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SANS 10160-5: BASIS OF STRUCTURAL DESIGN AND ACTIONS FOR BUILDINGS AND INDUSTRIAL STRUCTURES — PART 5: BASIS OF GEOTECHNICAL DESIGN AND ACTIONS

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

SABS Standards Division
Attention: Compliance and Development department
Private Bag X191
Pretoria
0001

Fax No.: (012) 344-1568 (for attention: dsscomments)
E-mail: dsscomments@sabs.co.za

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- The public enquiry stage will be repeated if the technical committee agrees to significant technical changes to the document as a result of public comment. Less urgent technical comments will be considered at the time of the next amendment.

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Edition 1

SOUTH AFRICAN NATIONAL STANDARD

Basis of structural design and actions for buildings and industrial structures

Part 5: Basis of geotechnical design and actions

WARNING — This document may only be used in conjunction with SANS 10160-1.

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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.

The SANS 10160 Series consisting of SANS 10160-1 to SANS 10160-8 supersedes SABS 0160:1989 (edition 2).

This document was published in xxxx 2009.

The SANS 10160 series 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.

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Annex B (Informative) Guidance on partial material and resistance factors

Annex C (Informative) Basic guidance on design of foundations and evaluation of earth pressures

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Basis of structural design and actions for buildings and industrial structures

Part 5: Basis of geotechnical design and actions

1 Scope

1.1 This part of the SANS 10160 Series falls within the general scope of application as given in SANS 10160-1 clause 1.1.

1.2 The requirements specified in SANS 10160-5 shall be applied in conjunction with the requirements specified in the following parts of the SANS 10160 Series :

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-6, *Actions induced by cranes and machinery*

Part-7, *Thermal actions*

Part-8, *Actions during execution*

1.3 This standard sets out the basis for geotechnical design and gives guidance on the determination of geotechnical actions on buildings and industrial structures including:

- a) vertical earth loading;
- b) earth pressure;
- c) ground water and free water pressure and
- d) actions caused by ground movement.

1.4 Procedures are given for determining representative values for geotechnical actions.

1.5 This standard does not cover the design of geotechnical structures such as slopes, embankments or free-standing retaining structures

2 Normative references

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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.

EN 1990 Eurocode - *Basis of structural design*

EN 1997-1 Eurocode 7 - *Geotechnical design – Part 1: General rules*

NA to BS EN 1997-1 *UK National annex to Eurocode 7: Geotechnical design – Part 1: General rules.*

SANS 10160-1 *Basis of structural design and actions for buildings and industrial structures Part 1: Basis of structural design*

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

SANS 10160-3, *Basis of structural design and actions for buildings and industrial structures Part 2: Wind actions*

SANS 10160-4 *Basis of structural design and actions for buildings and industrial structures Part 4: Seismic actions and general requirements for buildings*

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

SANS 10160-7 *Basis of structural design and actions for buildings and industrial structures Part 7: Thermal actions*

SANS 10160-8 *Basis of structural design and actions for buildings and industrial structures Part 8: Actions during execution*

3 Definitions and symbols

For the purpose of this part of the SANS 10160 the definitions and symbols given in SANS 10160-1 and the following apply.

3.1 Definitions

3.2.1

accidental action

A

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

3.2.2

accompanying value of a variable action ($\psi \times Q_k$)

value of a variable action that accompanies the leading action in a combination

3.2.3

action

F

- a) set of forces (loads) applied to the structure (direct actions);
- b) set of imposed deformations or accelerations caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect actions).

3.2.4

active earth pressure

minimum value of earth pressure exerted on a structure where the movement of the structure away from the retained earth is sufficient to fully mobilise the shear strength of the retained earth

3.2.5

bulk weight density

density of ground including the weight of moisture in the voids and pores

3.2.6

characteristic value

value of an action or material property having a prescribed probability of not being attained.

NOTE 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

3.2.7

design value

value obtained by multiplying or dividing the representative value of an action or material property by a partial factor

3.2.8

earth

ground

all earth materials (soil, gravel, rock, etc.) including fill or natural ground

3.2.9

earth pressure

lateral pressure exerted by earth and ground water on the structure

3.2.10

earth pressure at rest

pressure exerted by earth on an un-yielding structure where there is no movement of the structure relative to the ground

3.2.11

effect of action

E

effect of actions (or action effect) on structural members, (for example internal force, moment, stress, strain) or on the whole structure (for example deflection, rotation)

3.2.12

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favourable actions

actions that have a stabilizing effect

3.2.13

fill

imported soil or rock material placed on site with or without compaction

3.2.14

fixed action

action that has a fixed distribution and position

3.2.15

free action

action that may have various spatial distributions over the structure

3.2.16

geotechnical action

action exerted on the structure by the ground or ground water

3.2.17

irreversible serviceability limit states

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

3.2.18

load case

compatible arrangement of loads, sets of deformations and imperfections considered simultaneously with fixed variable actions and permanent actions for a particular verification

3.2.19

limit states

states beyond which the structure no longer fulfils the relevant design criteria

3.2.20

natural ground

in situ soil or rock in its natural undisturbed state

3.2.21

nominal value

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

3.2.22

passive earth pressure

maximum value of earth pressure exerted on a structure where the movement of the structure towards the retained earth is sufficient to fully mobilise the strength of the retained earth

3.2.23

permanent action

G

action for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until a certain limit value is attained

3.2.24

representative value of an action

F_{REP}

value used for the verification of a limit state. A representative value may be the characteristic value or, in the case of a variable actions, an accompanying value

3.2.25

reversible serviceability limit states

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

3.2.26

serviceability limit states

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

3.2.27

ultimate limit states

states associated with collapse or with other similar forms of structural failure

3.2.28

unfavourable actions

actions that have a destabilizing effect

3.2.29

variable action

Q

action for which the variation in magnitude with time is neither negligible nor monotonic

3.2 Symbols

A_d is the design value of an accidental action

a_d is the design value of geometric data

a_{nom} is the nominal value of geometric data

c is the cohesion intercept

c' is the cohesion intercept in terms of effective stress

c_u is the un-drained shear strength

$c_{u,d}$ is the design value of un-drained shear strength

$c_{u,k}$ is the characteristic value of un-drained shear strength

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E_d	is the design value of the effect of actions
G_k	is the characteristic value of a permanent action
$G_{k,j}$	is the characteristic value of permanent action j
q_u	is the unconfined compressive strength
Q_k	is the characteristic value of a variable action
$Q_{k,1}$	is the characteristic value of the leading variable action
$Q_{k,j}$	is the characteristic value of accompanying variable action j
$R_{b,cal}$	is the pile base resistance calculated from ground test results at the ultimate limit state
$R_{b,d}$	is the design value of the base resistance of a pile
R_c	is the compressive resistance the ground around a pile at the ultimate limit state
$R_{c,cal}$	is the R_c calculated from ground test results
$R_{c,d}$	is the design value of R_c
R_d	is the design value of resistance to an action
R_k	is the characteristic value of resistance to an action
$R_{s,cal}$	is the pile shaft resistance calculated from ground test results at the ultimate limit state
$R_{s,d}$	is the design value of the shaft resistance of a pile
W_k	is the characteristic value of a variable action due to wind
X_d	is the design value of a material property
X_k	is the characteristic value of a material property
ϕ_a	is the change made to nominal geometrical data for particular design purposes
ϕ	is the angle of shearing resistance
ϕ'	is the angle of shearing resistance in terms of effective stress

ϕ'_d	is the design value of ϕ'
ϕ'_k	is the characteristic value of ϕ'
γ	is the bulk weight density of ground
γ'	is the submerged weight density of ground
γ_a	is the partial factor for anchorages
γ_b	is the partial factor for pile base resistance
γ_c	is the partial factor for effective cohesion
γ_{cu}	is the partial factor for un-drained shear strength
γ_g	is the partial factor for permanent actions
γ_G	is the partial factor for permanent actions also accounting for model uncertainties
$\gamma_{G,j}$	is the γ_G for permanent action j
γ_m	is the partial factor for a material property
γ_Q	is the partial factor for variable actions
γ_{qu}	is the partial factor for unconfined strength
$\gamma_{Q,j}$	is the γ_Q for variable action j
$\gamma_{R,d}$	is the partial factor for uncertainty in a resistance model
γ_s	is the partial factor for pile shaft resistance
$\gamma_{s,t}$	is the partial factor for tension resistance of a pile
γ_t	is the partial factor for total resistance of a pile
γ_ϕ	is the partial factor for angle of shearing resistance (applied to $\tan \phi'$)
γ_γ	is the partial factor for weight density
ψ	is the combination factor for a variable action

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ψ_j is the combination factor for variable action j

ψ_{wind} is the combination factor for a variable action due to wind

4 Classification of geotechnical actions

4.1 Vertical earth loading

4.1.1 Vertical earth loading may either be permanent or variable.

4.1.2 Permanent vertical earth loading, for example fill placed above the structure or parts thereof, shall be classified as a permanent fixed action.

4.1.3 Temporary earth loading, for example a stockpiles of earth, shall be classified as a variable action or as a quasi-permanent action and may be a free or fixed action as appropriate.

4.2 Earth pressure

4.2.1 Earth pressures due to the weight of the retained ground and ground water shall be classified as permanent actions.

4.2.2 Earth pressures due to surcharges shall be classified as variable or permanent actions depending on the classification of the surcharge loading.

4.2.3 Passive earth pressures shall be classified as permanent actions. Where the effect of ground water or surcharge is beneficial, the partial load factor applicable to favourable actions shall be used.

4.3 Ground water and free water pressure

4.3.1 Ground water pressure may act on the top, bottom or sides of a structure.

4.3.2 Ground water pressure acting on top of a structure shall be taken into account in the determination of vertical earth loading and shall not be classified as a separate action except as noted in 4.3.5 and 4.3.6.

4.3.3 Ground water pressure on the sides of the structure shall be taken into account in the determination of earth pressures and shall not be regarded as a separate action except as noted in 4.3.5 and 4.3.6.

4.3.4 Ground water pressure below a structure due to a quasi-permanent water table within the surrounding ground shall be classified as a permanent action.

4.3.5 Free water above the ground surface, including the additional water pressure within the ground caused by the presence of such water, shall be classified as a variable action.

4.3.6 Additional water pressure, both within the ground and above the ground surface, arising from temporary flooding or other transient causes shall be classified as a variable or accidental action as appropriate.

4.4 Actions caused by ground movement

4.4.1 Actions caused by relative movement between the ground and the structure include those that give rise to additional loading on the structure and those that impose deformations on the structure. The additional loading may be unfavourable (for example downdrag forces on piles) or favourable (for example wall friction on a retaining wall).

4.4.2 Downdrag and uplift on piles or other buried structures due to settlement or heave respectively of surrounding ground shall be classified as permanent actions except where such heave is due to seasonal variations in moisture content.

4.4.3 All actions on piles or other buried structures due to seasonal changes in moisture content shall be classified as variable actions.

4.4.4 Irreversible imposed deformations caused by differential movement of the ground shall be classified as permanent actions.

4.4.5 Favourable actions due to ground movement should only be considered where there is reasonable certainty that the movement will occur and that the action will not be negated by the effects of moisture content changes, vibration or similar causes.

5 Geotechnical and geometrical data

5.1 Geotechnical investigations

5.1.1 Geotechnical investigations shall be carried out to provide sufficient data on the ground and ground water conditions at and around the site for a proper description of the nature of the soil profile, an assessment of its likely behaviour and the reliable evaluation of parameters to be used in design calculations.

5.1.2 The scope of the geotechnical investigation shall be commensurate with the Geotechnical Category of the project in accordance with annex A and the extent to which the properties of the ground will influence the performance of the structure.

5.1.3 In all cases except simple structures on relatively simple ground conditions where there is negligible risk in terms of overall stability or significant ground movements, the investigation shall produce quantitative data for design purposes.

5.1.4 The extent of the investigation shall be such that the properties of the ground lying outside the area of investigation will not exert a significant effect on the design or performance of the structure.

5.1.5 The investigation shall include a detailed description of the soil profile to enable visible deviations from the conditions assumed at design stage to be detected during construction. See annex A.

5.2 Ground properties

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5.2.1 Ground properties, as quantified for design purposes by geotechnical design parameters, shall be obtained from laboratory or field tests (either directly or through correlation), theory, empiricism and other relevant data.

5.2.2 Account shall be taken of the possible differences between the ground properties obtained from test results and those governing the behaviour of the soil or rock mass in situ.

5.2.3 The ground properties selected for design should take account of:

- a) the level of confidence in the test result and the method by which they were obtained;
- b) the stress level, strain level and mode of deformation of the ground;
- c) the effect of soil or rock fabric (joints, fissures, laminations, slickensides, large particles, etc.) on the behaviour of the soil or rock mass;
- d) the rate of loading and of pore water dissipation;
- e) the effect of field moisture content and changes in moisture content;
- f) the effect of construction activities on the properties of the ground.

5.3 Geotechnical parameters

5.3.1 Characteristic values

5.3.1.1 The characteristic value of a geotechnical parameter shall be selected on the basis of the results of laboratory tests or values derived from field tests, complemented by reliable experience.

5.3.1.2 The characteristic value of a geotechnical parameter shall be selected as a cautious estimate of the value affecting the occurrence of the limit state under consideration.

5.3.1.3 The selection of characteristic values shall take account of:

- a) geological and other background information including data from previous projects;
- b) the variability of the ground property considered;
- c) the extent of the field and laboratory investigation;
- d) the extent to which the occurrence of the limit state will be governed by local variations in the ground properties;
- e) the ability of the structure to redistribute loads to stronger or stiffer zones in the ground.

5.3.1.4 If the behaviour of the structure at the limit state under consideration is governed by the lowest or highest value of the ground property, the characteristic value should be a cautious estimate of the lowest or highest value occurring in the zone of ground governing the behaviour.

5.3.1.5 If the behaviour of the structure at the limit state concerned is governed by the average properties of the ground around or below the structure, the characteristic value should be a cautious

estimate of the mean value occurring in the zone of ground governing the occurrence of the limit state.

5.3.1.6 If statistical methods are used, the characteristic value so determined should be such that the probability of a worse value governing the occurrence of the limit state under consideration is not greater than 5 %.

NOTE In this respect, a cautious estimate of the mean value is the value selected from the limited data set to represent the mean value in the field with a confidence level of 95 %. Where local failure is concerned, a cautious estimate of the lowest or highest value is a 5 % fractile.

5.3.2 Design values

5.3.2.1 The design values of geotechnical parameters (X_d) shall either be derived from the characteristic value using the following equation:

$$X_d = X_k / \gamma_M \quad (1)$$

or shall be assessed directly.

5.3.2.2 The value of γ_M depends on the limit state under consideration (see SANS 10160-1).

5.4 Geometrical data

5.4.1 The ground level and slope, water levels, levels of interfaces between strata and excavation levels shall be treated as geometrical data in accordance with the requirements of SANS 10160-1.

5.4.2 Characteristic values of the level of ground, ground water or free water shall be measured, nominal, or estimated upper or lower levels.

5.4.3 In cases where variations in ground water or free water have a significant effect on the reliability of the structure, the characteristic value for the elevation of the free water surface or phreatic surface shall be taken as a cautious estimate of the most adverse level likely to occur during the design lifetime of the structure. This estimate shall consider the hydraulic and hydrogeological conditions on the site, the effect of variation in the permeability on the ground water regime, the presence of perched or artesian water tables, any physical controls on the level of the phreatic or free water surface and historical records.

5.4.4 Characteristic values of levels of and dimensions of geotechnical structures should usually be nominal values.

5.4.5 In cases where deviations in geometric data have a significant effect on the reliability of the structure, design values of geometrical data (a_d) should either be assessed directly or derived from nominal values using the following equation:

$$a_d = a_{\text{nom}} \pm \Delta a \quad (2)$$

where:

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a) in the case of single piles or two pile groups, the deviation of position of the pile head from the nominal position Δa should be taken as of 75 mm at piling platform level plus 1/60 of the depth of the pile cut-off below platform level for vertical piles and 1/30 for raking piles;

b) in the case of foundation loads with large eccentricities exceeding one third of the width of a rectangular foundation or 0,6 of the radius of a circular foundation, a tolerance Δa of 100mm should be considered.

6 Design verification

6.1 Ultimate limit states

6.1.1 When considering the ultimate limit state, it shall be verified that:

$$E_d < R_d \quad (3)$$

for each of the ultimate limit states described in SANS 10160-1.

6.1.2 The design value of the effect of actions E_d shall be determined in accordance with SANS 10160-1 using the relevant partial action factors and action combination factors.

6.1.3 The design value of the resistance to actions R_d may be determined by applying partial factors to ground properties (X) or resistances (R) or to both as follows:

$$R_d = R \left\{ \gamma_F \times F_{\text{rep}} ; X_k / \gamma_M ; a_d \right\} \quad (4a)$$

or

$$R_d = R \left\{ \gamma_F \times F_{\text{rep}} ; X_k ; a_d \right\} / \gamma_R \quad (4b)$$

or

$$R_d = R \left\{ \gamma_F \times F_{\text{rep}} ; X_k / \gamma_M ; a_d \right\} / \gamma_R \quad (4c)$$

6.1.4 The application of partial material factors in each of the ultimate limit states considered in SANS 10160-1 shall be in accordance with SANS 10160-1.

6.1.5 Partial resistance factors are applied in the design of axially loaded piles and anchors where the resistance of the pile or anchor is obtained by calculation using values of ground parameters.

6.1.6 Suggested values for partial material factors and partial resistance factors are given in annex C.

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6.1.7 More severe partial factor values than those referred to above should be used in cases of abnormal risk or unusual or exceptional ground or loading conditions (Geotechnical category 4 – see annex A).

6.1.8 Less severe partial factor values than those referred to above may be used for temporary structures or transient design conditions, on condition the likely consequences can be justified.

6.2 Verification of serviceability limit states

6.2.1 Serviceability limit states shall be verified in accordance with SANS 10160-1 or using the simplified approach given in 6.2.6.

6.2.2 When considering serviceability limit states that develop over a short period of time, the combination of actions applicable to irreversible serviceability as covered in SANS 10160-1 shall be used. This combination of actions would apply, for example, to settlement of granular soils where the bulk of the settlement occurs as immediate settlement.

6.2.3 When considering serviceability limit states that develop over a long period of time, the combination of actions applicable to long term serviceability as covered in SANS 10160-1 shall be used. This combination of actions would apply, for example, to the assessment of consolidation settlement or creep settlement which occur over a period of time.

6.2.4 The values of partial factors for soil parameters used in verification of the serviceability limit state shall be taken as equal to 1,0.

6.2.5 Characteristic values for ground properties should be selected appropriately where these can be affected by variations that may occur during the life of the structure due to factors such as ground water lowering, desiccation, saturation, etc.

6.2.6 Serviceability limit states may be verified by ensuring that a sufficiently low fraction of the strength of the ground is mobilised to keep deformations within the required serviceability limits, provided this simplified approach is only applied where:

- a) quantification of the deformation is not required to check the serviceability limit state;
- b) established comparable experience exists with similar ground, structures and application methods.

6.2.7 Calculations of total and differential settlement shall take account of:

- a) the rate of settlements and ground movements;
- b) random and systematic variations in ground properties;
- c) the distribution of loading;
- d) the construction method including the sequence and rate of loading, and
- e) the stiffness of the structure during and after construction.

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7 Determination of geotechnical actions

7.1 General

7.1.1 Consideration shall be given to the possibility of variable actions occurring both jointly and separately.

7.1.2 The duration of actions shall be considered with reference to the time effects in the material properties of the soil (for example permeability, consolidation, etc.).

7.1.3 Actions that produce a dynamic response in the ground shall be identified for appropriate analysis.

7.1.4 Where unfavourable and favourable permanent actions originate from a single source, a single partial action factor may be applied to the sum of these actions or to the sum of their effects.

7.2 Vertical earth loading

7.2.1 The determination of vertical earth loading shall be carried out using the bulk weight density of the earth and shall allow for possible changes in moisture content that may occur.

7.2.2 Allowance shall be made for variations in the thickness of the earth layer and its extent as defined in 5.4.5.

7.2.3 For non-uniform earth loading, for example conical stockpiles, allowance may be made for redistribution of loads due to arching.

7.2.4 Where fill material is placed on a surface of varying stiffness (for example over a buried culvert), allowance shall be made for attraction of load towards areas that will settle least, i.e. those providing stiffer support.

7.2.5 Any surcharge on the ground surface or free water above the surface shall be considered separately as an imposed load.

7.3 Earth pressure

7.3.1 In the context of this code, earth pressure includes the pressure exerted by the ground and any ground water that may be present, the combination of which shall be regarded as a single action.

7.3.2 The calculation of earth pressure shall make allowance for surcharges on the ground surface, the inclination of the retaining wall, the surface slope of the retained ground, water tables and seepage forces in the ground, the shear strength and weight density of the ground and the roughness of the wall.

7.3.3 The determination of earth pressure shall take account of the mode and magnitude of movement of the retaining structure that may occur at the limit state under consideration. The value of earth pressure at an ultimate limit state is generally different from its value at a serviceability limit state where deformations of the ground may be insufficient to mobilise the shearing resistance

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of the ground. Consequently, when expressed as an action, earth pressure cannot have a single characteristic value.

7.3.4 Where no movement of the supporting structure relative to the ground takes place, the earth pressure shall be calculated from the “at rest” state of stress. The determination of the at rest earth pressure shall take account of the stress history of the ground.

7.3.5 Limiting values of earth pressure in the active or passive states occur when there is sufficient movement of the retaining structure away from or into the retained ground respectively. Active and passive earth pressure conditions shall be considered only where movement of the structure is sufficient to fully mobilise the shear strength of the ground in the active and passive states, noting that significantly more movement is required to mobilize passive pressures than active pressures.

7.3.6 Intermediate values of earth pressure occur if the wall movements are insufficient to mobilise the limiting values. The determination of intermediate values of earth pressure shall take account of the amount of wall movement and its direction relative to the ground.

7.3.7 The distribution of earth pressures over the height of a retaining structure is dependent on the mode of relative movement (for example translation, rotation about top, rotation about toe, etc.). Account shall be taken of the distribution of earth pressure against the structure where this could affect the reliability of the structure.

7.3.8 The determination of earth pressures shall take account of the additional pressures generated by the methods used for the placing and compaction of backfill. Appropriate construction procedures shall be specified to ensure that the assumptions made in the determination of earth pressures with regard to placement and compaction procedures are not violated.

7.3.9 Water pressures within the retained ground shall be determined taking account of the requirements of 5.4.3.

7.3.10 For structures retaining earth of low or medium permeability (silts and clays), water pressures shall be assumed to act behind the wall. Unless a reliable drainage system is implemented or infiltration is effectively prevented, the values of water pressure shall correspond to a water table at the surface of the retained material.

7.3.11 The formation of tension or shrinkage cracks shall be considered in cohesive soils together with the possibility of such cracks becoming filled with water.

7.3.12 If the safety or serviceability of the structure depends on the successful performance of a drainage system or infiltration barrier, the consequences of failure of these measures shall be considered, having regard to safety and the cost of repair. One or both of the following conditions shall apply:

- a) a maintenance programme for the drainage system shall be specified and the design shall allow access for this purpose; or
- b) it shall be demonstrated both by comparable experience and by assessment of any water discharge, that the drainage system will operate adequately without maintenance.

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7.3.13 Account shall be taken of any swelling potential of the ground in the calculation of earth pressures.

7.3.14 The favourable effects of wall friction shall not be considered in the calculation of earth pressures where there is a likelihood that wall friction may not be mobilised or may be negated by factors such as vibration. The significant effect of wall friction on reducing active pressures and increasing passive resistance should be carefully evaluated and should be included only where such friction can be reliably mobilised.

7.4 Ground water and free water pressure

7.4.1 Ground water and free water pressure shall be considered in accordance with 4.3.

7.4.2 In determining the magnitude of ground water pressure, the requirements of 7.3.9 to 7.3.12 shall be considered.

7.5 Downdrag and uplift

7.5.1 Downdrag and uplift forces arise from the mobilisation of shear stresses between the structure and the surrounding ground due to relative displacements as a result of settlement or heave of the surrounding soil respectively.

7.5.2 Downdrag and uplift forces shall be calculated using the upper characteristic value for soil shear strength parameters.

7.5.3 In assessing the magnitude of the uplift or downdrag forces, account shall be taken of any changes in soil properties that could occur over time including strengthening, softening or swelling (causing an increase in lateral pressure) of the soils caused by changes in moisture content.

7.5.4 The shear stresses mobilised between the structure and the ground strata overlying the expansive or compressible strata also shall be taken into account.

7.6 Structural deformations caused by ground movement

7.6.1 Where total or differential movement of the ground or of foundations could affect the safety or serviceability of the structure or of service connections (water, sewerage, etc), the magnitude of settlement or heave (or both) shall be estimated. Induced deformations of the structure should be included as actions in the analysis of the structure.

7.6.2 The serviceability limit state design loads should be used when calculating the foundation displacements for comparison with serviceability criteria.

7.6.3 The effect of neighbouring foundations and fill should be taken into account when calculating the stress increase in the ground and its influence on ground movements.

7.6.4 Calculations of settlement shall include both intermediate and long term settlement, including the possible effects of future changes in moisture content.

7.6.5 Special consideration should be given to organic soils and soft clays in which significant creep movements may occur.

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7.6.6 Allowance should be made for differential settlement of the ground unless it is prevented by the stiffness of the structure.

7.6.7 Differential settlement calculations that ignore the stiffness of the structure and its ability to redistribute loads tend to produce over-predictions. In the case of minor structures, a simple, single iteration, re-evaluation of settlement may suffice. In the case of more complicated or sensitive structures, a detailed ground-structure interaction analysis may be justified.

7.6.8 Experience has shown that the accuracy of settlement predictions is poor, i.e. significant differences may occur between predicted and observed settlements. This should be taken into account in the design process.

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Annex A
(informative)

**Influence of geotechnical categories
on engineering requirements**

A.1 Scope and application

A.1.1 The minimum requirements for investigation, design, construction control and monitoring of geotechnical works will vary according to the nature of the ground conditions, the complexity of the structure, the nature of the loading and the associated risks.

A.1.2 This annex sets out the basic requirements for investigation, design, construction control and monitoring of geotechnical works and provides guidance on each of these activities.

A.1.3 The approach adopted in this Annex is to classify geotechnical works into four categories as outlined in A.2 and to specify minimum engineering requirements for each category.

NOTE The approach followed is similar to that adopted in EN 1997-1 except that a four-fold, rather than three-fold, classification system is used in which geotechnical works are subdivided according to the risks involved and the degree of specialisation required in the investigation, design and execution of the works.

A.1.4 A preliminary classification of the works should generally be made prior to the geotechnical investigations. The classification should be checked and changed, if necessary, at any stage in the investigation, design or construction process.

A.1.5 Various aspects of a project can be classified into different Geotechnical Categories. It is not necessary to treat the whole of the project according to the highest category.

A.2 Geotechnical categories

A.2.1 Geotechnical category 1 includes:

- a) works in ground conditions shown by comparable local experience to be relatively straightforward;
- b) situations where there are no excavations below the water table and no excavations that are not battered to angle of repose except where comparable local experience indicates that the proposed excavation will be stable;
- c) small and relatively simple structures with no exceptional loading conditions; and
- d) where there is negligible risk in terms of overall stability or significant ground movements.

A.2.2 Geotechnical category 2 includes:

- a) conventional types of structures and foundations;
- b) with no exceptional risks or loading conditions or difficult ground conditions; and

c) for which design methods are well established.

Examples of structures that lie in this category are spread footings, raft foundations, piled foundations, walls and other structures retaining soil or water, embankments and earthworks.

A.2.3 Geotechnical category 3 includes:

- a) conventional types of structures and foundations; with
- b) with no exceptional risk or loading conditions and
- c) for which the nature of the ground or design complexity requires specialist geotechnical input.

Examples of structures that may lie in this category include anchored retaining structures, tunnels, deep excavations below the water table, dynamically loaded foundations and structures involving soil-structure interaction analysis.

A.2.4 Geotechnical category 4 includes:

- a) structures or parts of structures that lie outside the limits of categories 1 to 3, for example very large or unusual structures;
- b) structures involving abnormal risks; or
- c) on in unusual, unstable or exceptionally difficult ground conditions.

A.2.5 The provisions covered in this standard apply directly to the design of structures in geotechnical categories 1 and 2. The standard may be used as guidance in the design of geotechnical category 3 or 4 structures but such designs are likely to require additional or alternative rules and provisions to those given in this standard.

A.3 Investigation, design, construction control and monitoring

A.3.1 Basic requirements

A.3.1.1 Table A.1 summarises the basic requirements for geotechnical investigation, design of the geotechnical aspects of the project, construction control and on-going monitoring appropriate to the various geotechnical categories.

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Table A.1 — Basic requirements for various geotechnical categories

1	2	3	4
Activity	Category 1	Category 2	Categories 3 and 4
Geotechnical investigation	Qualitative geotechnical investigations including a systematic description of the soil profile and groundwater conditions and identification of problem soils. May be supplemented by basic field and laboratory tests.	As per category 1 supplemented by routine field and laboratory tests producing quantitative geotechnical data for design purposes.	As per category 2 but including specialised field and laboratory tests as specified by the geotechnical engineer.
Design of geotechnical aspects	Routine methods of design including “deemed-to-satisfy” design procedures.	Quantitative design checks of geotechnical aspects such as bearing capacity, settlement, earth pressure, etc., generally at a level that can be executed as part of the structural design process.	Specialised geotechnical design involving cooperation between geotechnical engineer and structural designer. Use of advanced methods of analysis and soil structure interaction analysis as appropriate.
Supervision and checking during construction	Routine inspection at critical stages of work. Normal construction quality control (compaction densities, index tests, etc).	Systematic checking by designer to confirm validity of design assumptions. Periodic inspections by geotechnical engineer at appropriate times. Additional laboratory and field tests if required. Formal construction quality control programme.	Regular and detailed inspection by geotechnical engineer with additional field and laboratory tests as appropriate. May require a more rigorous and/or comprehensive construction quality control programme.
Monitoring	Generally none or reactive only.	Monitoring generally limited to ensuring critical performance criteria are met (e.g. settlement monitoring of foundations). May make use of datum surveys only with follow-up surveys only if required.	Monitoring of critical parameters such as piezometer levels, ground movements, anchor loads etc. during construction often coupled with use of observational method during design and construction. Planned inspection / monitoring of critical aspects of work into the future.

A.3.1.2 Geotechnical category 4 should normally include provisions and rules in addition to those given for categories 1 to 3 above or alternative to those in this standard.

A.3.2 Geotechnical investigations

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A.3.2.1 The scope and intensity of geotechnical investigations should be adjusted to suit the geotechnical category of the project as indicated in table A.1.

A.3.2.2 The minimum requirements for any investigation include a systematic description of the ground profile and the ground-water conditions and identification of any problem ground conditions on the site.

A.3.2.3 For categories 2 to 4, a phased approach may be appropriate including a desk study, preliminary fieldwork and detailed investigations.

A.3.2.4 The investigation should include sufficient field and laboratory tests to provide a reliable assessment of the characteristic values of ground parameters for use in design calculations. The method whereby such parameters are determined should be commensurate with the sensitivity of the design to the parameter concerned and the risks involved. For geotechnical category 1 structures, typical parameters based on a systematic description of the ground profile or comparable local experience may suffice. For categories 2 to 4, a quantitative assessment is generally required. Specialised laboratory and field tests are likely to be required for categories 3 and 4.

A.3.2.5 The extent of the investigation in plan and depth should be such that, as far as reasonably practicable, the properties of the ground lying outside the extent of the investigation will not exert a significant effect on the design or performance of the structure.

A.3.2.6 The results of the investigation should be compiled into a geotechnical report which should form part of the geotechnical design report described in A.3.3. The geotechnical report should normally include:

- a) a description of the proposed project;
- b) a description of the purpose, extent and methods of investigation;
- c) a presentation of all available geotechnical data and information including physical features of the site (geology, topography, vegetation, etc.);
- d) a description of the ground profile and ground water conditions and their variation across the site;
- e) results of field and laboratory tests; and
- f) a geotechnical interpretation of the information stating assumptions made, problem conditions expected and any special precautions required during design or construction.

A.3.3 Geotechnical design

A.3.3.1 The level of geotechnical design should be adjusted to suit the geotechnical category of the project as indicated in table A.1.

A.3.3.2 The minimum requirements for any design should be the assessment of safety and serviceability of the structure. For category 1 structures, an assessment based on comparable local experience or “deemed-to-satisfy” design rules may suffice. For categories 2 to 4, design calculations are required.

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A.3.3.3 The assumptions, data, methods of analysis and results of the verification of safety and serviceability shall be recorded in a geotechnical design report. The level of detail of the report will vary according to the geotechnical category. For category 1, a single sheet may suffice. In the case of categories 1 and 2 structures, the geotechnical design report may form part of the Structural design report.

A.3.3.4 For categories 2 to 4, the geotechnical design report shall include a plan for construction control, monitoring and maintenance as appropriate.

A.3.4 Supervision and checking during construction

A.3.4.1 The level of supervision and checking should be adjusted to suit the geotechnical category of the project as indicated in table A.1. and should be specified in the geotechnical design report.

A.3.4.2 Supervision and checking should ensure that:

- a) the conditions encountered on site during construction are compatible with the conditions assumed in the design; and
- b) the construction work is carried out in accordance with the requirements of the design.

A.3.4.3 Any difference between the conditions observed on site during construction and those described in the geotechnical design report should be referred to the designer without delay for a decision whether a change to the design or sequence of construction is required.

A.3.4.4 Site operations shall be checked for compliance with the method and sequence of construction assumed in the geotechnical design report. Any non-compliance should be reported to the designer without delay.

A.3.4.5 All temporary loading caused by construction activities including construction plant, stockpiling of materials, etc. should be checked to ensure that the loading for which the structure was designed is not exceeded.

A.3.4.6 The results of all supervision and checking activities and follow-up actions should be recorded in writing.

A.3.5 Monitoring during construction and design life

A.3.5.1 The level of monitoring required should be adjusted to suit the geotechnical category of the project as indicated in table A.1. and should be specified in the geotechnical design report.

A.3.5.2 Monitoring should include observations and measurements of the behaviour of the structure, the ground and the surroundings as appropriate.

A.3.5.3 Monitoring should evaluate:

- a) any differences between predicted and observed performance during construction that may require, for example, the implementation of remedial work or a change in construction sequence; and
- b) the long term safety and serviceability of the structure.

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A.3.5.4 In the case of unexpected events or performance, the extent and frequency of monitoring should be reviewed.

A.3.5.5 The measurements and observations from the monitoring programme should be evaluated and interpreted without delay. Such evaluation should normally be done in a formalised and quantitative manner. Any unexpected performance of the structure should be reported to the designer and the cause thereof investigated on site without delay.

A.4 Observational method

A.4.1 Where prediction of geotechnical behaviour is difficult, it may be appropriate to apply the observational method in which the design is reviewed during construction and amendments made as required.

A.4.2 When applying this method, the following requirements should be met before construction commences:

- a) acceptable limits of behaviour should be established;
- b) the range of likely behaviour should be assessed and it should be shown that there is an acceptable probability that the actual behaviour will be within acceptable limits;
- c) a plan of monitoring should be devised to assess whether the actual behaviour lies within acceptable limits. The monitoring should be done at sufficiently short intervals to clearly identify the likelihood of transgression of acceptable limits of behaviour at an early stage to allow contingency action to be undertaken successfully;
- d) the response time of monitoring measures and the procedures for analysing the output should be sufficiently rapid to permit timely intervention having due regard to the rapidity with which the situation could progress to an unacceptable state; and
- e) a plan of contingency actions should be devised which may be implemented if the monitoring reveals behaviour outside acceptable limits.

A.4.3 During construction, monitoring should be carried out as planned.

A.4.4 The results of the monitoring should be assessed as soon as possible and the planned contingency actions should be put into place if the prescribed limits of behaviour are exceeded.

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Annex B
(informative)

Guidance on partial material and resistance factors

B.1 Scope and application

B.1.1 The clauses of SANS 10160-1 on design verification require the use of partial material factors in the verification of the various limit states. Further reference to the use of these partial factors is made in clauses 5 and 6.

B.1.2 Values of partial material factors and partial resistance factors would normally be provided in a geotechnical design code. In the absence of a South African geotechnical design code, guidance on the selection and application of these factors is given in this annex. These factors may be amended in a future South African geotechnical design code.

B.1.3 The partial factors given in this informative annex may be used conjunction with SANS 10160-1.

B.2 Partial factors for material properties

B.2.1 Design values for geotechnical parameters may, in accordance with 5.3.2, be derived from the characteristic value using the following equation:

$$X_d = X_k / \gamma_M \quad (\text{B.1})$$

B.2.2 The values of the partial material factors γ_M to be used in this equation are given in table B1 for each of the limit states given in SANS 10160-1.

Table B.1 — Partial factors for soil parameters (γ_M)

1	2	3	4	5
Soil parameter	Symbol	Partial factor		
		Limit state ^b		
		STR, STR-P	EQU, GEO	ACC
Angle of shearing resistance ^a	γ_ϕ	1,0	1,25	See B.2.5
Effective cohesion	γ_c	1,0	1,25	
Undrained shear strength	γ_{cu}	1,0	1,4	
Unconfined strength	γ_{qu}	1,0	1,4	
Weight density	γ_γ	1,0	1,0	
^a This factor applied to $\tan \phi'$.				
^b From SANS 10160-1.				

B.2.3 The value of the partial factor should be taken as the reciprocal of the specified value if such a reciprocal value produces a more onerous effect than the specified value.

B.2.4 The design values of geotechnical parameters should be used in the determination of both geotechnical actions (for example earth pressure) and geotechnical resistances (for example bearing capacity).

B.2.5 For the ACC limit state, the partial factors on soil properties should be selected according to the particular circumstances of the accidental situation. In general, geotechnical actions acting in combination with accidental actions should be calculated using un-factored soil properties.

B.3 Partial resistance factors for piles and anchors

B.3.1 In the design of axially loaded piles and anchors, and where static equilibrium is provided by tension piles or anchors, the design values for the resistance of such elements may determined from equation B.2.

$$R_d = R_k / (\gamma_R \times \gamma_{R;d}) \quad (B.2)$$

where γ_R is a partial resistance factor and $\gamma_{R;d}$ is a partial factor for uncertainty in the resistance model (see B3.3).

NOTE 1 EN 1997-1 assigns different partial resistance factors to bored, driven and CFA piles. It also distinguishes between base, shaft and combined pile resistance.

NOTE 2 Analyses of model uncertainty carried out on South African data (Dithinde, 2007) do not support the adoption of different partial resistance factors for the various pile types. The data is insufficient to permit differentiation between the contributions of shaft and base resistance in assigning partial resistance factors.

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NOTE 3 In EN 1997-1, no guidance is given on an appropriate value for the partial resistance model factor. The value of this factor is to be provided in the National Annexes.

NOTE 4 The partial resistance factors and partial resistance model factors (γ_R and $\gamma_{R,d}$) given below are based on values given in NA to BS EN 1997-1 and reliability analyses carried out on South African data,

B.3.2 The values of the partial resistance factors to be used in equation B.2 are given in table B.2 for each of the limit states given in SANS 10160-1. These values apply irrespective of whether the characteristic resistance is derived from calculations based on ground parameters or load tests.

Table B.2 — Partial resistance factor (γ_R) for piles and anchors

1	2	3	4	5	6	7
Element	Loading	Symbol	Partial resistance factors			
			Limit state ^a			
			STR, STR-P	EQU	GEO	ACC
Piles	Compression	γ_t	1,0	-	1,6	1,0
	Tension	$\gamma_{s,t}$	1,25	1,4	1,7	1,0
Anchors	Tension	γ_a	1,1	1,4	1,1	1,1

^a From SANS 10160-1.

B.3.3 Where the characteristic resistance of a pile or anchor is derived from calculations based on ground parameters, the partial factor for uncertainty in the resistance model $\gamma_{R,d}$ should be taken as 1,5. No such factor is applied when characteristic values are determined from load tests.

B.3.4 The partial resistance factors in table B2 are applied in conjunction with unfactored material parameters, i.e. $\gamma_M=1,0$, except as noted in B.3.5.

B.3.5 The partial resistance factors in table B2 are applied in conjunction with the partial factors for soil parameters given in table B1 for the calculation of unfavourable actions on piles owing to, for example, transverse loading or negative skin friction in the GEO limit state.

B.3.6 Where such unfavourable actions are the result of uplift due to heave or downdrag due to negative skin friction, an upper bound to the force which the ground can transmit to the pile should be introduced as a design action. This upper bound value should be based on an assessment of the highest design value of skin friction likely to occur.

NOTE In such cases, note the requirement in B.2.3 regarding the use of reciprocal values of the partial material properties.

B.3.7 Models used for calculating the resistance of piles or anchors from ground test results should have been verified by load tests and from comparable experience.

Annex C
(informative)

**Basic guidance on design of foundations
and evaluation of earth pressures**

C.1 Principle

C.1.1 Some additional guidance may be required from other standards and publications before these requirements can be applied to common design situations such as the design of foundations and the determination of earth pressures.

C.1.2 Basic guidance is provided for the ultimate limit state design of spread footings and piled foundations and the assessment of earth pressures for category 1 and category 2 structures.

C.1.3 In the case of foundation design, the contents are applicable only to foundations subject to vertical or near vertical compressive loading in the absence of extraneous loading due to ground movement.

C.1.4 In the case of earth pressure, this annex applies only to the calculation of the active earth pressures or earth pressure at rest for normally consolidated, granular soils on retaining structures up to 7,5 m high in granular soils. Factors such as the effects of heaving clay, tension cracking, etc are not included.

C.1.5 Detailed geotechnical design using a recognised geotechnical design standard is required for structures or situations outside the above limitations.

C.1.6 This annex deals only with verification of the ultimate limit state by calculation based on ground test results. Empirical design methods, the accident limit state and the serviceability limit state are not considered.

C.1.7 The basic procedures involved in the design is set out rather than to recommend or prescribe any particular method of analysis.

C.1.8 The procedures described are based on the requirements of EN 1997-1.

C.2 Design of spread footings

C.2.1 Basic steps

C.2.1.1 The design of spread footings involves three basic steps:

- a) sizing the footing to prevent bearing capacity failure (failure of the ground);
- b) design the footing to prevent structural failure; and
- c) checking the settlement of the foundation against the serviceability requirements of the structure.

C.2.1.2 C.2.1.1(a) and C.2.1.1(b) are the basic steps dealt with in this annex.

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C.2.2 Bearing resistance – Sizing the foundation

C.2.2.1 The basic steps involved in determining the required size of a spread footing are outlined in C.2.2.2. to C.2.2.3.

C.2.2.2 For each load case, combine the actions in accordance with SANS 10160-1. In particular, the STR, STR-P and GEO combinations should be evaluated. Where the foundations are subject to horizontal loading or overturning moments, the effect of both favourable and unfavourable vertical actions should be considered. The actions considered should include the weight of the foundation, the weight of the foundation and any backfill material, imposed loads and earth pressures. Water pressures not caused by the loading of the soil should be included.

C.2.2.3 Select characteristic values of soil parameters including soil strength and density taking due account of the variation of such parameters with depth and across the site.

C.2.2.4 Based on judgement and information from the geotechnical investigation of the site, select a provisional size of foundation and depth of founding.

C.2.2.5 Using a recognised analytical model, calculate the design bearing resistance of the foundation (R_d). The calculations should be carried out using both factored soil parameters for the GEO and EQU limit states and unfactored soil parameters for the STR and STR-P limit states as described in B.2.

NOTE 1 Sample analytical methods are given in EN 1997-1. The method used should be capable of dealing with load eccentricity, load inclination and the presence of a water table where applicable.

NOTE 2 Figures C.1 and C.2 give the bearing resistance of shallow spread footings subject to concentric, vertical loads for undrained loading on clays and drained loading on granular soils respectively.

C.2.2.6 For each load combination, check whether the requirements for the design verification of the ultimate limit state given in SANS 10160-1 have been satisfied. In the case of the GEO and EQU limit states, the resistance calculated using factored soil parameters should be used.

NOTE Most bearing resistance equations yield a value for the vertical component of bearing capacity of the foundation. Provided the bearing resistance has been calculated taking account of the load inclination and eccentricity, it is generally sufficient to verify that the vertical component of the foundation load is less than the vertical bearing resistance of the foundation. Special consideration should be given for loads with large eccentricities foundations or a significant horizontal component.

C.2.2.7 Adjust the size of the footing if required and repeat the process. The minimum size of footing is that required to ensure that the requirements for the design verification of the ultimate limit state given in SANS 10160-1 are satisfied for all action combinations under all limit states (STR, STR-P, GEO etc).

C.2.3 Structural design

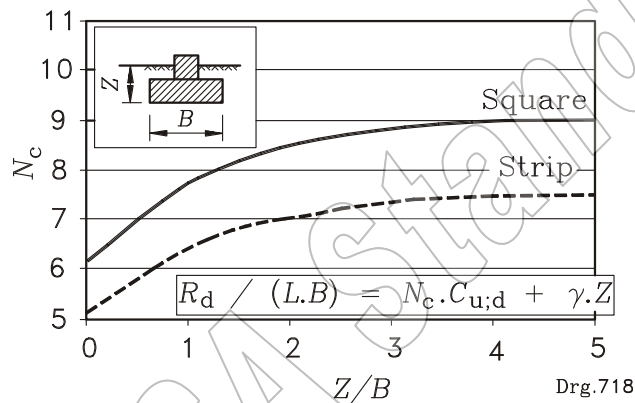
C.2.3.1 The design resistance of the foundation system should be determined in accordance with the relevant material-specific design codes.

C.2.3.2 The structural resistance of the footing should be verified for the physical dimensions of the footing as selected in C.2.2(g) even if a smaller footing would satisfy the requirements of a particular limit state.

C.2.3.3 The bearing pressure beneath a rigid foundation may be assumed to be distributed linearly.

C.2.3.4 The structural resistance of the foundation should be checked under all load combinations as different load combinations may govern the sizing of the footing and the structural design thereof.

NOTE In many instances, the size of the footing will be governed by the GEO limit state. The structural design is usually governed the STR or STR-P limit state. The critical limit state and load combination for structural design can often be assessed by inspection. Where any uncertainty exists, the structural resistance of the foundation should be verified for all potentially critical limit states load combinations.

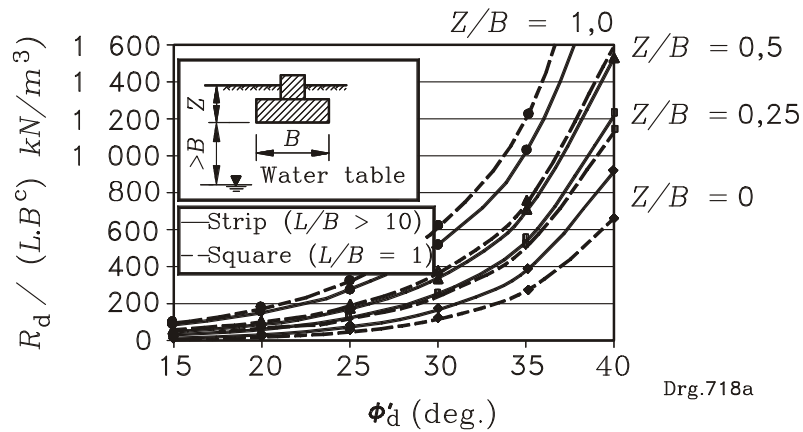


For STR and STR-P limit states $C_{u,d} = C_{u,k}$

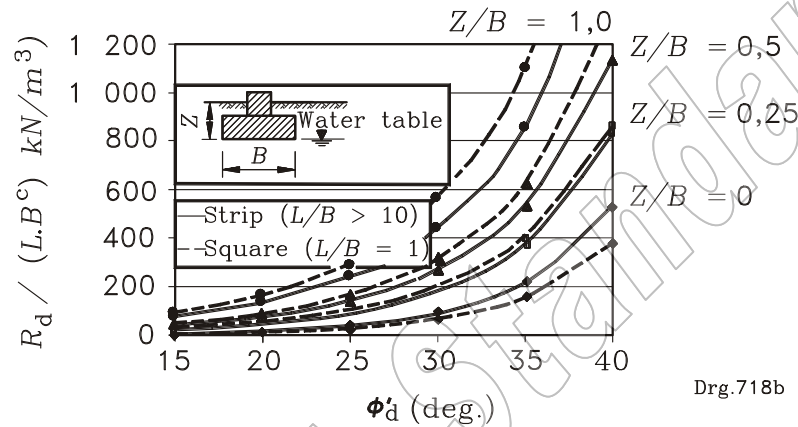
For GEO and EQU limit states $C_{u,d} = C_{u,d} / 1,4$

Figure C.1 — Design resistance of spread footings on un-drained clay for vertical, concentric loading

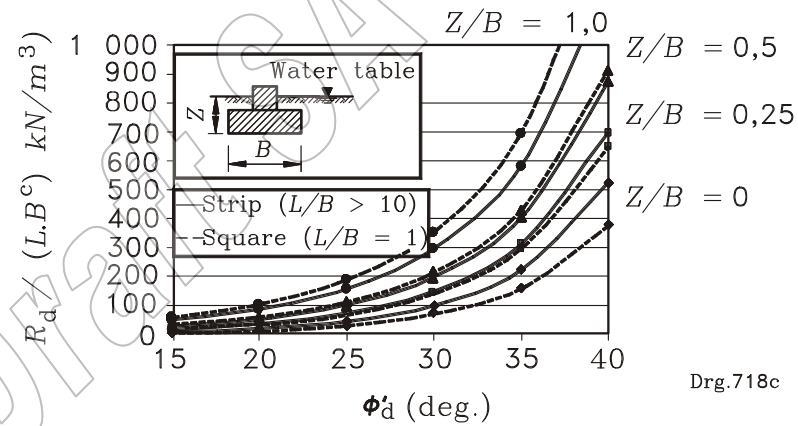
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a) Deep water table



b) Water table at founding level



c) Water table at surface

NOTE 1 For STR and STR-P limit states $\phi'_d = \phi'_k$

NOTE 2 For GEO and EQU limit states $\phi'_d = \text{atan}(\tan \phi'_k) / 1,25$

NOTE 3 Based on $\gamma = 18\text{kN/m}^3$, $\gamma' = 10\text{kN/m}^3$. (After EN1997-1, annex D.4)

Figure C.2 — Design resistance of shallow spread footings on a non-cohesive soil under drained conditions for vertical, concentric loading

C.3 Design of axially loaded piles

C.3.1 General

C.3.1.1 EN 1997-1 provides three methods for the design of axially loaded piles based on:

- a) load testing of piles (static or dynamic);
- b) analysis of pile driving records (driving formula or wave equation analysis); or
- c) calculations using ground test results.

C.3.1.2 In C.3.1.1(a) and C.3.1.1(b) the characteristic value of the compressive resistance of the pile is a function of the mean of the measured or estimated pile capacities and the minimum pile capacity, each divided by a correlation factor. The value of the correlation factors depends on the type of test and the number of piles tested.

C.3.2 Calculation of compressive resistance from ground test results

C.3.2.1 General

C.3.2.1.1 In the assessment of compressive resistance by calculation using ground test results, the design compressive resistance of the pile is derived from:

$$R_{c,d} = R_{b,d} + R_{s,d}$$

where, for each pile

$$R_{b,d} = \frac{R_{b,k}}{\gamma_b \times \gamma_{R,d}} \quad \text{and} \quad R_{s,d} = \frac{R_{s,k}}{\gamma_s \times \gamma_{R,d}}$$

C.3.2.1.2 The values of the partial resistance factors (γ_s and γ_b) for various pile types are given in 7-4.3. The value of the resistance model factor $\gamma_{R,d}$ is discussed in C.3.2.4

C.3.2.1.3 Two methods are given in En1997-1 for determining the characteristic values of base and shaft resistance, one based on profiles of in situ tests (CPT, SPT, pressure meter, etc) and the other on soil strength parameters as described in C.3.2.2 and C.3.2.3.

C.3.2.2 Calculated resistance from profiles of in situ test results

C.3.2.2.1 A profile of in situ test results is a series of results from in situ tests carried out at intervals over the full depth of the soil profile at a single location on the site. An example is the results of SPT tests from a single borehole.

C.3.2.2.2 The compressive resistance of the pile ($R_{c,cal} = R_{b,cal} + R_{s,cal}$) is calculated using a commonly recognised analytical model for each profile of test results.

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C.3.2.2.3 The characteristic value of the compressive resistance is then determined from the mean and minimum of the calculated pile capacities, each divided by a correlation factor, in the same way as with load test results. This takes account of the variation of pile capacity across the site in determining the characteristic value of the compressive resistance of the pile.

C.3.2.3 Calculated resistance from soil strength parameters

C.3.2.3.1 When the compressive resistance of the pile is determined from soil strength parameters, a recognised analytical model should be used. Un-factored characteristic strength parameters are used in the calculation.

C.3.2.3.2 The variability of the conditions on site should be taken into account in the selection of the characteristic strength parameters, taking due account of the volume of soil from which the resistance is derived. In the case of pile base resistance, localised pockets of low strength material could govern the capacity of the pile base, however, the shaft resistance will depend on the average strength of the soil over the length of the shaft. Thus, different characteristic values may be applicable for the assessment of base and shaft resistance.

C.3.2.4 Resistance model factor

C.3.2.4.1 The value of the partial factor for uncertainty in the resistance model ($\gamma_{R,d}$) is not defined in EN1997-1 and depends on the analytical model used in the calculation. Typical resistance model factors for use with recognised analytical models would range from 1,0 to 1,5. Guidance is given in B.3.3.

C.3.2.4.2 Ideally, the value of the resistance model factor should be determined by comparing the calculated pile resistance with the resistance determined from load tests carried out on the site.

C.3.3 Analysis of pile groups

C.3.3.1 General

C.3.3.1.1 The analysis of pile groups should be carried out using a recognised analytical model. Many such models are based on an idealised assessment of the stiffness of individual piles under axial and transverse loading using elastic subgrade methods, Winkler springs or $P - y$ curves.

C.3.3.1.2 The ground and pile stiffness parameters should be selected to represent the range of values likely to be encountered on the site. No partial factors are applied to the stiffness parameters used in the analysis.

C.3.3.1.3 The pile group should be analysed for all load combinations and all applicable limit states (STR, STR-P, GEO, etc).

C.3.3.1.4 In the analysis of single piles or pile groups consisting of two or more piles in a single row, the possible deviation of piles or the point of load application from their intended positions should be considered (see 5.4.5).

C.3.3.1.5 The minimum value of favourable vertical loading should be checked in the case of pile groups subject to horizontal loading.

C.3.3.2 Geotechnical design

For each load combination, the geotechnical design of the pile group should demonstrate that the following limit states are not exceeded:

- a) ultimate limit states of failure of a single pile under compressive or tensile or horizontal loads;
- b) ultimate limit states of failure of the pile foundation as a whole under compressive or tensile or horizontal loads.

C.3.3.3 Structural design

C.3.3.3.1 The design resistance of the piles and pile cap should be determined in accordance with the relevant materials codes.

C.3.3.3.2 For each load combination, the structural design of the pile group should demonstrate that the following limit states are not exceeded:

- a) ultimate limit states of failure of a single pile in tension, compression, bending, shear or torsion;
- b) ultimate limit states of failure of the pile cap in bending or shear;
- c) ultimate limit states of collapse or severe damage to a supported structure caused by excessive displacement or differential displacements of the pile foundation;
- d) serviceability limit states in the supported structure caused by displacement of the piles.

C.4 Earth pressures

C.4.1 General

C.4.1.1 Earth pressures acting on structures should be determined using established soil mechanics principles in accordance with the requirements of 7.3.

C.4.1.2 In the particular case of granular backfill placed against a vertical surface, the approximate earth pressure distributions given in C.4.2 may be used in conjunction with an appropriate model factor.

C.4.2 Approximate earth pressure distributions

C.4.2.1 Where granular backfill with a horizontal surface is placed against a vertical wall, the approximate earth pressure distributions given in figure C.3 may be used. The use of these approximate distributions is limited to walls less than 7,5 m high (approximately equivalent to two basement levels). Situations falling outside these limitations must be designed in accordance with C.4.1.

C.4.2.2 The approximate earth pressure distributions are based on the following two types of granular backfill:

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Type 1: Coarse grained sands or gravels with a low fines content such that the compacted material has the properties of a free draining granular material.

Type 2: Fine grained silty sands with low plasticity fines.

NOTE The distributions given below are based on an effective angle of shearing resistance of 35° for a type 1 material and 30° for type 2 material (characteristic values). The bulk weight density of the compacted material is assumed to be 20 kN/m^3 and 18 kN/m^3 respectively for type 1 and type 2 materials. In both cases, wall friction and cohesion have been ignored.

C.4.2.3 The approximate earth pressure distributions given in figure C.3 include:

- a) earth pressure due to self weight of soil and ground water;
- b) earth pressure due to uniformly distributed surcharge q on ground surface and
- c) earth pressure due to compaction.

C.4.2.4 The earth pressure at any depth is the larger of the compaction earth pressure and the sum of the first two components (earth pressure due to self weight of soil and ground water plus earth pressure due to surcharge). The resulting earth pressure distribution with depth is shown by the dotted line labelled “approximate earth pressure distribution” in figure C.3.

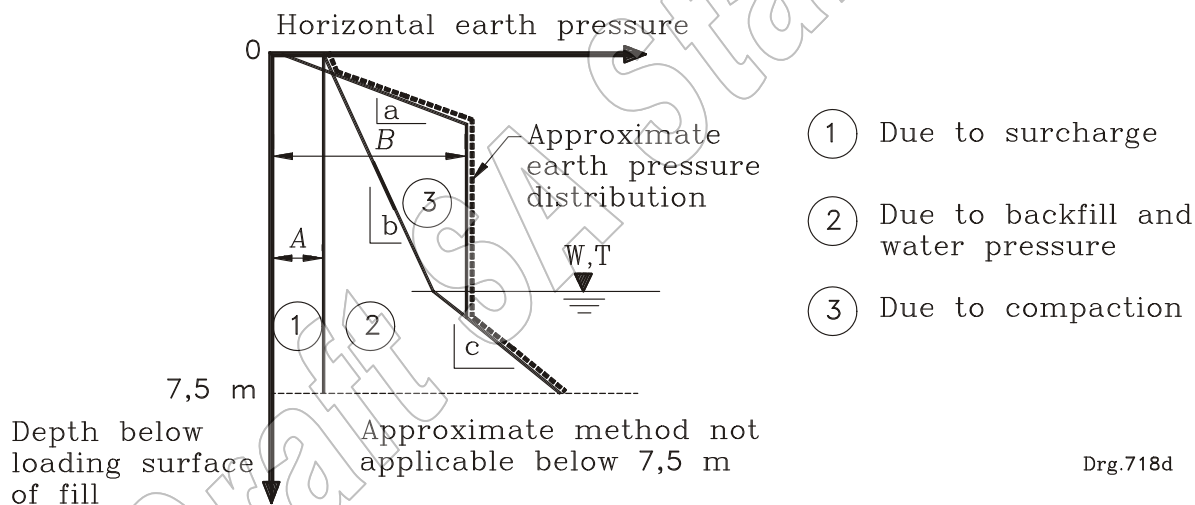


Figure C.3 — Horizontal earth pressure distribution (approximate method)

The values of the parameters A, B, a, b and c are given in Table B.1.

Table C1 — Parameters for calculation of approximate earth pressure distribution

1	2	3	4	5	6	7
Backfill type	Wall type	Parameters				
		GEO Limit state (Design values)				
		A kPa	B kPa	a kPa/m	b kPa/m	c kPa/m
1	Rigid	$0,73q$	$5,3\sqrt{p}$	46	12,1	17,8
	Yielding	$0,49q$	$5,3\sqrt{p}$	69	8,2	15,5
2	Rigid	$0,83q$	$5,1\sqrt{p}$	38	12,8	18,0
	Yielding	$0,59q$	$5,1\sqrt{p}$	54	9,0	15,8
STR and STR-P LIMIT STATES (Characteristic values)						
1	Rigid	$0,47q$	$4,1\sqrt{p}$	56	10,1	16,6
	Yielding	$0,30q$	$4,1\sqrt{p}$	87	6,4	14,5
2	Rigid	$0,55q$	$3,9\sqrt{p}$	44	11,0	7,3
	Yielding	$0,37q$	$3,9\sqrt{p}$	66	7,3	14,9

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NOTE 1 A rigid wall corresponds to at rest K_0 conditions, a yielding wall corresponds to active K_a conditions.

NOTE 2 q = uniform surcharge on surface (kPa), typically not less than 10 kPa for areas accessible by vehicles.

NOTE 3 P = compaction line load per unit length taken as the sum of the static load and centrifugal vibrator load per unit width of the roller drum. For a typical 10 t static mass, single drum, self-propelled vibratory roller,
 $p = 150$ kN/m. For a typical 750 mm wide, tandem drum pedestrian roller, $P = 33$ kN/m.

NOTE 4 a , b , and c are rates of increase in earth pressure with depth.

NOTE 5 The horizontal earth pressure at any depth is the larger of the earth pressure due to compaction and the sum of the earth pressures due to the self weight of the soil or ground water and surface surcharge.

NOTE 6 Earth pressures due to surcharge and compaction are treated as variable actions. Earth pressures due to self weight of the soil and ground water are treated as permanent actions.

NOTE 7 The values given for the GEO limit state are design values which include a partial material factor $\gamma_{\phi'} = 1,25$ and a partial action factor $\gamma_q = 1,3$ on variable actions.

NOTE 8 The values given for the STR and STR-P limit states are characteristic values in which all partial material factors and partial action factors are taken as 1,0. Partial action factors should be applied as appropriate to obtain design values (see also note 6).

NOTE 9 The values in the table include a partial factor for uncertainties in modelling of the effects of actions $\gamma_{s,d} = 1,1$.

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