c) the appearance of the structure.

NOTE 1 In the context of serviceability, the term "wellbeing" means the prevention of discomfort and distress to users of the structure.

NOTE 2 In the context of serviceability, the term "appearance" is concerned with such criteria as high deflection and extensive cracking, rather than aesthetics.

NOTE 3 Serviceability requirements may be agreed upon for individual projects.

NOTE 4 Deformations affecting the strength and stability of a building or of its parts are taken into account in the process of structural design for the ultimate limit states. It is however necessary that designers be aware of certain cases involving static or dynamic instability where the conditions existing during the normal use of the building may have a considerable effect on the ultimate limit state. See annex D.6

5.4.2 Design situations for the serviceability limit state shall be classified in accordance with the time related nature of the application of the action as well as the structural consequence as :

a) **irreversible design situations,** causing serviceability consequences which will remain until the structure has been repaired;

b) **reversible design situations,** causing serviceability consequences which will remain only as long as the cause of the limit being exceeded is present and

c) **long-term design situations,** causing serviceability consequences that may develop over a long period of time.

NOTE 1 : Examples of irreversible limit states are deformations that may cause damage to structural and non-structural elements.

NOTE 2 : Examples of reversible limit states are;

- deformations and vibrations that may cause discomfort to people
- deformations and vibrations due to the effect of variable actions that may affect the functioning of the structure.

NOTE 3 : Examples of long-term effects are deformations due to shrinkage, creep or relaxation that may affect the appearance of the structure.

5.4.3 The verification of serviceability limit states should be based on criteria concerning the following aspects:

a) deformations that affect:

1) the functioning of the structure (including the functioning of machines or services),

2) finishes or non-structural members,

3) or the appearance of the structure;

b) vibrations that:

1) cause discomfort to people, or

2) limit the functional effectiveness of the structure;

c) damage that is likely to adversely affect:

1) the appearance;

2) the durability or

3) the functioning of the structure.

5.5 Actions

5.5.1 Classification of actions

5.5.1.1 Actions shall be classified by their variation in time as follows:

(a) **Permanent actions** (G), for example self-weight of structures, fixed equipment, earth pressure an actions caused by shrinkage and uneven settlement;

b) **Variable actions** (Q): imposed loads for example on building floors and roofs (Q_I), wind actions (Q_W), thermal actions (Q_T), actions during execution of structure (Q_{EX}), crane induced actions (Q_{CI}), machinery induced actions (Q_{MI}) and certain geotechnical actions (Q_G);

c) **Accidental actions** (A), for example impact from vehicles and

d) **Seismic actions** (A_E), are treated as a special case of accidental actions;

NOTE 1 Actions caused by imposed deformations can be either permanent or variable.

NOTE 2 Actions caused by water may be considered as permanent and/or variable actions depending on the variation of their magnitude with time

NOTE 3 Where snow loads are treated as an imposed load due to the site location, appropriate design standards such as EN 1991-1-3 or specialist literature can be consulted.

5.5.1.2 Actions shall also be classified by:

a) their origin, as direct or indirect;

b) their spatial variation, as fixed or free and

c) their nature or the structural response (or both), as static or dynamic

5.5.2 Characteristic values of actions

5.5.2.1 The characteristic value F_k is the main representative value of the action.

5.5.2.2 The self-weight of the structure may be represented by a single characteristic value and be calculated on the basis of the nominal dimensions and mean unit masses.

5.5.2.3 For variable actions, the characteristic value (Q_k) shall correspond to either :

a) an upper value with an intended probability of not being exceeded or a lower value with an intended probability of being achieved, during some specific reference period;

(b) a nominal value, which may be specified in cases where a statistical distribution is not known.

NOTE The characteristic value for climatic actions is based on a probability of 0,02 of its time varying part being exceeded for a reference period of one year. This is equivalent to a return period of 50 years for the time varying part. However, in some cases the character of the action or the selected design situation (or both) makes another fractile or return period (or both) more appropriate.

5.5.2.4 The design value for accidental actions A_d shall be specified in agreement with the associated risk in terms of the probability of the occurrence and the consequences of the actions considered.

5.5.2.5 The values for the accidental actions specified in parts 2, 3, 5 and 6 of SANS 10160 should be applied as design value A_d.

5.5.2.6 For seismic actions, the design value A_{Ed} shall be used according to SANS 10160-4.

5.5.2.7 For multi-component actions, the characteristic action should be represented by groups of values, each group to be considered separately in design calculations.

NOTE The horizontal and vertical actions induced by cranes are an example of multi-component actions.

5.5.3 Representative values of accompanying variable actions

The representative value of accompanying variable actions, which are combined with a leading variable action, represent the point-in-time value of the action, which is the most likely action on the structure at any instant of time.

5.5.4 Fatigue actions

5.5.4.1 The models for fatigue actions should be those that have been developed in the relevant parts of the structural materials design standards (see clause 2) and used together with SANS 10160, from evaluation of structural response to fluctuations of loads for common structures (for example response of tall slender structures to wind actions, crane induced actions on support structures and actions induced by stationary machinery).

5.5.4.2 For structures outside the field of application of models established in the relevant parts of the structural materials design standards (see clause 2), fatigue actions should be defined from the evaluation of measurements or equivalent studies of the expected action spectra.

5.5.5 Representation of dynamic actions

5.5.5.1 When dynamic actions do not cause significant acceleration of the structure, the effects of acceleration is included implicitly in the static equivalent characteristic values for the actions or explicitly by applying dynamic magnification factors to the characteristic static loads.

NOTE Limits on the use of these static equivalent load models are given in the relevant sections of SANS 10160.

5.5.5.2 When dynamic actions cause significant acceleration of the structure, dynamic analysis of the system should be performed.

5.6 Material and product properties

5.6.1 Properties of materials (including soil and rock) or products should be represented by characteristic values X_k.

5.6.2 When a limit state verification is sensitive to the variability of a material property, upper and lower characteristic values of the material should be taken into account.

5.6.3 Unless otherwise stated in the respective structural materials design standards, the characteristic value of a material or product property should be defined as:

a) the 5% fractile value where a low value is unfavourable

b) the 95% fractile where a high value is unfavourable.

5.6.4 Material property values shall be determined from standardised tests performed under specified conditions. A conversion factor shall be applied where it is necessary to convert the results into values which can be assumed to represent the behaviour of the material or product in the structure or ground.

5.6.5 Where insufficient statistical data are available to establish the characteristic values of a material or product property, nominal values may be taken as the characteristic values, or design values of the property may be established directly. Where upper or lower design values of a material or product property are established directly (for example friction factors, damping ratios), they should be selected so that more adverse values would affect the probability of occurrence of the limit state under consideration to an extent similar to other design values.

5.6.6 The characteristic upper value of the strength should be used where an upper estimate of strength is required (for example for capacity design measures and for tensile strength of concrete for the calculation of the effects of indirect actions).

5.6.7 The reductions of the material strength or product resistance to be considered resulting from the effects of repeated actions can lead to a reduction of the resistance over time due to fatigue, as specified in the structural materials-based design standards (see clause 2).

5.6.8 The structural stiffness parameters (for example moduli of elasticity, creep coefficients and thermal expansion coefficients) should be represented by mean values. Different values should be used depending on the duration of the load.

NOTE In some cases, a lower or higher value than the mean for the modulus of elasticity may have to be taken into account (for example in the case of stability, in which case the requirements of 5.6.3 apply).

5.6.9 Where a partial factor for materials or products is needed, a conservative value shall be used, unless suitable statistical information exists to assess the reliability of the value chosen.

NOTE Suitable account may be taken where appropriate of the unfamiliarity of the application or materials/products used.

5.6.10 Characteristic values for materials or products should be applied directly as design values for the determination of characteristic resistance for accidental design situations, conditionally to maintaining sufficient ductility in order to achieve alternative load paths as intended. If sufficient ductility is not achieved, the consequences of non-ductile failure should be accounted for.

5.7 Geometrical properties

5.7.1 Geometrical data shall be represented by their characteristic values, or (for example in the case of imperfections) directly by their design values.

5.7.2 The dimensions specified in the design may be taken as characteristic values.

5.7.3 Where their statistical distribution is sufficiently known, values of geometrical quantities that correspond to a specific fractile of the statistical distribution may be used.

5.7.4 Imperfections that should be taken into account in the design of structural members should be specified as characteristic values in the structural materials design standards (see clause 2).

5.7.5 Tolerances for connected parts that are made from different materials shall be mutually compatible considering the material properties.

5.8 Geotechnical parameters and actions

5.8.1 Geotechnical parameters shall be represented by their characteristic values as given in SANS 10160-5.

6 Combination values of variable actions

6.1 The combination value of a variable action is the product of the combination factor Ψ and the characteristic value Q_k. The combination value of a variable action should also be considered as the quasi-permanent part of the variable action for consideration of long-term and appearance serviceability. The combination factors Ψ are specified in Table 2 where the accompanying variable action is not correlated to the leading variable actions.

Table 2 – Action combination factors for uncorrelated variable actions

1	2	3	4	5
Variable actions	**Part**	**Category**	**Specific use**	**Combination factor Ψ**
Imposed Loads for occupancy class category	2	A	Domestic and residential areas	0,3
		B	Public areas not susceptible to crowding	0,3
		C	Public areas where people may congregate	0,3
		D	Shopping areas	0,3
		E1:	Light industrial use	0,5
		E2:	Industrial use	0,6
		E3:	Storage areas	0,8
		$FL_1 – FL_6$	Fork lifts	0,6
		F	Traffic and parking areas for vehicles ≤ 25 kN	0,8
		G	Traffic and parking areas for vehicles 25 – 160 kN	0,3
		H	Inaccessible roofs	0
		J	Accessible flat roofs, excluding occupancy categories A – D	0,3
		K	Accessible flat roofs with occupancies A – D	As per category A to D
		HCL1-HCL2	Helicopter load	0
Wind actions	3		Applied to accompanying action	0
			Applied to reversible and long-term serviceability actions	0,3
Geotechnical actions: Variable	5			$\Psi_{geotechnical}$
			Groundwater	(1,0)
			Ground water (Fluids)	(1,0)
Actions due to Cranes (horizontal & vertical)	6			Ψcrane [1]
Thermal actions	7			0,3
Other types of variable loads not considered above (for example material loads) in the absence of more detailed information				Ψ[2]

[1] Refer to Part 6 for the determination of an appropriate value of Ψ_{crane}.
[2] Appropriate value, based on value of variable action with similar arbitrary-point-in-time properties

6.2 The degree of dependence or correlation between the leading variable action and the accompanying variable action should be taken into account. Where full correlation between the leading variable action and the accompanying variable action occurs, an action combination factor $\Psi = 1,0$ should be taken. For intermediate values interpolated values between 1,0 and the action combination factor given in Table 2 should be used.

NOTE For example, for two cranes working in tandem, $\Psi = 1$ would apply; for cranes in adjacent bays that operate completely independently, $\Psi = 0,5$ may be applied to the additional action from the second crane.

7 Ultimate limit states design verification

7.1 Verification

7.1.1 The following ultimate limit states shall be verified as relevant :

a) **STR and STR-P** – Internal failure or excessive deformation of the structure or structural members in which the strength of the structural material is significant in providing resistance. STR-P represents the case of dominant permanent action;

b) **EQU** – Loss of static equilibrium of the structure or any part of it or the ground considered as a rigid body or involving uplift due to water pressure (buoyancy) or other vertical actions, where the strengths of construction materials or ground are generally not governing;

c) **GEO** – Failure or excessive deformation of the ground in which the strength of the ground is significant in providing resistance;

d) **ACC** – Limit states involving accidental and seismic actions, and .

e) **FAT** – Fatigue failure of the structure or structural member.

NOTE Design for fatigue is done according to the structural materials design standards (see clause 2).

7.1.2 In the case of fire, the structural resistance shall be adequate for the required period of time.

7.2 Criteria of failure for resistance and static equilibrium

7.2.1 When considering an ultimate limit state it shall be verified that:

$$E_d < R_d \tag{1}$$

where

E_d is the design value of the effect of actions as defined in equation (2) :

R_d is the design value of the corresponding resistance as defined in equation (3) :

$$E_d = E\left\{\sum \gamma_{F,i} \times \psi_i \times F_{k,i}\right\} \tag{2}$$

$$R_d = \frac{1}{\gamma_R} R\left\{\sum \frac{x_{k,i}}{\gamma_m}\right\} \quad (3)$$

where

$E\{-\}$ is a function defining the effect of actions

$R\{-\}$ is a function defining the resistance for a particular limit state

$\sum$ implies “the combined effect of”

$\gamma_{F,i}$ is the partial factor which allows for the variability in the action, the uncertainty in
modelling the action and in some cases the modelling of the action effect

Note: In a more general case the effects of actions depend on material properties.

ψ_i is the combination factor for an accompanying variable action that accounts for the probability of simultaneous occurrence of this accompanying action with the corresponding leading action; if the combination factor does not apply, ψ_i =1.

$F_{k,i}$ is the characteristic value of action *i*

γ_R is a partial factor covering uncertainty in the resistance model, plus geometric
deviations if these are not modelled explicitly

$x_{k,I}$ is the characteristic value of material property *i*

γ_m is the partial material factor which allows for uncertainty in the material property

7.2.2 When considering an ultimate limit state of static equilibrium of the structure it shall be verified that:

$$E_{d,dst} \leq E_{d,stb} \quad (4)$$

where

$E_{d,dst}$ the design value of the effect of destabilising actions

$E_{d,stb}$ the design value of the effect of stabilising actions

7.2.3 Where appropriate the equation for the limit state of static equilibrium may be supplemented by additional terms, including for example a term for friction between rigid bodies.

7.3 Combination of actions

7.3.1 General

7.3.1.1 For each critical load case, the design values of the effects of actions (E_d) shall be determined by combining the values of actions that are considered to occur simultaneously.

7.3.1.2 The fundamental combination of actions for use in the verification of the ultimate limit state is given by equation (5):

$$\sum_{j\geq 1}\gamma_{\mathrm{G,j}}\times G_{\mathrm{k,j}}\ \text{"+"}\ P\ \text{"+"}\ \gamma_{\mathrm{Q,1}}\times Q_{\mathrm{k,1}}\ \text{"+"}\ \sum_{i>1}\gamma_{\mathrm{Q,i}}\times\psi_{\mathrm{i}}\times Q_{\mathrm{k,i}}\ \text{"+"}\ A_{\mathrm{d}}$$

where

"+" implies “to be combined with”

$\sum$ implies ”the combined effect of”

$\gamma_{\mathrm{G,j}}$ the partial factors for the permanent action j

$G_{\mathrm{k,j}}$ the characteristic value of permanent action j

P the relevant representative value of the prestressing action

$\gamma_{\mathrm{Q,1}}$ the partial factors for the leading variable action

$Q_{\mathrm{k,1}}$ the characteristic value of the leading variable action

$\gamma_{\mathrm{Q,i}}$ the partial factor for the accompanying variable action i

$Q_{\mathrm{k,i}}$ the characteristic value of the accompanying variable action i

Ψ_{i} the action combination factor corresponding to the accompanying variable action i

A_{d} the design value of the accidental action

7.3.1.3 The partial factors for actions γ_{F} are specified in Table 3 for the various ultimate limit states from 7.1.1.

Table 3 Partial factors for actions for the ultimate limit state

1	2	3	4	5	6	7	8	9	10	11	12	13	14
Type of action			**Part**	**Partial action factor** γ_F									
				Ultimate limit state									
				STR		**STR-P**		**EQU**		**GEO**		**ACC**	
				Un-F[4]	**F**[5]	**Un-F**[4]	**F**[5]	**Un-F**[4]	**F**[5]	**Un-F**[4]	**F**[5]	**Un-F**[4]	**F**[5]
Permanent Actions	Self-weight		2	1,2	0,9	1,35	-	1,2	0,9	1,0	1,0	1,0	
	Geotechnical actions	Soil parameters un-factored	5	1,2	0,9	1,35	-	not applicable				1,0 $^{(1)}$	
		Soil parameters factored		not applicable				1,0		1,0			
	Loads from fluids with a physical control on the maximum fluid level.		5	1,2	0	1,35	0	1,2	0	1,0	0	1,0	0
	Imposed deformations due to pre-stressing			1,0	1,0	1,0	1,0	$^{(2)}$		$^{(2)}$		1.0	
	Other imposed permanent deformations (e.g. settlement)			1,2	-	1,2	-	$^{(2)}$		$^{(2)}$		1.0	
Variable Actions	Imposed loads: floors and roofs		2	1,6	0	1,0	0	1,6	0	1,3	0	1,0	
	Wind action		3	$1{,}3^{(3)}$	0	1,0	0	$1{,}3^{(3)}$	0	$1{,}3^{(3)}$	0	1,0	
	Imposed variable deformation (e.g. temperature)		1; 7	1,6		1,0	0	$^{(2)}$		$^{(2)}$		1.0	
	Overhead travelling cranes and machinery		6	1,6	0	1,0	0	1,6	0	1,3	0	1,0	
	Geotechnical actions:	Soil parameters un-factored	5	1,6	0	1,0	0	not applicable				1,0 $^{(1)}$	
		Soil parameters factored		not applicable				1,0		1,0			
	Loads from fluids that vary with time.		5	1,6	0	1,0	0	1,6	0	1,3	0	1,0	0
	Other types of variable loads not considered above (for example material loads) in the absence of more detailed information			1,6	0	1,0	0	1,6	0	1,3	0	1,0	0
	Accidental and seismic actions		1; 4	not applicable								1,0	

[1] Soil parameters for the Accidental Design Situation are determined according to SANS 10160-5.

[2] Imposed deformations need not be considered in cases where the achievement of the limit state involves large deformations or bodily movement.

[3] For slender non-redundant structures that exhibit significant cross-wind response $\gamma_F = 1{,}5$.

[4] Un-F = Un-favourable

[5] F = Favourable

7.3.2 Combination of actions for structural resistance (STR and STR-P)

7.3.2.1 The combination of actions for persistent or transient design situations for resistance is given by equation (6), using the partial factors γ_G and γ_Q from Table 3 for the STR combination case:

$$\sum_{j\geq 1}\gamma_{G,j}\times G_{k,j} \text{ "+" } P \text{ "+" } \gamma_{Q,1}\times Q_{k,1} \text{ "+" } \sum_{i>1}\gamma_{Q,i}\times\psi_i\times Q_{k,i} \qquad (6)$$

7.3.2.2 equation (7) for permanent actions combined only with the appropriate leading variable action, using the partial factors γ_G and from Table 3 for the STR-P combination case if it results in a more unfavourable effect than that of 7.3.2.1:

$$\sum_{j\geq 1}\gamma_{G,j}\times G_{k,j} \text{ "+" } P \text{ "+" } \gamma_{Q,1}\times Q_{k,1} \qquad (7)$$

NOTE Equation (7) will only result in a more unfavourable action effect in a situation where the permanent action is large compared to the variable actions.

7.3.3 Combination of actions for static equilibrium (EQU)

7.3.3.1 The combination of actions for the persistent or transient design situations for static equilibrium is given by equation (6), using the partial factors γ_G and γ_Q from Table 3 for the EQU combination case, considering unfavourable and favourable actions to be destabilising and stabilising respectively according to 7.2.2. This combination also applies where the verification of static equilibrium involves the resistance of structural elements.

7.3.3.2 Where a distinction has to be made between favourable and unfavourable effects of permanent actions, different partial factors shall be used.

7.3.3.3 Where the results of verification for static equilibrium are very sensitive to variations of the magnitude of a permanent action from place to place in the structure, the unfavourable and the favourable parts of this action shall be considered as separate individual actions.

7.3.4 Combination of actions involving geotechnical actions (GEO)

7.3.4.1 Where geotechnical actions or resistances are present (e.g. footings, basement walls, etc.), the resistance of structural members and the resistance of the ground should be verified by the more severe of the following combinations of actions for geotechnical actions, with factors for soil parameters in accordance with SANS 10160-5:

a) equation (6), using the partial factors γ_G and γ_Q from Table 3 for the STR and STR-P combinations, with geotechnical actions calculated using un-factored soil parameters;

b) equation (6), using the partial factors γ_G and γ_Q from Table 3 for the GEO combination, with geotechnical actions calculated using factored soil parameters.

NOTE In general, structural resistance is governed by Combination a) and the sizing of foundations is governed by Combination b).

7.3.4.2 Consideration of geotechnical actions for static equilibrium according to 7.2.2 should be verified using factored soil parameters in accordance with SANS 10160-5.

7.3.5 Combination of actions for accidental and seismic design situations (ACC)

7.3.5.1 The combination of actions for the accidental and seismic design situations should be expressed as, using the partial factors γ_G and γ_Q from Table 3 for the ACC combination case:

$$\sum_{j\geq 1}\gamma_{\mathrm{G,j}}\times G_{\mathrm{k,j}} \text{ "+" } P \text{ "+" } A_{\mathrm{d}} \text{ "+" } \sum_{i\geq 1}\gamma_{\mathrm{Q,i}}\times\psi_{\mathrm{i}}\times Q_{\mathrm{k,i}} \tag{8}$$

where

A_{d} is the design value of the accidental or seismic action

7.3.5.2 Combinations of actions for accidental design situations should either:

a) involve an explicit accidental action A_{d} (such as impact) or

b) refer to a situation after an accidental event (in this case take A_{d}=0).

7.3.5.3 If sufficient ductility for structural resistance can be provided the design value for R_{d} in accordance with equation (2) should be determined taking the partial factors γ_{R} and γ_{m} as 1,0.

7.4 Strategies for accidental design situations

7.4.1 Use of consequence classes

7.4.1.1 The strategies for accidental design situations should be based on the following consequence classes set out in annex B :

a) CC1 Low consequence of failure

b) CC2 Medium consequence of failure

c) CC3 High consequence of failure

d) CC4 Very high consequence of failure

NOTE 1 : In some cases it might be appropriate to treat some parts of the structure as belonging to a different consequence class (e.g. a structurally separate low rise wing of a building that is serving a less critical function than the main building).

NOTE 2 : The effect of preventive and/or protective measures is that the probability of damage to the structure is removed or reduced. For design purposes, this can sometimes be taken into consideration by assigning the structure to a lower consequence class. In other cases, reduction of forces on the structure may be more appropriate.

NOTE 3 : A suggested classification of buildings into consequence classes relating to buildings is provided in Annex B, Table B.1.

7.4.1.2 Accidental design situations for the different consequence classes may be considered in the following manner :

a) CC1 : no specific consideration is necessary with regard to accidental actions except to ensure that the robustness and stability rules given in material based design standards (see clause 2) as applicable, are adhered to;

b) CC2 : depending upon the specific circumstances of the structure, a simplified analysis by static equivalent action models may be adopted or prescriptive design/detailing rules may be applied;

c) CC3 : in addition to static equivalent action models and prescriptive design/detailing rules, collapse upon notional removal of structural elements should be considered;

d) CC4 : a systematic risk assessment will be required, making use of refined methods such as dynamic analysis, non-linear models and load-structure interaction, if considered appropriate.

7.4.2 Strategies for unidentified accidental design situations

7.4.2.1 In the design the potential damage to the structure arising from an unspecified cause shall be mitigated.

NOTE Guidance for design of building structures for consequences of localized failure due to unspecified causes is given in Annex B.

7.4.2.2 The mitigation should be reached by adopting one or more of the following strategies :

a) designing the structure so that neither the whole structure nor a significant part of it will collapse if a local failure (e.g. single element failure or damage) occurs,

NOTE An indicative limit of local failure for building structures is 100 m^2 on two adjacent floors caused by the removal of any supporting column or wall.

b) designing key elements, on which the structure is particularly reliant, to sustain effects of a notional model of accidental action A_d For building structures a key element should be capable of sustaining a notional accidental design action of A_d = 34 kN/m^2 applied in horizontal and vertical directions (in one direction at a time on a specific member) to the member and any attached components having regard to the ultimate strength of such components and their connections. Such notional accidental design loading should be applied simultaneously with normal loading. This will require the use of rules for combination of actions given in 7.3.5.

NOTE Although this accidental action is derived from a load model representing an internal gas explosion, it is applied in this case as a notional load to test robustness and integrity of a structure.

c) applying prescriptive design/detailing rules that provide acceptable robustness for the structure (e.g. three-dimensional tying for additional integrity, or minimum level of ductility of structural elements subject to impact).

7.4.3 Strategies for identified accidental design situations

7.4.3.1 The accidental actions to be taken into account depend upon :

a) The measures taken for preventing or reducing the risk from accidental action

b) The probability of occurrence of the identified accidental action

c) The consequences of the identified accidental action

d) The level of acceptable risk

NOTE 1 : In practice, the occurrence and consequences of accidental actions can be associated with a certain risk level. If this level cannot be accepted, additional measures are necessary. A zero risk level, however, is unlikely to be reached and in most cases it is necessary to accept a certain level of risk. Such a risk level will be determined by various factors such as the potential number of casualties involved, economic consequences and the cost of safety measures etc.

NOTE 2 : A comparison with risks generally accepted by society in comparable situations provide some guidance on an acceptable level of risk (Refer to EN 1991-1-7 annex B). The risk implied by the levels of reliability for structures specified in the SANS 10160 series can serve as reference to such a comparison. Note that the acceptable level of risks for structures forms the upper limit of acceptable risk for accidental design situations.

7.4.3.2 Localised damage due to accidental actions may be acceptable, provided that it will not endanger the structure and that the overall load-bearing capacity is maintained during an appropriate length of time to allow necessary emergency measures to be taken.

7.4.3.3 In the case of building structures such emergency measures may involve the safe evacuation of persons from the premises and its surroundings.

7.4.3.4 Measures to mitigate the risk of accidental actions should include, as appropriate, one or more of the following strategies :

a) preventing the action from occurring or reducing the probability and/or magnitude of the action to an acceptable level through the structural design process (e.g. providing sacrificial venting components with a low mass and strength to reduce the effect of internal explosions),

b) protecting the structure against the effects of an accidental action by reducing the actual loads on the structure (e.g. protective bollards or safety barriers),

c) ensuring that the structure has sufficient robustness by adopting one or more of the following approaches :

1) by designing certain key components, on which the stability of the structure depends, to be of enhanced strength so as to raise the probability of their survival following an accidental action.

2) by designing structural members to have sufficient ductility, capable of absorbing significant strain energy without rupture.

3) by incorporating sufficient redundancy in the structure, so as to facilitate the transfer of actions to alternative load paths following an accidental event.

NOTE 1 : The effect of preventing actions may be limited; particularly if it is dependent upon factors which, over the life span of the structure, are commonly outside the control of the structural design process. Preventative measures often involve periodic inspection and maintenance during the life of the structure.

NOTE 2 : For the design of structural members with sufficient ductility, see annex B together with materials based standards (see clause 2).

7.4.3.5 The accidental actions shall, where appropriate, be applied simultaneously in combination with other permanent and variable actions as given in 7.3.5.

7.4.3.6 Where more unfavourable effects are obtained by the omission of variable actions as a whole, or in part, this shall be taken into account. Consideration shall also be given to the safety of the structure immediately following the occurrence of the accidental action.

NOTE : This includes the consideration of progressive collapse. (See annex B).

8 Serviceability limit states design verification

8.1 Criterion of failure

When considering a serviceability limit state it shall be verified that:

$$E_{\mathrm{d}} \leq C_{\mathrm{d}} \qquad (9)$$

where

E_{d} is the design value of the effect of actions specified in the serviceability criterion, determined on the basis of the relevant combination

C_{d} is the limiting design value of the relevant serviceability criterion

8.2 Serviceability criteria

8.2.1 General

8.2.1.1 Specific serviceability criteria should cover, for example, floor stiffness, differential floor levels, storey sway or/and building sway and roof stiffness. Stiffness criteria should be expressed in terms of limits for vertical deflections and for vibrations. Sway criteria should be expressed in terms of limits for horizontal deflections.

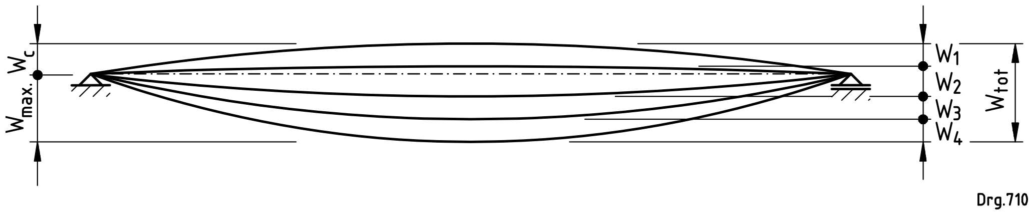
8.2.1.2 For other specific serviceability criteria such as crack width, stress or strain limitation, slip resistance, etc. refer to the structural materials design standards.

8.2.2 Deflections and deviations

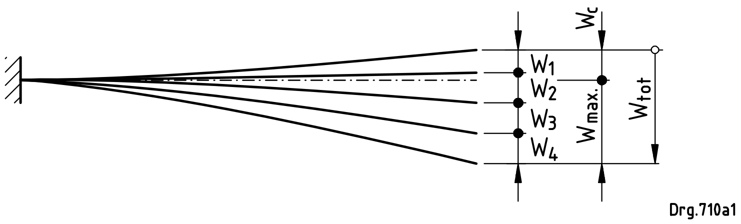
Vertical and horizontal deflections should be calculated in accordance with the structural materials design standards (see clause 2, by using appropriate combinations of actions according to 8.3 taking into account the serviceability requirements given in 5.4. Special attention should be given to the distinction between irreversible and reversible limit states.

NOTE 1 : A distinction is made between deflection, which is the movement of a defined point in a defined direction, and a deviation, which is the distance of a defined point from a defined datum.

NOTE 2 : Vertical and horizontal deflections are represented schematically in Figure.1(a) & (b) and Figure 2 respectively.



(a) Medial deflections



(b) Terminal deflections

Figure 1 — Vertical deflections

where

w_c is the pre-camber in the unloaded structural member

w_1 is the initial part of the deflection under structural self-weight

w_2 is the initial part of the deflection under non-structural self-weight

w_3 is the additional part of the deflection due to the variable actions (short-term)

w_4 is the long-term part of the deflection under permanent and quasi-permanent leads (creep-deflection)

w_{tot} is the total deflection as sum of w_1, w_2, w_3 and w_4

w_{max} is the deviation of the respective middle or end point of the member from a reference position

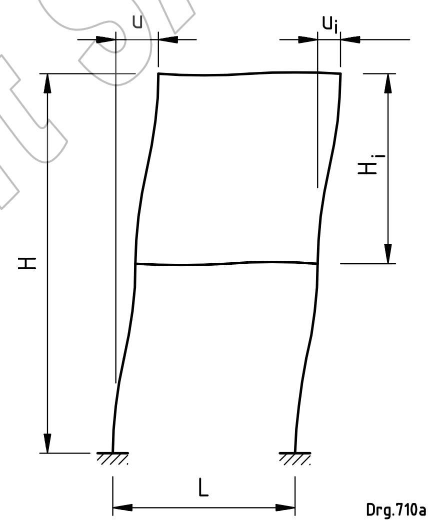


Figure 2 — Definition of horizontal deflections

where

u is the overall horizontal displacement over the building height H
u_i is the horizontal displacement over the storey height H_i for storey i

In the same way as the vertical deflections any horizontal deflection u or u_i can also be subdivided into the following components of deflections:

u_1 is the initial part of the deflection under structural self-weight
u_2 is the initial part of the deflection under non-structural self-weight
u_3 is the additional part of the deflection due to the variable actions (short-term)
u_4 is the long-term part of the deflection under permanent and quasi-permanent leads (creep-deflection)

8.2.3 Deflection and deviation effects

8.2.3.1 All aspects of deflections and their effects on the serviceability of the structure should be considered

NOTE Annex D provides assistance to identify those aspects of deformation that affect the suitability of a building for the purposes for which it is intended, and to suggest criteria by which the performance of the building can be assessed.

8.2.3.2 If the functioning of or damage to the structure or to finishes, or to non-structural members (for example partition walls, claddings) is being considered, the verification for deflections should take account of those effects of permanent and variable actions that occur after the execution of the member or finish concerned.

NOTE : It may be necessary to differentiate between the components of the deflection due to the structural self-weight and the non-structural self-weight .

8.2.3.3 If the appearance of the structure is being considered, the effects of the permanent and quasi-permanent values of the variable actions should be used. The appearance of the structure is considered in terms of deviations.

8.2.3.4 If the wellbeing of the user or the functioning of machinery is being considered, the verification should take account of the effects of the relevant variable actions and the resulting deviations.

8.2.3.5 Long term deformations due to shrinkage, relaxation or creep should be considered where relevant, and calculated by using the effects of the permanent actions and quasi-permanent values of the variable actions.

8.2.4 Deflection and deviation limits

8.2.4.1 The serviceability criteria should be specified in accordance with the guidelines provided in annex C. Deviations from these guidelines may be considered when justified.

8.2.4.2 Deformation limits recommended in the structural materials-based design standards should be considered..

NOTE 1 : The deformation limits recommended in the structural materials-based design standards are only be applied in cases where they are more stringent than the deformation limits recommended in SANS 10160.

NOTE 2 : In addition to the functional serviceability criteria specified in SANS 10160, specific materials-based serviceability criteria are to be taken into account, for example crack width limitations on concrete structures.

8.2.5 Vibrations

8.2.5.1 For the serviceability limit state of a structure or a structural member not to be exceeded when subjected to vibrations, the natural frequency of vibrations of the structure or structural member should be kept above acceptable values which depend on the building and the source of the vibration, and agreed with the client or relevant authority.

8.2.3.2 If the natural frequency of vibrations of the structure is lower than the acceptable value, a more refined analysis of the dynamic response of the structure, including the consideration of damping, should be performed.

8.2.3.3 Possible sources of vibration that should be considered include walking, synchronised movements of people, machinery, ground borne vibrations from traffic, and wind actions. These and other sources should be specified for each project and agreed with the client.

8.3 Combination of actions

8.3.1 Combination of actions for irreversible serviceability

8.3.1.1 The combination of actions for irreversible serviceability limit states should be expressed as:

$$\sum_{j\geq 1} \gamma_{\mathrm{G,j}} \times G_{\mathrm{k,j}} \text{ "+" } P \text{ "+" } \gamma_{\mathrm{Q,1}} \times Q_{\mathrm{k,1}} \text{ "+" } \sum_{i>1} \gamma_{\mathrm{Q,i}} \times \psi_{\mathrm{i}} \times Q_{\mathrm{k,i}} \qquad (10)$$

where

$\gamma_{\mathrm{G,j}}$ = 1,1 for unfavourable and
= 1,0 for favourable permanent actions due to self-weight j considering the permanent actions which are relevant, according to 8.2.3.2

γ_{Q} = 0,6 for wind loads

$\gamma_{\mathrm{Q,i}}$ = 1,0 for all other imposed loads i

8.3.1.2 The contribution of material creep effects to the irreversible serviceability limit state should be determined according to 8.3.3.

8.3.2 Combination of actions for reversible serviceability

The combination of actions for reversible serviceability limit states should be expressed as:

$$\sum_{j\geq 1} \gamma_{\mathrm{G,j}} \times G_{\mathrm{k,j}} \text{ "+" } P \text{ "+" } \sum_{i>1} \psi_{\mathrm{i}} \times Q_{\mathrm{k,i}} \qquad (11)$$

where

$\gamma_{G,j}$ = 1,1 for unfavourable and
= 1,0 for favourable permanent actions due to self-weight

8.3.3 Combination of actions for long-term and appearance serviceability

8.3.3.1 For long-term serviceability limit states equation (11) including the specified partial factors should be applied.

8.3.3.2 For serviceability limit states considering the appearance of the structure equation (11) should be applied, in accordance with 8.3.2.

NOTE For a structure where its appearance is sensitive to cracking, the application of equation (10) may be considered for the combination of actions.

8.3.3.3 Although the construction sequence should be considered to determine which permanent actions *j* are to be included in equation (11), the contribution of all permanent actions contributing to material creep effects should be taken into account.

9 Design assisted by testing

9.1 Design may be based on a combination of tests and calculations.

NOTE 1 Testing may be carried out, for example, in the following circumstances :

- if adequate calculation models are not available
- if a large number of similar components are to be designed and constructed, to conform by control checks, assumptions made in the design,
- if doubts exist about the adequacy of the design or construction of an existing building.
- if doubts exist about the adequacy of a building that is under construction
- if damage or deterioration due to fire or other causes exists,
- if the loading conditions have changed in an existing building with or without structural records ,
- if no design information is available of an existing building or structural component.

NOTE 2 Refer to annex E for more information.

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9.2 Design assisted by test results shall conform to the level of reliability required for the relevant design situation. The statistical uncertainty due to a limited number of test results shall be taken into account.

9.3 Partial factors, including those for model uncertainties, comparable to those used in the materials-based design standards shall be used.

9.4 The planning, execution and evaluation of load tests in accordance with the principles and guidelines shall be carried out by competent persons.

Annex A
(informative)

Management of structural reliability

A.1 Scope and field of application

A.1.1 Additional guidance to 4.4 is provided and to SANS 10160, together with the structural materials design standards (see clause 2).

NOTE Reliability differentiation rules may be specified for particular aspects in the structural materials-based design standards.

A.1.2 The approach given recommends the following procedures for the management of structural reliability for structures with regards to ultimate limit states, excluding fatigue :

a) With reference to 4.4.1.2 and 4.4.1.4, classes are introduced and are based on the assumed consequences of failure and the exposure of the structures to a hazard. A procedure for allowing moderate differentiation in the partial factors for actions and resistances corresponding to the classes is given in A.3.

NOTE Reliability classification can be represented by β-index values which takes account of accepted or assumed statistical variability in action effects and resistances and model uncertainties (see EN 1990).

b) With reference to 4.4.1.5(c) and 4.4.1.5(d), a procedure for allowing differentiation between various types of structures in the requirements for quality levels of the design and execution processes are given in A.3 .

NOTE Those quality management and control measures in design, detailing and execution which are given in A.3 , aim to eliminate failures due to gross errors, and ensure that the resistances assumed in the design are achieved.

c) The procedures are formulated in such a way so as to produce a framework to allow different appropriate reliability levels to be used.

A.2 Reliability levels for representative structures

The level of reliability is specified probabilistically in terms of the safety index β_t which is related to the notional probability of failure $p_f = \Phi(-\beta_t)$ where Φ is the cumulative normal distribution function. The specification is formulated to achieve the following levels of reliability for a 50 year reference period or design working life :

a) For the reference class of structures representing a medium level of consequences of structural failure with ductile and gradual modes of failure, the minimum level of reliability is expressed in terms of $\beta_t > 3{,}0$ or $p_f < \Phi\,(-3{,}0) \approx 0{,}001$.

b) For the reference class of structures but with brittle and sudden modes of failure, the minimum level of reliability is $\beta_t > 4{,}0$.

c) For the failure at connection details between components of the reference class of structures, the minimum level of reliability is $\beta_t > 4{,}5$.

A.3 Reliability differentiation

A.3.1 General

A.3.1.1 Reliability differentiation provided for in 4.4.2.3 may be applied to take account of reliability classes for buildings and structures, design supervision, inspection during execution and inspection classes.

A.3.2 Differentiation by measures relating to partial factors

A.3.2.1 For the purpose of reliability differentiation, reliability classes (RC) may be established by considering the consequences of failure or malfunction of the structure as given in Table A.1. Differentiated values for the safety index β_t are also tabulated. RC2 serves as Reference Class for which reliability verification procedures are generally specified in SANS 10160.

A.3.2.2 For the same design supervision and inspection levels, a multiplication factor K_F as given in Table A.1 may be applied to the partial factors for actions (γ_F) to be used in fundamental combinations for persistent design situations. K_F should be applied only to unfavourable actions.

NOTE Other measures as described are normally preferred to using K_F in particular for classes RC3 and RC4.

Table A.1 — Reliability classification according to function of facility or risk of failure

1	2	3	4	5
Reliability class	**Function of facility, probability or consequence of failure**	**Examples**	Minimum level of reliability β_t	Multiplication factor K_F
RC1	**Low** for loss of human life, economic, social OR **small or negligible** for environmental consequences	Agricultural buildings with infrequent human occupancy (for example storage buildings, greenhouses)	2,5	0,9
RC2	**Moderate** for loss of human life, economic, social OR **considerable** for environmental consequences	Residential and office buildings, public buildings where consequences of failure are moderate (for example office building)	3,0	1,0
RC3	**High** for loss of human life, OR **Very great** for economic, social or environmental consequences	Grandstands, public buildings where consequences of failure are high (for example concert halls)	3,5	1,1
RC4	**Post-disaster** function or consequences beyond	Hospitals, communication centres, fire and rescue centres	4,0	1,2

	the boundaries of the facility			

A.3.2.3 Depending on the structural form and decisions made during design, particular members of the structure may be designated in the same, higher or lower reliability class than for the entire structure.

A.3.2.4 Reliability differentiation may also be applied through partial factors on resistance γ_M. However this is not normally used, with the exception of fatigue verification. See also A.3.5.

A.3.2.5 Accompanying measures, for example the level of quality control for the design and execution of the structure, may be associated with the reliability class. A four level system of control during design and execution, as in clauses A3.3 and A3.4 may be applied. Design supervision levels and inspection levels associated with the reliability classes are suggested.

A.3.3 Design supervision differentiation

Four possible design supervision levels (DSL) are shown in Table A.2. The design supervision levels may be linked to the reliability class selected or chosen according to the importance of the structure or the design brief, and implemented through appropriate quality management measures.

Table A.2 — Design supervision level (DSL)

1	2	3
Design supervision level	**Characteristics**	**Minimum recommended requirements for checking of calculations, drawings and specifications**
DSL1 relating to RC1	Basic supervision	Self-checking: Checking performed by the person who has prepared the design
DSL2 relating to RC2	Normal supervision	Procedural separate checking: Checking by different persons than those originally responsible and in accordance with the procedure of the organisation
DSL3 relating to RC3	Extended supervision	Third party checking: Checking performed by an organisation different from that which has prepared the design
DSL4 relating to RC4	Regulated supervision	Regulated third party checking: Checking performed to satisfy requirements of regulatory or supervisory authority

A.3.4 Inspection during execution

Four inspection levels (IL) may be introduced as shown in Table A.3. The inspection levels may be linked to the quality management classes selected and implemented through appropriate quality management measures.

Table A.3 — Inspection levels (IL)

1	2	3
Inspection level	**Characteristics**	**Minimum recommended requirements for inspection levels during execution**
IL1 relating to RC1	Basic inspection	Self inspection
IL2 relating to RC2	Normal inspection	Inspection in accordance with the procedures of the organisation

IL3 relating to RC3	Extended inspection	Third party inspection
IL4 relating to RC4	Regulated inspection	Regulated third party inspection: Inspection performed to satisfy requirements of regulatory or supervisory authority

A.3.5 Partial factors for resistance properties

A partial factor for material or product property or a member resistance may be reduced if an inspection class higher than that required according to Table A.3 or more severe requirements (or both) are used.

NOTE Such a reduction, which allows for example for model uncertainties and dimensional variation, is not a reliability differentiation measure: it is only a compensating measure in order to keep the reliability level dependent on the efficiency of the control measures.

Annex B
(Informative)

Design for consequences of localised failure due to unspecified causes

B.1 Scope and field of application

Rules and methods are provided for designing buildings to sustain an extent of localised failure from an unspecified cause without disproportionate collapse. Whilst other approaches may be equally valid, adoption of this strategy is likely to ensure that the building is sufficiently robust to sustain a limited extent of damage or failure, depending on the consequence class (see 7.4.1), without collapse.

B.2 General requirements

B.2.1 Designing the building such that neither the whole building nor a significant part of it will collapse if localised damage were sustained, is an acceptable strategy, as stated in 7.4.2, for ensuring that the building is sufficiently robust to survive a reasonable range of unidentified accidental actions.

B.2.2 The minimum period that a building needs to survive following an accident should be that needed to facilitate the safe evacuation and rescue of personnel from the building and its surroundings. Longer periods of survival may be required for buildings used for handling hazardous materials, provision of essential services, or for national security reasons.

B.3 Consequence classes of buildings

Table B.1 provides a categorization of building types in terms of occupancies to consequence classes. This categorisation relates to the low, medium, high and very high consequence classes given in 7.4.1.1.

Table B.1 — Examples categorisation of consequence classes

1	2
Consequence class	**Building type and occupancy**
CC1 low risk group	a) Single occupancy residential buildings not exceeding 3 storeys. b) Agricultural buildings. c) Buildings which people rarely enter, provided no part of the building is closer to another building, or area which people frequently enter, than a distance of 1.5 times the building height.
CC2 medium risk group	a) Buildings not exceeding 4 storeys with residential occupancies, flats or apartments, hotels or offices. b) Industrial buildings not exceeding 3 storeys. f) Retailing premises not exceeding 3 storeys of less than 1000 m^2 floor area in each storey. c) Educational buildings not exceeding 2 storeys. d) All buildings not exceeding 2 storeys to which members of the public are admitted and which contain floor areas not exceeding 2000 m^2 in each storey.
CC3 high risk group	a) Hotels, flats, apartments and other residential buildings, office buildings, with more than 4 storeys but not exceeding 15 storeys. b) Retailing premises with more than 3 storeys but not exceeding 15 storeys. c) Educational buildings exceeding 2 storeys storey but not exceeding 15 storeys. d) Hospitals not exceeding 3 storeys. e) All buildings to which members of the public are admitted and which contains floor areas exceeding 2000 m^2 but not exceeding 5000 m^2 in each storey. f) Stadiums accommodating less than 5000 spectators.
CC4 very high risk group	a) All buildings defined above as CC2 and CC3 that exceed the limits on area and/or number of storeys. b) All buildings to which members of the public are admitted in significant numbers. c) Stadiums accommodating more than 5000 spectators.
NOTE 1 For buildings intended for more than one type of use, the highest applicable consequence class should be selected. NOTE 2 m In determining the number of storeys, basement storeys may be excluded provided such basement storeys fulfil the requirements of CC3 risk group. NOTE 3 m Table B.1 is not exhaustive.	

B.4 Recommended strategies

Adoption of the following recommended strategies should ensure that the building will have an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse :

a) For buildings in CC1 : provided the building has been designed and constructed in accordance with the rules given in the materials-based standards (see clause 2) satisfying stability in normal use, no further specific consideration is necessary with regard to accidental actions from unidentified causes;

b) For buildings in CC2 : Effective horizontal ties, or effective anchorage of suspended floors to walls, as defined in B.5.1 for framed construction and B.5.2 for load-bearing wall construction should be provided;

c) For buildings in CC3 : Two alternative strategies are :

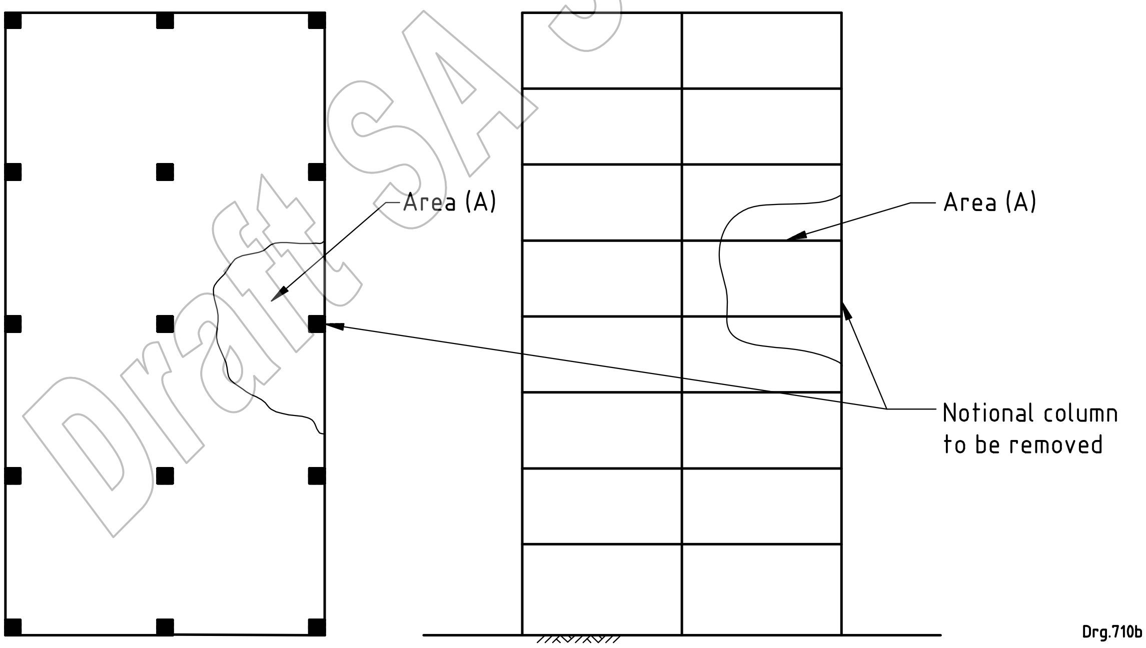
i) providing effective horizontal ties, as defined in B.5.1 for framed construction and B.5.2 for load-bearing wall construction, together with effective vertical ties, as defined in B.6, in all supporting columns and walls

ii) checking the building to ensure that upon the notional removal of each supporting column and each beam supporting a column, or any nominal section of load-bearing wall as defined in B.7 below, (one structural element at a time in each storey of the building) that the building remains stable and that any local damage does not exceed a certain limit, (see NOTE 2).

Where the notional removal of such column, beam or section of wall would result in an extent of damage in excess of the above limit, or other such specified limit, then such elements should be designed as key elements (see B.8).

In case of buildings of load bearing wall construction, the notional removal of a section of the wall, one at a time, is likely to be the most practical strategy to adopt.

NOTE 1 The limit of admissible local damage may be different for each type of building. The ecommended value is 15% of the floor, or 100 m^2, whichever is smaller, in each of two adjacent storeys. See Figure B.1



Key :
(A) local damage not exceeding 15% of the floor area in each of two adjacent storeys

Figure B.1 — Indicative area of local damage

NOTE 2 In case of buildings of load bearing wall construction, the notional removal of a section of the wall, one at a time, is likely to be the most practical strategy to adopt.

d) For buildings in CC4 a systematic risk assessment of the building should be undertaken taking into account both foreseeable and unforeseeable hazards.

NOTE For guidance on the preparation of a risk assessment, refer to En 1991-1-7.

B.5 Effective horizontal ties

B.5.1 Framed structures

B.5.1.1 Effective horizontal ties should be provided around the perimeter of each floor and roof level and internally in two right angle directions to tie the column and wall elements securely to the structure of the building. The ties should be continuous and be arranged as closely as practicable to the edges of floors and lines of columns and walls. At least 30% of the ties should be located within proximity to the lines of columns and walls.

B.5.1.2 Effective horizontal ties may comprise rolled steel sections, steel reinforcement in concrete slabs, or steel mesh reinforcement and profiled steel sheeting in composite steel/concrete floors (if directly connected to the steel beams with shear connectors), or a combination of the above types.

B.5.1.3 Each continuous tie, including its end connections, should be capable of sustaining a design tensile load for accidental limit state of T_i in the case of internal ties, and T_p in the case of perimeter ties, equal to the following values :

for internal ties, $T_i = 0{,}8(g_k + \psi \times q_k)s \times L$ or 75 kN, whichever the greater

for perimeter ties, $T_p = 0{,}4(g_k + \psi \times q_k)s \times L$ or 75 kN, whichever the greater.

where

g_k is the characteristic self-weight

q_k is the characteristic imposed load

s is the spacing of the ties

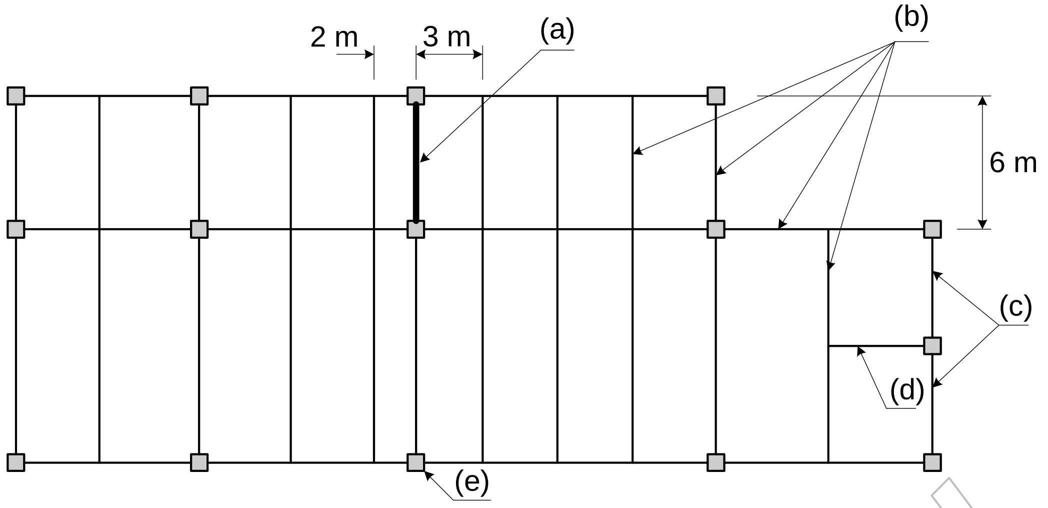
L is the span of the tie

ψ is the combination factor according to the accidental load combination

NOTE See Example B.1.

EXAMPLE B.1

Calculating the accidental design tensile force T_i in a 6.0 m span internal beam as indicated as beam (a) in figure B.2.



Key :

(a) 6.0 m span beam as internal tie
(b) All beams designed to act as ties
(c) Perimeter ties
(d) Tie beam anchoring the column
(e) Edge column

Figure B.2 — Plan view of floor and beam lay-out

Characteristic floor loading : $q_k = 5.0$ kN/m^2 and $g_k = 3.0$ kN/m^2

$$T_i = 0{,}8(g_k + \psi \times q_k)s \times L = 0{,}8(3{,}0 + 0{,}5 \times 5{,}0)\frac{3+2}{2} \times 6{,}0 = 66{,}0 \text{ kN}$$

which is less than 75 kN, therefore $T_i = 75$ kN.

B.5.1.4 Members used for sustaining non-accidental loading may be utilised for the above ties without consideration of the combination of actions as given in 8.3.

B.5.2 Load-bearing wall construction

B.5.2.1 For CC2 buildings : Appropriate robustness should be provided by adopting a cellular form of construction designed to facilitate interlocking of all components including an appropriate means of anchoring the floor to the walls.

B.5.2.2 For CC3 buildings : Continuous effective horizontal ties should be provided in the floors. These should be internal ties distributed throughout the floors in both orthogonal directions and peripheral ties extending around the perimeter of the floor slabs within a 1,20 m width of slab. The design tensile load in the ties should be determined as follows :

a) For internal ties:

T_i the greater of F_t [kN/m] or

$$\frac{F_t(g_k + \psi \times q_k)}{7{,}5} \times \frac{z}{5} \text{ [kN/m]}$$

where

F_t is 60 kN/m or (20+4 n_s) kN/m, whichever is less

n_s is the number of storeys

z is the lesser of :

5 times the clear storey height H_c, or

the greatest distance in metres in the direction of the tie, between the centres of the columns or other vertical load-bearing members whether this distance is spanned by :

- a single slab or
- by a system of beams and slabs or

b) For peripheral ties $T_p = F_t$

where

F_t = 60 kN or (20 + 4 n_s) kN, whichever is less

NOTE Factors H_c and z are illustrated in Figure B.3

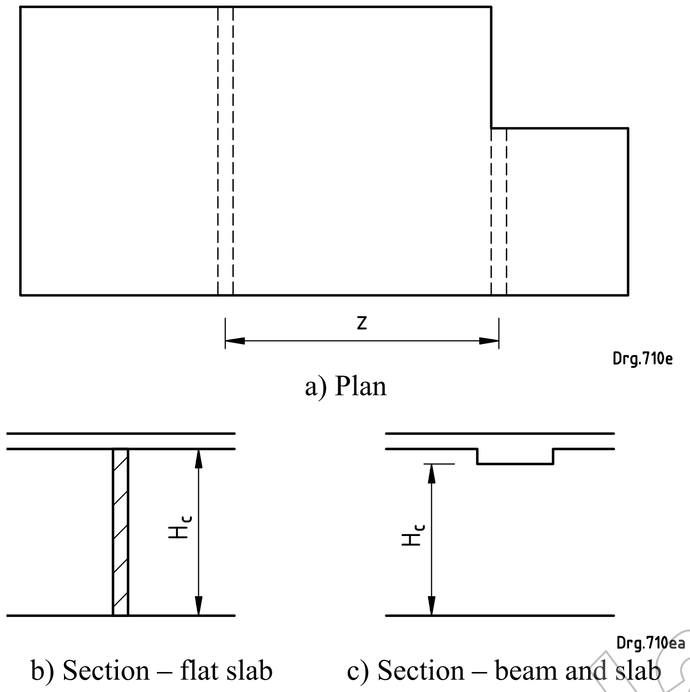


Figure B.3 — Illustration of factors H_c and z

B.6 Effective vertical ties

B.6.1 Each column and wall should be tied continuously from the foundation to the roof level.

B.6.2 In the case of framed buildings (e.g. steel or reinforced concrete structures) the columns and walls carrying vertical actions should be capable of resisting an accidental design tensile force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey. Such accidental design loading should not be assumed to act simultaneously with non-accidental loads.

B.6.3 In case of load-bearing wall construction, the vertical ties may be considered effective if all of the following are satisfied :

a) In the case of masonry walls their thickness is at least 150 mm, and if they have a minimum compressive strength of 5 MPa.

b) The clear height of the wall, H_c, measured in metres between faces of floors or roof does not exceed $20\,t$, where t is the thickness of the wall in metres.

c) The vertical force T that should be resisted by the tie is :

$$T = \frac{34t}{8}\left(\frac{H_c}{t}\right)^2$$ [kN/m], or 100 kN/m of wall, whichever is the greater.

where :

H_c is the clear height of the wall (m)
t is the thickness of the wall (m)

d) The vertical ties are grouped at 5 m maximum centres along the wall and occur no greater than

2,5 m from an unrestrained end of wall.

B.7 Nominal section of load bearing wall

The nominal length of load-bearing wall construction referred to in B.4 (c) (ii) should be taken as follows :

a) in the case of a reinforced concrete wall, a length of not exceeding 2,25 H_c

b) in case of an external masonry wall, timber or steel stud wall, the length measured between vertical lateral supports

c) in the case of an internal masonry wall, timber or steel stud wall, a length not exceeding 2,25 H_c

where H_c is the storey height in metres.

B.8 Key elements

For building structures a key element, as referred to in B.4 (c) , should be capable of sustaining an accidental design action of A_d = 34 kN/m^2 (see 7.4.2.2 (b)) applied in horizontal and vertical directions (in one direction at a time) to the member and any attached components having regard to the ultimate strength of such components and their connections. Such an accidental design action should be considered an accidental load and applied in accordance with the rules for combination of accidental actions given in 7.3.5.

Annex C
(Informative)

Recommended criteria for deformation of buildings

C.1 General

C.1.1 Deflection criteria is presented that may be used in accordance with the requirements for design verification for the serviceability limit state as stipulated in clause 8.

C.1.2 The various actions and their effects, and other sources of deflection to which the criteria apply, are also indicated.

C.1.3 The deflection criteria are based on a general discussion of the causes, effects, remedies and methods for assessment which are presented in annex D, as adapted from ISO 4356.

C.1.4 The effect of deflections on the stability of the structure should be considered in the materials based codes. See annex D.6 for a general discussion on the effect of deflections on strength and stability.

C.2 Irreversible serviceability limit state

C.2.1 The recommended criteria for the irreversible serviceability limit state are given in table C.1 (see next page)

Table C.1 — Summary of recommended criteria for the irreversible serviceability limit state

1	2	3	4	5	6	7	8	9	10	11	12
			Actions and deflections [1]								
Deformation	**Effect**	**Criterion**	Construction deviation & Camber	Differential settlement	Structural self-weight	Non-structural self-weight	Pre-stressing	Imposed load	Wind	**Clause**	**Conditions and comments**
Medial deflection of floors	Damage at supports	Span/300			C	E C	C	E C		D.7.2	Also D.6.2
	Ceiling damage	-			C	E C	C	E C		D.7.3	Varies with construction Usually less critical than partitions
	Partition damage Partition isolated from floor (for span/height <3,5)	Span/500 to Span/300			C	E C	C	E C		D.7.4	Partition follows movement of floor beneath
		10 mm									Excessive deflection of floor below partition
		10 to 15 mm									Excessive deflection of floor or roof above partition
Medial deflection of roofs or roof members	Damage at supports	Span/300			C	E C	C	E C		D.6.2.3	
	Ceiling damage	-			C	E C	C	E C		D.7.3	Varies with construction
	Partition damage – isolated (for span/height<3,5)	10 – 15 mm			C	E C	C	E C		D.7.4	Deflection at nodes for truss
	Roof covering damage	Span/250 – Span/125			C	E C	C	E C	E	D.7.5	
Terminal deflection of cantilever floors	Ceiling damage	-			C	E C	C	E C		D.7.3	Varies with construction
	Partition damage	Span/500 – Span/300			C	E C	C	E C		D.7.4	
Terminal deflection of cantilever roofs	Ceiling damage	-			C	E C	C	E C		D.7.3	Varies with construction
	Partition damage - isolated	10 – 15 mm			C	E C	C	E C		D.7.4	Deflections at nodes for truss
	Roof covering damage	Span/250 – Span/125			C	E C	C	E C	E	D.7.5	
Terminal deflection of non-cantilever horizontal members	Damage at supports	Span/100		E C						D.6.2.3	
	Partition damage	Span/500		E C						D.7.4	

Terminal deflection of vertical members	Damage at supports	Storey height/100							E	D.6.2.3	
	Partition damage	Storey height/500							E	D.7.4	
NOTE (1) : Columns 4-10 indicate which actions and displacements must be considered when calculating compliance of the structure with the given criterion — E = elastic effect, C = creep effect											

C.3 Reversible and long-term serviceability limit state

C.3.1 The recommended deflection criteria for the reversible and long-term serviceability limit state are presented in table C.2.

Table C.2 — Recommended criteria for the reversible and long-term serviceability limit state

1	2	3	4	5	6	7	8	9	10	11	12
			Actions and deflections [(1)]								
Deformation	**Effect**	**Criterion**	**Construction deviation & Camber**	**Differential settlement**	**Structural self-weight**	**Non-structural self-weight**	**Pre-stressing**	**Imposed load**	**Wind**	**Clause**	**Comments**
Medial deviation of floors	Appearance	Visible length/250 or 30 mm	Yes		E C	E C		E C		D.8.1	
	Use (curvature)	Span/300	Yes		E C	E C		E C		D.9.1	
Medial deviation of roofs or roof members	Appearance	Visible length/250 or 30 mm	Yes		E C	E C		E C		D.8.1	Quasi-permanent component of imposed load
Terminal deviation of cantilever floors	Appearance	Visible length/250 or 15 mm	Yes	E C	E C	E C		E C		D.8.1	
	Use (curvature)	Span/125	Yes	E C	E C	E C		E C		D.9.1	
	Use (rotation)	Span/100	Yes	E C	E C	E C		E C		D.9.2	
Terminal deviation of cantilever roofs	Appearance	Visible length/250 or 15 mm	Yes	E C	E C	E C		E C		D.8.1	Quasi-permanent component of imposed load
Terminal deviation of non-cantilever horizontal members	Use (slope)	Span/100	Yes	E C						D.9.1	
Terminal deviation of vertical members	Appearance	Storey height/250	Yes	E C	E C	E C		E C		D.8.2	Where self-weight, imposed and wind loads act eccentrically
Oscillations of members	Resonance	-						E	E	D.9.3	
	Use	-						E	E		
Oscillations of building as a whole	Use	-						E	E		
Horizontal terminal deflection of high-rise buildings		Building height/500									

NOTE (1) : Columns 4-10 indicate which actions and displacements must be considered when calculating compliance of the structure with the given criterion — E = elastic effect, C = creep effect

Annex D
(Informative)

Deformation of buildings

D.1 General

D.1.1 The information given is based on information given in ISO 4356.

D.1.2 It will assist the designer to identify those aspects of deformation that affect the suitability of a building for the purposes for which it is intended, and to suggest criteria by which the performance of the building can be assessed. In addition, numerical values for some of these criteria are recommended in order to give some guidance where this might be desired.

NOTE In view of the wide range of acceptable values of some of the criteria, and in view also of the difficulties of estimating deformations, it is believed that some guidance towards uniformity and degree of compliance would be of assistance, particularly as the economics of modern building designs are increasingly controlled by deformation and maintenance during use. Some suggestions are therefore made with regard to the methods for controlling the assessment of deformations.

D.1.3 The recommendations for criteria of deformation, and the recommended limiting values, are summarised in annex C.

D.1.4 Deviations from the recommended numerical limiting values for deflections given may be applied with proper justification. Such a variation may be particularly appropriate for temporary buildings, for buildings with post-disaster functions, for buildings having to satisfy special requirements, and for buildings of unusual type or constructed of unusual materials.

D.2 Scope of application

This annex refers to the deformations at the serviceability limit states of buildings such as dwellings, offices, public buildings, and factories. It does not refer to the deformations of bridges, roads, masts, underground works, non-residential farm-buildings, or special-purpose buildings such as nuclear power stations or industrial plant. Some of the general principles on which this annex is based may nevertheless serve as a guide when the deformations of such other structures are being considered.

NOTE Whilst it is undesirable that the deformations of a building should damage adjacent buildings, or cause inconvenience their occupants or other members of the public, such matters are normally the subject of legislation and are not appropriate to this annex. Nevertheless, attention may here be drawn to the fact that the provision of movement joints between adjacent buildings and the avoidance of interference with neighbouring foundations are normal good building practice.

D.3 Causes of deformation

Deformations are caused by major ground movements, by differential settlements of foundations, by environmental and occupational loads, by pre-stressing forces and by movements of building materials due to creep and change in temperature, moisture content and chemical composition.

D.4 Deformations – effects and remedies

D.4.1 Besides possibly affecting the strength or stability of a structure, deformations may affect serviceability by causing damage to adjacent parts of the building, by disturbing or harming the occupants, or by preventing proper use of the building.

D.4.2 In many such cases the designer may be able to avoid troublesome effects either by removing the original cause, or by taking suitable precautions in the processes of design and construction to permit some or all of the deformation to occur freely, before or after completion of the building, masking the remainder by suitable constructional or decorative treatment. This course of action has the advantage that it avoids the problem of precisely estimating the magnitudes of causes and their effects. It can be adopted when the deformations, and the constructional measures taken, do not conflict with other requirements of the design. Some troubles that may often be dealt with in this way are listed in D.12.

D.4.3 Camber can be used to reduce the final value of deflections. The normal use of camber is to reduce the contribution to deformations that is made by self-weight and other permanent or long-term temporary action. In other cases, the designer may have no option but to provide sufficient stiffness to limit the deformations and thus reduce their effects to acceptable levels; this will inevitably increase the first cost of the structure. The designer may choose this course or choose to combine both approaches. Where such limits are to be set, the following clauses D.7 to D.12 apply.

D.5 Limitations

D.5.1 Limitations may need to be applied to vertical or horizontal deflections or deviations, to inclinations, to curvatures, to the widths of cracks or to the effects of vibrations.

D.5.2 The limitation of beam or slab deformations may be basically a matter of deflection, rotation or curvature. However, these requirements are specified throughout SANS 10160 in terms of deflection, or of deflection in relation to span, since this is the most easily observable parameter.

NOTE For simply supported spans under uniformly distributed loading, the slope at the ends may be taken as equal to 3 times the ratio of medial deflection to span, and the radius of curvature at the middle as equal to the span divided by 10 times the deflection/span ratio.

D.6 Strength and stability

D.6.1 General

Deformations affecting the strength and stability of a building or of its parts are taken into account in the process of structural design for the ultimate limit state. It is, however, necessary that the designers be aware of certain cases involving static or dynamic instability where the conditions existing during normal use of the building may have a considerable effect on the ultimate limit state.

D.6.2 Eccentric loading of walls and columns

D.6.2.1 Eccentric loading of walls and columns may occur as a result of excessive constructional deviation through inclination of these members or through deflections of floors or roof members. In both cases, the effects may be progressive and lead to collapse.

D.6.2.2 Inclination of vertical members may be due to constructional deviations or to the effects of wind load, or of permanent and imposed loads acting eccentrically or causing differential settlement. The presence of properly designed stiffening elements such as shear walls, central service cores, enclosed liftwells or staircases will usually improve stability.

D.6.2.3 Change of slope of floors or roof members at junctions with supporting walls or columns, taking place after construction, may produce loading of the latter that is both eccentric and inclined. Such changes of slope may be due to the effects of permanent and imposed loads on the floors or roof members, the permanent load causing creep deflection and the imposed loads causing elastic deflection and possibly creep deflection.

D.6.3 Resonance

Near-coincidence of forcing and natural vibrations may produce deflection due to resonance, which is a special case of deflection, of any building element. The degree of resonance may be reduced by appropriate adjustment of either of the two frequencies, or by the provision of vibration insulation or adequate damping. The problem arises mainly where the disturbing force is of large magnitude, i.e. in auditoria, dance halls, stands in sport stadia and in buildings having longspan suspended floors with a natural frequency of about 1-5 Hz, or containing machines with large unbalanced forces.

D.7 Deformations affecting serviceability by causing damage to buildings

D.7.1 General

Deformations, although possibly not affecting the strength or stability of a building, may cause damage to members (load-bearing or otherwise) and to finishes and claddings. They may produce unpleasant psychological effects, even to the extent of causing alarm. Finally, they may be physically such as to effectively prevent the use of the building for its intended purpose or to impair the health of the occupants. Some deformations may produce more than one kind of effect.

D.7.2 Cracking and spalling of walls

D.7.2.1 Changes of slope of floors and roofs at junctions with supporting walls or columns and lifting of the insufficiently restrained corners of torsional stiff floor slabs may cause horizontal cracking (particularly undesirable where floors are carried through to the face of the external wall) and also spalling of internal or external finishes. The actions involved are permanent load causing creep deflection and the imposed floor load causing elastic deflection and creep deflection.

D.7.2.2 Differential settlement and wind forces may also cause such cracking and spalling. Thermal and moisture movements in finishes are also involved. A more severe limitation may be necessary if deep edge stiffening beams are incorporated into the walls.

D.7.3 Cracking and spalling of ceilings

Curvature of the floor or roof may cause cracking in decoration on the underside of concrete slabs. Curvature subsequent to plastering may cause cracking of the plaster in the span and spalling in regions of negative curvature. The actions involved are the permanent load of the floors or roofs causing creep deflection and the imposed load causing elastic deflection and possibly creep deflection. Repeated thermal and moisture movements in the plaster may also be involved. Good extensibility of the plaster and good distribution of concentrated loads are ameliorating factors as is also the fact that cracks may be covered by redecoration. The permissible degree of cracking is largely subjective but depends on the use of the building.

D.7.4 Cracking and spalling of brittle partitions and non-load-bearing walls

D.7.4.1 Apart from cracking, spalling and local bulging due to thermal and moisture movements in the partitions themselves or in the supporting structure, damage to brittle partitions may arise as a result of the differential settlement of foundations, deflections of floors or roofs, or lateral movements of the building.

D.7.4.2 Estimation of this damage depends on a determination of the total tensile or compressive effects arising from all causes, together with information about the limiting tensile and compressive properties of the partitions, the effects on the number and width of cracks of any restraints to movement, and the degree of cracking that can be tolerated for the given type of surface finish and the given use of the building. Such a procedure is not yet sufficiently developed and it is meanwhile recommended that the deformation arising from various causes be dealt with separately. The suggested limiting values may permit a certain amount of cracking. Where this cannot be accepted, a more severe limitation, or more tolerant partitions, may be called for.

D.7.4.3 Differential settlement of foundations subsequent to the erection of partitions may produce diagonal cracking across the body of the latter. The actions involved are the self-weight load, including that of the partitions, and all long-term variable actions capable of influencing settlement.

D.7.4.4 Deflections of floors or roofs may damage partitions in a number of ways. In all cases the effects involved are those occurring after the erection of partitions, i.e. the self-weight load of the floor or roof, and in some cases that of the partitions, together with any pre-stress, causing creep deflections; the imposed floor or roof load (including any self-weight loads such as screeds and floor finishes applied after erection of partitions) causes elastic deflection and creep deflection; also curvature and other movements of the floor due to possible unrestrained moisture movements. In general, the greater the rigidity of the floor transverse to the span, the worse are the effects of its deformations. Where the partition has a high compressive strength and limit of deformability; where the ratio of length to height is less than approximately 3,5; where the partition is longitudinally restrained by the structure or by adjacent walls or partitions; and where there are few openings or continuous vertical sliding joints to interfere with the arching, the floor below the partition deflects more than the partition, a horizontal crack may be formed along the base of the partition, or a horizontal or arc-shaped crack may be formed in the lower portion of the partition, together with diagonal cracks across the upper corners due to extension of the under-surface of the floor above. If the floor or roof above the partition deflects more than the partition and there is no compressible packing at the head of the partition, the latter tends to be crushed and there may be vertical cracks in the lower portion and diagonal cracks across the upper corners. When the partition is loaded by the upper floor and carries these loads by strut-action to the ends of the span of the lower floor diagonal cracks radiating from the corners of the openings may also be produced.

D.7.4.5 Lateral deflection of a building as a result of wind forces may cause diagonal cracking across a partition. The action involved is that of the wind gust in having duration of sufficient length to produce the necessary deflection. Low-cycle fatigue damage may occur. Strong shear walls, central core zones or enclosed staircases have an ameliorating effect.

D.7.5 Damage to roof coverings, cladding and glazing

D.7.5.1 Deflections of roofs may cause damage to roof coverings of felt or metal, to roof sheeting or to roof glazing or tiling and may produce ponding of rainwater. The actions involved are permanent load producing creep deflections, any imposed loads and wind gusts of appropriate duration producing elastic deflections.

D.7.5.2 The cladding fixing should be designed so that structural loads are not transferred to cladding panels when the structural frame deforms.

D.7.5.3 The limitations of deflection may need to be more restrictive for roofs covered with sheet materials which become brittle with age.

D.8 Deformations affecting appearance

D.8.1 Visible sag of floors and ceilings

D.8.1.1 Visible deviations of floors and ceilings from the straight line or plane (unless obviously intentional) cause subjective feelings that are unpleasant and possibly alarming. The actions involved are the permanent load and the imposed loads, producing elastic deflections and possibly creep deflections, and also constructional deviations and thermal and moisture movements and, in the case of cantilevers, differential settlement. The provision of a camber or of a false ceiling can improve matters.

D.8.1.2 Subjective appraisal depends on the type of roof or floor (whether flat soffit, beam and slab, trough or ribbed construction), the area of it that is visible, its height and its relationship to other elements of the construction (particularly elements that are horizontal or in a horizontal plane), and the lighting conditions.

D.8.2 Visible lean of walls and columns

Visible deviation of vertical members from the vertical (unless obviously intentional) is also a source of subjective unrest. The actions involved are the self-weight and imposed loads causing differential settlements, but constructional deviations and the overturning effects of eccentric and inclined loads on walls and columns may be contributing factors. Persons vary in their appraisal of lean but are often guided by neighbouring vertical elements.

D.9 Deformations affecting use

D.9.1 Curvature of floors

Curvature of floors and the inclinations that it produces may cause people to stumble or slip, trolleys to move, furniture and equipment to tilt or rock, and spilt liquids to spread. Curvature may be due to constructional deviations and to elastic deflections and creep deflections (possibly upward) under permanent load alone or under permanent load and imposed floor loads, or to thermal or moisture movements. The provision of screeds or a camber may be appropriate.

D.9.2 Non-horizontality of floor supports

Unintentional lack of horizontality of floor supports causes many of the effects referred to in D.9.1. It may be due to constructional deviations or to differential settlement under self-weight and imposed floor loads (rotation of the point of support in the case of cantilevers).

D.9.3 Oscillations generated within the building or by wind forces

Apart from man-made external sources of vibration, such as nearby industrial and transport activities, whose effects are not covered, the main sources of oscillations in buildings are foot traffic and machinery within the building, together with wind gusts. Such oscillations may cause unpleasant sensations, including alarm, or prevent the carrying on of required activities, dependent on human sensitivity, on the activity to be pursued, on the degree of damping present, on the duration of the impulses and the interval between them. Recommendations for the limitation of oscillations of frequency exceeding 1 Hz are given in ISO 2631.

D.9.4 Deformations affecting special requirements in use

D.9.4.1 General

In certain types of buildings there may be special requirements in connection with, for example, particular activities of occupants or the use of machinery or precision apparatus. Examples of such requirements are as follows:

D.9.4.2 Deflections of overhead crane runway girders

D.9.4.2.1 Travelling cranes produce:

(a) vertical deflections of the runway girders (and of supporting brackets in some cases) due to their self-weight and that of the load carried;

(b) horizontal lateral and longitudinal deflections of the supporting columns due to the forces of acceleration and braking. (It is assumed here that the effects of constructional deviations and any subsequent movements of supports have been negated by the levelling and lining up the crane rails. Any upward deflection due to pre-stress may be taken into account.)

D.9.4.2.2 In the case of vertical deflections of the runway girders, there may be a problem of clearances. The principal problems, however, are the overloading of the means of propulsion due to the slope of the runway girders when under load and the maintenance of steady motion over the point of support.

D.9.4.2.3 In the case of horizontal deflections of the columns, it is necessary to limit the transverse deflection to prevent the crane gantry itself rotating excessively about the vertical (slewing) or becoming dislodged, and also to limit both transverse and longitudinal deflections to prevent excessive deformations of the supporting columns leading to damage to cladding and fixings (or to instability; see D.6).

D.9.4.3 Other special requirements

These requirements should be agreed in advance of design and construction with the client and the suppliers of any equipment involved. Examples of problems that may arise are:

a) vibration of weighing and measuring apparatus;

b) damage to impermeable membranes used for isolation or protection from liquids and gases;

c) twist of floors carrying machines operating on sheet materials;

d) inclinations affecting co-linearity of apparatus or levels of liquids;

e) interference with fine manual movements.

D.10 Deformations requiring general overall control

D.10.1 Cracking

D.10.1.1 Cracks in structural building elements may damage coverings, permit corrosion of reinforcing elements or allow penetration of liquids, gases or radiation (thus, for example, reducing thermal or airborne sound insulation, or admitting rain, dust or light). Cracks may also constitute disfigurement or cause alarm. (They are unlikely to cause structural collapse unless extremely wide and extensive, but they are early evidence of excessive action.)

D.10.1.2 In many cases, cracks may be avoided by appropriate initial design and construction measures. In other cases, the requirements of standards for other types of deformation may prevent the formation of cracks.

D.10.1.3 However, bearing in mind that design and construction measures may be only partially successful in controlling cracking, and that, in any event, cracks may occur in circumstances other than those provided for in standards; it is necessary to impose a general overall limitation on the width of cracks.

D.10.1.4 In laying down limitations, consideration must be given to the building materials involved, whether the cracks are through-cracks or surface cracks, whether they are likely to open further or close, whether they are repairable or capable of being covered by decoration, whether penetration of liquids, etc., is a factor, and the probable attitude of persons affected, in view of the intended use of the building.

D.10.1.5 In the case of possible corrosion of reinforcement, the permissible width of cracks should be laid down in design standards for the respective materials. Where corrosion of reinforcement is not in question it is suggested:

a) that through-cracks should not be permitted at positions where the transfer of water (e.g. by gravity, wind pressure or capillary actions) to the inside surfaces of rooms could occur;

b) that cracks should individually not exceed an average width of 0,2 mm if it is intended that they be coverable by redecoration;

c) that, if likely to be permanent, neither through-cracks nor surface cracks should individually exceed an average width of 2 mm, or such lower figure as may be required in particular circumstances (for example, in the presence of corrosive or humid atmospheres).

D.10.1.6 The widths of cracks and any resulting out-of-plane dislocations may be controlled by pre-stressed (or other) reinforcement.

D.10.2 Deformations due to earthquakes

Apart from the pounding of adjacent buildings due to insufficient clearance as referred to in D.12(d), oscillations during an earthquake may cause considerable damage. Methods of predicting and assessing the damage are still the subject of disagreement between experts, and research continues. It is therefore not possible at present to make any specific recommendation regarding limitation of deformation during an earthquake.

D.11 Methods of assessing probable deformations

D.11.1 The method used to assess or control the probable deformation is a matter for the structural designer. For example, he may determine deformations by calculation or by model or prototype testing; he may control them by the adoption of limiting span/depth ratios or other measures. The method used should be such that it gives an acceptable probability of meeting the requirements given. A reliability limit of $\beta_{t,serviceability} \approx 2$ is suggested as a desirable minimum for irreversible serviceability effects and $\beta_{t,serviceability} \approx 1$ for reversible effects.

D.11.2 When deformations are determined by calculation, such calculation should be based on actions as specified in SANS 10160-1.

D.11.3 The calculations should take into account constructional deviations, thermal movements, moisture movements, cracking of reinforced materials, and creep of materials under permanent and long-term temporary loads. In addition, the assistance received from various sources (for example, partial fixity at ends of beams and slabs, partial support from partitions), that cannot be sufficiently relied upon when strength properties are assessed, may be taken into account.

D.11.4 In calculating any required camber, it is suggested that the pre-camber is used to compensate for the deflection caused by the permanent loads. The designer should consider, if the quasi-permanent component of the variable actions should also be taken into account. The pre-camber should however be limited by considering its effects on the function and appearance of the building, in the absence of variable actions and creep which would only manifest with time. Limiting values for pre-camber should correspond to the values applied to conform to serviceability criteria.

D.11.5 The deformation limitation to be met should be the most severe of any values suggested for any particular criterion.

D.12 Common causes of deflection and deformation

The following is a summary of the more common actions that are responsible for deflection and deformation in buildings:

a) damage by mining subsidence or by movements of moisture-reactive soils (where movements are usually so great that special constructional measures are required)

b) relative movement between adjacent buildings, or at the point of entry or exit of services, due to differential settlement;

c) differential settlement causing nipping of walls, partitions and services on a ground-bearing floor slab;

d) pounding of inadequately spaced buildings during an earthquake;

e) ponding of water on roofs;

f) vibrations of cladding and oscillations produced by wind;

g) differential settlement causing nipping of windows and doors and jamming or demounting of sliding doors;

h) thermal expansion, particularly of roofs and exposed columns, and differential thermal expansion of different building materials or of thin exposed members such as cladding;

i) differential shrinkage of different building materials or of different qualities of the same material, possibly at different stages in their moisture movement;

j) long-term expansion of clay products, particularly in parapets, fascias, and floor coverings;

k) chemical deterioration, e.g. formation of sulpho-aluminates or of rust or other corrosion products and

l) upward creep deflection of unrestrained pre-stressed roof members.

Annex E
(Informative)

Guidance for design assisted by testing

E.1 Scope and field of application

E.1.1 Guidance is provided on the testing of structures, models of structures, parts of structures and structural materials for application in design according to clause 9 and

E.1.2 It is not intended to replace acceptance rules given in product specifications or execution standards.

E.2 Types of tests

E.2.1 A distinction needs to be made between the following types of tests :

a) tests to establish directly the ultimate resistance or serviceability properties of structures or structural members for given loading conditions. Such tests can be performed, for example, for fatigue loads or impact loads;

b) tests to obtain specific material properties using specified testing procedures, for instance, ground testing *in situ* or in the laboratory, or testing of new materials;

c) tests to reduce uncertainties in parameters of load or load effect models, for instance, by wind tunnel testing;

d) tests to reduce uncertainties in parameters used in resistance models, for instance, by testing structural members or assemblies of structural members (for example roof or floor structures);

e) control tests to check the identity or quality of delivered products or the consistency of production characteristics, for instance, testing of cables for structural use, or concrete cube testing;

f) tests carried out during execution in order to obtain information needed for part of the execution, for instance, testing of pile resistance, testing of cable forces during execution;

g) control tests to check the behaviour of an actual structure or structural members after completion, for example to determine elastic deflections, vibrational frequencies or damping;

E.2.2 For test types (a), (b), (c) and (d), the design values should, where ever practicable, be derived from the test results by applying accepted statistical techniques, see E.4 and E.5.

NOTE Special techniques might be needed in order to evaluate type (3) test results.

E.2.3 Test types (e), (f) and (g) may be considered as acceptance tests where no test results are available at the time of design. Design values should be conservative estimates which are expected to be able to meet the acceptance criteria at a later stage.

E.3 Planning of tests

E.3.1 Prior to carrying out of tests, a test plan should be agreed with the relevant authority. This plan should contain the objectives of the test and all specifications necessary for the selection or production of the test specimens, the execution of the tests and the evaluation of the test results.

Adequate safety precautions should be taken to prevent injury to persons and avoid damage to property during the test, especially with regard to the possibility of a collapse of the element under test.

The test plan should cover :

a) **objectives and scope** that should be clearly stated, for example the required properties, the influence of certain parameters varied during the test and the range of validity. Limitations of the test and required conversions (for example scaling effects) should be specified.

The criteria against which the results of the test will be judged and therefore the acceptability of the structure will be evaluated, should be specified.

b) **prediction of test results** where all properties and circumstances that can influence the prediction of test results, are taken into account, including:

1) geometrical parameters and their variability;

2) geometrical imperfections;

3) material properties;

4) parameters influenced by fabrication and execution procedures; and

5) scale effects of environmental conditions taking account of any sequencing, if relevant.

The expected modes of failure and/or calculation models, together with the corresponding variables, should be described. If there is significant doubt about which failure modes might be critical, then the test plan should be developed on the basis of pilot tests.

NOTE Attention needs to be given to the fact that a structural member can possess a number of fundamentally different failure modes.

c) **specification of test specimens and sampling** where the test specimens are specified, or obtained by sampling, in such a way as to represent the conditions of the real structure.

Factors to be taken into account include:

1) dimensions and tolerances;

2) material and fabrication of prototypes;

3) number of test specimens;

4) sampling procedure; and

5) restraints, if any

The objective of the sampling procedure should be to obtain a statistically representative sample.

Attention should be drawn to any difference between the test specimens and the product population that could influence the test results.

d) **loading specifications environmental conditions** are specified for the test and include:

1) loading points

2) loading history

3) restraints

4) temperatures

5) relative humidity

6) loading by deformation or force control.

Load sequencing should be selected to represent the anticipated use of the structural member, under both normal and severe conditions of use. Interactions between the structural response and the apparatus used to apply the load should be taken into account where relevant.
Where structural behaviour depends upon the effects of one or more actions that will not be varied systematically, then those effects should be specified by their representative values.

e) **testing arrangements** where the test equipment are relevant for the type of tests and the expected range of measurements. Special attention should be given to measures aimed at obtaining sufficient strength and stiffness of the loading and supporting rigs, and clearance for deflections.

f) **measurements** where all relevant properties for each individual test specimen are measured and listed prior to testing. Additionally a list should be made of the measurement locations. The procedures for recording results should be specified, including if relevant :

1) time histories of deflections;

2) velocities;

3) accelerations;

4) strains;

5) forces and pressures;

6) required frequency;

7) accuracy of measurements;

8) appropriate measuring devices;

9) date and time; and

10) temperature and humidity.

g) **evaluation and reporting** where specific guidance is given in E.4 and E.5. Any standards on which the tests are based, should be reported.

E.4 Derivation of design values

E.4.1 The derivation of the design values for a material property, a model parameter or a resistance should be carried out in one of the following ways:

a) by assessing a characteristic value, which is then divided by a partial factor and possibly multiplied, if necessary, by an explicit conversion factor and

b) by direct determination of the design value, implicitly or explicitly accounting for the conversion of results and the total reliability required

NOTE : In general method (1) is to be preferred provided the value of the partial factor is determined from the normal design procedure (see (c) below).

E.4.2 The derivation of a characteristic value from tests (method (1)) should take into account:

a) the scatter of test data;

b) statistical uncertainty associated with the number of tests; and

c) any available prior statistical knowledge.

E.4.3 The partial factor to be applied to a characteristic value should be taken from the appropriate standard provided there is sufficient similarity between the tests and the usual field of application of the partial factor used in numerical verifications.

E.4.4 If the response of the structure or structural member or the resistance of the material depends on influences not sufficiently covered by the tests such as:

a) time and duration effects;

b) scale and size effects;

c) different environmental, loading and boundary conditions; and

d) resistance effects.

then the calculation model should take such influences into account as appropriate.

E.4.5 In special cases where the method given in E.4(a)(2) is used, the following should be taken into account when determining design values :

a) the relevant limit states;

b) the required level of reliability;

c) compatibility with assumptions relevant to the achievement of the required level of reliability for actions;

d) where appropriate, the required design working life; and

e) prior knowledge obtained from similar cases.

E.5 Principles of statistical evaluations

E.5.1 When evaluating test results, the behaviour of test specimens and failure modes should be compared with theoretical predictions. When significant deviations from predictions occur, an explanation should be sought; this might involve additional testing, perhaps under different conditions, or modifications of the theoretical model.

E.5.2 The evaluation of test results should be based on statistical methods, with the use of available statistical information about the type of distribution to be used and its associated parameters. The methods given may be used only when the following conditions are satisfied:

a) the statistical data (including prior information) are taken from identified populations which are sufficiently homogeneous

b) a sufficient number of observations is available.

NOTE At the level of interpretation of test results, three main categories can be distinguished

- where one test only (or very few tests) is (are) performed, no classical statistical interpretation is possible. Only the use of extensive prior information associated with hypotheses about the relative degrees of importance of this information and of the test results, make it possible to present an interpretation as statistical (Bayesian procedure, see ISO 12491)

- if a larger series of tests is performed to evaluate a parameter, a classical statistical interpretation might be possible. This interpretation will still need to use some prior information about the parameter

- when a series of tests is carried out in order to calibrate a model (as a function) and one or more associated parameters, a classical statistical interpretation is possible.

E.5.4 The result of a test evaluation should be considered valid only for the specifications and load characteristics considered in the tests. If the results are to be extrapolated to cover other design parameters and loading, additional information from previous tests or from theoretical bases should be used.

Annex F
(informative)

Principles of structural performance

F.1 General

The basis of design for structural performance is to establish the ability of structures to sustain actions and maintain their integrity and robustness. SANS 10160 provide the principles and design rules together with the actions that need to be taken into account in the design of an outlined scope of structures, consisting of buildings and similar industrial structures. The basis of design for structural performance also applies to the design rules for structural resistance as provided in the materials-based structural design standards which refer to SANS 10160.

F.2 Compliance with basic principles

The basic principle for structural performance is that the structure will, during its intended life, with appropriate degrees of reliability and in an economic way, sustain all actions and influences likely to occur during execution and use and to remain fit for the use for which it is intended.

The application of the normative stipulations of this standard and the related materials-based design standards is deemed to achieve compliance with the basic principles due to the technological and experience base of the standard. The requirements are based on experience with satisfactory performance of structures designed according to the specified principles, rules and models for structural behaviour and reliability, allowing for local conditions and practice, but referring also to international practice and improving harmonisation with such practice.

F.3 Provision for abnormal events

In addition to compliance with the basic principles, the structure is also required to have integrity and robustness against the effects of abnormal events resulting in exceptional conditions and actions on it. Abnormal events, which are not considered likely to occur during the design life of the structure, may be identifiable, or may result from conditions that can not be clearly identified in advance. Abnormal events and conditions include earthquakes, fire, impact from vehicles or falling and swinging objects; explosions due to gas, ignition of industrial liquids or boiler failure; adjacent excavation or flooding causing severe local foundation failure; very high winds such as cyclones or tornadoes; and the consequences of human error.

The principle of design for integrity and robustness is that damage to the structure is accepted, but that it will not be damaged to an extent disproportionate to the original cause of the abnormal unidentified or identified events. In the case of fire, the requirements for structural integrity and robustness are related to the time needed for emergency measures. Although the effects of war and terrorism fall outside the scope of the Standard, provision for the integrity and robustness of the structure will reduce its vulnerability to such activities.

A sufficiently low probability of occurrence of abnormal events forms the fundamental consideration for the treatment of these events. The risk resulting from such low probability events with large consequences should be tolerably small, particularly in comparison to the acceptable risk as implied by the basic requirements for buildings. A minimum degree of robustness is required in terms of the provision for unidentified abnormal events, situations and actions.

Certain situations and actions are identified and classified as accidental in the normative stipulations for considering such actions. Examples are the provisions for seismic actions, provisions for fire, and specific situations for imposed loads, wind actions and crane induced actions. These requirements are covered by normative stipulations for actions on structures as set out in the respective Parts of SANS 10160.

For specific projects, additional abnormal situations and actions may be identified for treatment as accidental situations and related actions. Such incorporation of identified accidental actions for specific projects is made in agreement with the owner and relevant authorities. Such a decision may be based on risk assessment of the system related to the structure. Risk levels should however be acceptable in comparison with the risk implied by the reliability levels applied in this standard.

F.4 Relation of SANS 10160 to materials-based design standards

The actions stipulated in SANS 10160 and the ability of structures to sustain such actions, as stipulated in the materials-based design standards, are clearly related. The influence of the revision of SANS 10160 on the existing materials-based standards therefore requires consideration.

The principle has been maintained that acceptable performance of structures designed according to existing procedures provides confirmation of sufficient levels of reliability. This provides the basis for the continued use of existing structural materials-based design standards together with SANS 10160. The full potential of the extended reliability framework provided in SANS 10160, in the design of more efficient or advanced structures and utilising modern structural materials, will be realised when the materials-based design standards are also revised accordingly, or when new standards are introduced.