Recommendations on Excavations
EAB

3rd Edition

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Deutsche Gesellschaft für Geotechnik e. V.
German Geotechnical Society

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Preface

In response to a recognisably overwhelming requirement, the *Deutsche Gesellschaft für Erd- und Grundbau e.V.* (German Society for Geotechnical and Ground Engineering) – now the Deutsche Gesellschaft für Geotechnik (*German Geotechnical Society*) – called the Working Group for Tunnel Engineering into life in 1965 and awarded chairmanship to the highly respected and now sadly missed Prof. *J. Schmidbauer*. The wide-ranging tasks of the Working Group were divided into three subgroups "General", "Open Cut Methods" and "Trenchless Technology". The "Open Cut Methods" Working Group, under the chairmanship of Prof. Dr.-Ing. habil. Dr.-Ing. E. h. *Anton Weißenbach*, at first busied itself only with the urgent questions of analysis, design and construction of excavation enclosures. The German Society for Geotechnical and Ground Engineering published the preliminary results of the Working Group as the "Recommendations for Calculation of Braced or Anchored Soldier Pile Walls with Free Earth Support for Excavation Structures, March 1968 Draft".

During the course of work involving questions concerning the analysis, design and construction of excavation enclosures, it was recognised that these matters were so comprehensive that the *Deutsche Gesellschaft für Erd- und Grundbau e.V.* decided to remove this area from the "Tunnel Engineering" Working Group and transfer it to a separate Working Group, that of "Excavations"; the personnel involved were almost completely identical with those of the previous "Open Cut Methods" Group. The new Working Group's first publication, with the title "Recommendations of the Working Group for Excavations", appeared in the journal "*Die Bautechnik*" (Construction Technology) in 1970. It was based on a thorough revision, restructuring and enhancement of the proposals published in 1968 and consisted of 24 numbered Recommendations, primarily dealing with the basic principles of the analysis of excavation enclosures, soldier pile walls, sheet pile and in-situ concrete walls for excavations, and with the influence of buildings located adjacent to excavations.

In the years following this, the Working Group for Excavations published new and revised Recommendations at two-year intervals. As a stage was reached at which no further revisions were envisaged, the *Deutsche Gesellschaft für Erd- und Grundbau e.V.* decided to summarise the 57 Recommendations strewn throughout the "*Die Bautechnik*" journal, Volumes 1970, 1972, 1974, 1976, 1978 and 1980, and to present them to the profession in a single volume.

In the 2nd edition, published in 1988, the Recommendations were partly revised and, in addition, supplemented by nine further Recommendations dealing with "Excavations in Water", published in draft form in the 1984 volume of *Bautechnik*, and by two further Recommendations for "Pressure Diagrams for

Braced Retaining Walls", published in *Bautechnik* in 1987. Four further Recommendations resulted from partial restructuring and endeavours to make the Recommendations more clearly understandable. The revisions and supplements are described in an article in the 1989 volume of *Bautechnik*.

In the 3rd edition, published in 1994, a number of the Recommendations were revised and three new Recommendations on "Excavations with Special Ground Plans" added. The revisions to the existing Recommendations are described in the 1995 volume of *Bautechnik*. In the same issue, the three new Recommendations were also presented to the professional public in draft form. In addition, the 3rd edition includes an appendix, containing the principal from building control standard regulations, where they are relevant to stability analysis.

At the same time that the 3rd edition of the EAB was being compiled, the Working Group for Excavations was deeply occupied with the implementation of the new partial safety factor approach in geotechnical and ground engineering. This was because, on the one hand, several members of the Working Group for Excavations were also represented in the "Safety in Geotechnical and Ground Engineering" Committee, which was compiling DIN V 1054-100. On the other hand, it became increasingly obvious that excavation structures were affected by the new regulations to a far greater degree than other ground engineering structures. In particular the specification in the new draft European regulations EN 1997-1 – applying partial safety factors to the shear strength on the one hand and to the actions on the other – was unacceptable. Compared to previously tried and tested practice it led to results that, in places, suggested considerably greater dimensions, but also to results that were not conservative enough. In contrast to this stood the draft DIN 1054 counter-model, in which the partial safety factors identified using the classical shear strength method were applied in the same manner to the external actions, earth pressure and soil resistances. In EAB-100, published in 1996 at the same time as the ENV 1997-1 and DIN 1054-100, the practical applications of both concepts were introduced and the differences illuminated. This was intended to make the decision in favour of the German proposals, which was still open, more straightforward for the profession.

Two important decisions were subsequently made: on the one hand, EN 1997-1 was published in a format that included the proposals of the new DIN 1054 as one of three allowable alternatives. On the other hand DIN 1054-100 was modified such that the originally envisaged superpositioning of earth pressure and passive earth pressure design values was no longer permissible, because this route could not be reconciled with the principle of strict separation of actions and resistances. In addition, one now has characteristic action effects and characteristic deformations when adopting characteristic actions for the given system, with the result that generally only one analysis is required for verification of both bearing capacity and serviceability. The 4th (German) edition of the EAB, published in 2009, rested entirely upon these points, but also expanded them by supplementary regulations, just as it has in the past. Moreover, all the Recommendations of the 3rd edition have been thoroughly revised. Recommendations on the use of the modulus of subgrade reaction method and the finite element method (FEM), as well as a new chapter on excavations in soft soils, have been added. These had previously been presented to the profession for comments in the 2002 and 2003 volumes of the *Bautechnik* journal, based on the global safety factor approach. Much correspondence, some very extensive, has been taken into consideration in the 4th edition.

Once the 4th edition was complete in 2006, *Anton Weißenbach* stepped down from his position as chairman after more than 40 years and retired from the working group along with a number of other long-term members.

Following this, one of the main emphases of the Working Group on Excavations – now under the Chairmanship of the undersigned – was Recommendation R 102 “Modulus of subgrade reaction method”, presented in draft to the professional community, completely revised, in 2011 in the journal *Bautechnik*. In line with the imminent introduction of the Eurocodes as binding building regulations it became necessary to adapt the 4th edition of the Recommendations to the provisions of DIN EN 1997-1:2009, in conjunction with National Annex DIN 1997-1/NA:2010-12 and the supplementary regulations of DIN 1054:2010-12. All Recommendations were thoroughly examined, revised where necessary and adapted to accommodate recent developments. The experienced user will note that the revisions to this 5th edition are relatively minor. It was possible to retain the majority of the tried and tested regulations, because the safety philosophy has not altered in principle compared to the 4th edition.

Section 10, “Excavations in water”, on the other hand, has been substantially revised. In future, the planner must examine risks arising from erosion processes, anisotropic permeability and hydraulic failure more extensively than was previously required. As a result of sophisticated developments in monitoring technology and increased demands, Section 14, “Measurements on excavations”, has been completely reformulated.

By revising existing Recommendations and publishing new ones, the Working Group for Excavations aims to:

a) Simplify the analysis of excavation enclosures;
b) Unify load approaches and analysis methods;
c) Guarantee the stability of the excavation structure and its individual components and;
d) Improve the economic efficiency of excavation structures.

The Working Group for Excavations would like to express thanks to all who have supported the work of the Working Group in the past, in correspondence or by other means, and requests your further support for the future.

A. Hettler

Notes for the User

1. The Recommendations of the Working Group for Excavations represent technical regulations. They are the result of voluntary efforts within the technical-scientific community, are based on valid and current professional principles, and have been tried and tested as *general best practice*.
2. The Recommendations of the Working Group for Excavations may be freely applied by anyone. They represent a yardstick for flawless technical performance; this yardstick is also of legal relevance. A duty to apply the Recommendations may result from legislative or administrative provisions, contractual obligations or other legal requirements.
3. Generally speaking, the Recommendations of the Working Group for Excavations are an important source of information for professional conduct in normal design cases. They cannot reproduce all possible special cases in which more advanced or more restrictive measures may be required. Note also that they can only reflect best practice at the time of publication of the respective edition.
4. Deviations from the suggested analysis approaches may prove necessary in individual cases, if founded on appropriate analyses, measurements or empirical values.
5. Use of the Recommendations of the Working Group for Excavations does not release anybody from their own professional responsibility. In this respect, everybody works at their own risk.

Contents

1 Introduction

1.1 Engineering prerequisites for applying the Recommendations (R l)

If no other stipulations are explicitly made in the individual Recommendations, they shall apply under the following engineering preconditions:

1. The complete height of the retaining wall is lined.
2. The soldier piles of soldier pile walls are installed such that intimate contact with the ground is ensured. The lining or infilling can consist of wood, concrete, steel, hardened cement-bentonite suspension or stabilised soil. It shall be installed such that the contact with the soil is as uniform as possible. Soil excavation should not advance considerably faster than plank installation. Also see DIN 4124.
3. Sheet pile walls and trench sheet piles are installed such that intimate contact with the ground is ensured. Toe reinforcement is permitted.
4. In-situ concrete walls are executed as diaphragm walls or as bored pile walls. Accidental or planned spacing between the piles is generally lined according to Paragraph 2.
5. In the horizontal projection, struts or anchors are arranged perpendicular to the retaining wall. They are wedged or prestressed such that contact by traction with the retaining wall is guaranteed.
6. Braced excavations are lined in the same manner on both sides with vertical soldier pile walls, sheet pile walls or in-situ concrete walls. The struts are arranged horizontally. The ground on both sides of the braced excavation displays approximately the same height, similar surface features and similar subsurface properties.

If these preconditions are not fulfilled, or those in the individual Recommendations, and no Recommendations are available for such special cases, this does not exclude application of the remaining Recommendations. However, the consequences of any deviations shall be investigated and taken into consideration.

1.2 Governing regulations (R 76)

1. Following its introduction, geotechnical analysis and design in Germany are controlled by DIN EN 1997-1: Eurocode 7: Geotechnical Design – Part 1: General Rules (Eurocode 7), in conjunction with the corresponding National Annex:

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– DIN EN 1997-1/NA: National Annex – Nationally Determined Parameters – Eurocode 7: Geotechnical design – Part 1: General rules
  and
– DIN 1054: Subsoil – Verification of the Safety of Earthworks and Foundations – Supplementary Rules to DIN EN 1997-1.

These three coordinated standards are summarised in the 'Handbuch Eurocode 7, Band 1'.
The National Annex represents a formal link between the Eurocode EC 7-1 and national standards. It states which of the possible analysis methods and partial safety factors are applicable in the respective national domains. Remarks, clarifications or supplements to Eurocode EC 7-1 are not permitted. However, the applicable, complementary national codes may be given. The complementary national codes may not contradict Eurocode EC 7-1. Moreover, the National Annex may not repeat information already given in Eurocode EC 7-1.

2. In addition, the following Eurocode programme standards govern excavation structures:

EN 1990 Eurocode 0:	Basis of structural design
EN 1991 Eurocode 1:	Actions on structures
EN 1992 Eurocode 2:	Design of concrete structures
EN 1993 Eurocode 3:	Design of steel structures
EN 1995 Eurocode 5:	Design of timber structures
EN 1998 Eurocode 8:	Design of structures for earthquake resistance

3. The Eurocode 7 Handbook, Volume 1 contains general rules for geotechnical engineering. It is supplemented by the analysis standards which, where necessary, have been adapted to the partial safety factor approach. The following codes in particular also represent the governing standards for excavation structures:

DIN 4084:	Global stability analyses
DIN 4085:	Subsoil – Calculation of earth pressure
DIN 4126:	Cast-in-situ concrete diaphragm walls; design and construction
DIN 4093:	Design of ground improvement – Jet grouting, deep mixing or grouting

4. The standards covering ground exploration, investigation and description are not affected by the adaptation to partial safety factors and therefore remain valid in their respective latest editions, or are superseded by Eurocode 7 and EN ISO standards:

EN 1997-2, Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing

EN 1997-2/NA: National Annex – Nationally Determined Parameters – Eurocode 7, Part 2: Ground investigation and testing
DIN 4020: Geotechnical investigations for civil engineering purposes – Supplementary rules to DIN EN 1997-2
DIN 4023: Geotechnical investigation and testing – Graphical presentation of logs of boreholes, trial pits, shafts and adits
EN ISO 22475-1: Geotechnical investigation and testing – Sampling by drilling and excavation and groundwater measurements – Part 1: Technical principles for execution, supersedes DIN 4021 and DIN 4022
EN ISO 14688-1: Geotechnical Investigation and testing – Identification and classification of soil – Part 1: Identification and description, superseded by DIN 4022-1
EN ISO 14688-2: Geotechnical Investigation and testing – Identification and classification of soil – Part 2: Principles for classification, superseded by DIN 4022-1
EN ISO 14689-1: Geotechnical Investigation and testing – Identification and classification of rock – Part 1: Identification and description, superseded by DIN 4022-1
EN ISO 22476-2: Dynamic probing
EN ISO 22476-3: Standard Penetration Test
DIN 4094-2: Subsoil – Field testing – Part 2: Borehole dynamic probing
DIN 18121 to DIN 18137: Investigation of soil samples
DIN 18196: Soil classification for civil engineering purposes
DIN 1055-2: Soil properties

5. The Eurocode 7 Handbook, Volume 1, only replaces the analysis section of the previous standards DIN 4014 "Bored piles", DIN 4026 "Driven piles", DIN 4125 "Ground anchorages – Design, construction and testing" and DIN 4128 "Grouted piles (in-situ concrete and composite piles) with small diameter". The new European standards from the "Execution of special geotechnical works" series now take the place of the execution sections of these standards:

EN 1536:	Bored piles
EN 1537:	Grouted anchors
EN 1538:	Diaphragm walls
EN 12063:	Sheet pile walls
EN 12699:	Displacement piles
EN 12715:	Grouting
EN 12716:	Jet grouting
EN 12794:	Precast concrete – foundation piles
EN 14199:	Micropiles

6. The following execution standards are not affected by the adaptation to European standards and therefore continue to govern excavation structures:

   DIN 4095: Drainage systems protecting structures
   DIN 4123: Excavations, foundations and underpinnings in the area of existing buildings
   DIN 4124: Excavations and trenches

1.3 Safety factor approach (R 77)

1. In contrast to the original probabilistic safety factor approach, this safety factor approach, upon which both the new European standards generation and the new national standards generation are based, no longer rests on probability theory investigations, e.g. the beta-method, but on a pragmatic splitting of the previously utilised global safety factors into partial safety factors for actions or effects and partial safety factors for resistances.

2. The foundation for stability analyses is represented by the characteristic or representative values for actions and resistances. The characteristic value is a value with an assumed probability which is not exceeded or fallen short of during the reference period, taking the lifetime or the corresponding design situation of the civil engineering structure into consideration; it is characterised by the index “k”. Characteristic values are generally specified based on testing, measurements, analyses or empiricism.

   Variable actions can also be given as representative values, thus taking into consideration that not all variable, unfavourable actions occur simultaneously at their maximum values.

3. If the bearing capacity in a given cross-section of the retaining wall or in an interface between the retaining wall and the subsoil needs to be analysed, the effects in these sections are required:

   – as action effects, e.g. axial force, shear force, bending moment;
   – as stresses, e.g. compression, tension, bending stress, shear stress or equivalent stress.

   In addition, further effects of actions may occur:

   – as oscillation effects or vibrations;
   – as changes to the structural element, e.g. strain, deformation or crack width;
   – as changes in the position of the retaining wall, e.g. displacement, settlement, rotation.

4. Two types of ground resistances are differentiated:

   a) The characteristic shear strength of the soil is the decisive basic resistance parameter. For consolidated soils or soils drained for testing these are the shear parameters φ'_k and c'_k, and for unconsolidated soils or soils not drained for testing the shear parameters $\varphi_{u,k}$ and $c_{u,k}$. These variables are defined as cautious estimates of the mean values, because the shear strength at a single point of the slip surface is not the decisive value but the average shear strength in the slip surface.
   b) The soil resistances are derived from the shear strength, directly:

      – the sliding resistance;
      – the bearing capacity;
      – the passive earth pressure;

      and indirectly via load tests or empirical values:

      – the toe resistance of soldier piles, sheet pile walls and in-situ concrete walls;
      – the skin resistance of soldier piles, sheet piles walls, in-situ concrete walls, and of ground anchors and soil and rock nails.

   The term “resistance” is only used for the failure state of the soil. As long as the failure state of the soil is not achieved by effects, the term “soil reaction” is used.

5. The cross-section and internal resistance of the material are the decisive factors in the design of individual components. The detailed specification standards continue to be the governing standards here.

6. The characteristic values of the effects are multiplied by partial safety factors, those of the resistances are divided. Where necessary, representative values should be adopted by applying combination factors. The variables acquired in this way are known as the design values of effects or resistances respectively and are characterised by the index *d*. Five limit states are differentiated for stability analyses, in line with R 78 (Section 1.4).

7. In terms of the GEO 2 and STR limit state safety analyses according to R 78, Paragraph 4 (Section 1.4), Eurocode EC 7-1 provides three options. DIN 1054 is based on design approach 2 inasmuch as the partial safety factors are applied to the effects and to the resistances. To differentiate between this and the other permitted scenario, in which the partial safety factors are not applied to the effects but to the actions, this procedure is designated as design approach 2* in the Commentary to Eurocode EC 7-1 [134].

8. In addition to the actions, the design situation shall be taken into consideration in the analyses. To this end the existing load cases LC 1, LC 2 and

LC 3, adopted for use in analyses to DIN 1054:2005-01, have been superseded by the design situations for use in analyses to the Eurocode 7 Handbook, Volume 1, and DIN EN 1990 as follows:

DS-P (persistent situation);
DS-T (transient situation) and;
DS-A (accidental situation).

The former LC 2/3 corresponds to design situation DS-T/A. In addition, there is the seismic design situation, DS-E. More detailed information can be found in the Eurocode 7 Handbook, Volume 1.

1.4 Limit states (R 78)

1. The term "limit state" is used with two different meanings:

   a) In soil mechanics, the state of the soil in which the displacement of the individual soil particles against each other is so great that the mobilisable shear strength achieves its greatest values in either the entire soil mass, or at least in the region of a failure plane, is known as the *limit state of plastic flow*. It cannot become greater even if more movement occurs, but may become smaller. The limit state of plastic flow characterises the active earth pressure, passive earth pressure, bearing capacity, slope stability and overall stability.
   b) A limit state in the sense of the new safety factor approach is a state of the load-bearing structure where, if exceeded, the design requirements are no longer fulfilled.

2. The following limit states are differentiated in conjunction with the partial safety factor approach:

   a) The ultimate limit state is a condition of the structure which, if exceeded, immediately leads to a mathematical collapse or other form of failure. In the Eurocode 7 Handbook, Volume 1, it is referred to as ULS (ultimate limit state). Five cases of ULS are differentiated, see Paragraphs 3, 4 and 5.
   b) The serviceability limit state (SLS) is a condition of the structure which, if exceeded, no longer fulfils the conditions specified for its use. In the Eurocode 7 Handbook, Volume 1, it is referred to as SLS (serviceability limit state).

3. Eurocode 7 defines the following limit states:

   a) EQU: loss of equilibrium of the structure, regarded as rigid, without the influence of soil resistances.

b) STR: inner failure or very large deformation of the structure or its components, whereby the strength of the materials is decisive for resistance.
c) GEO: failure or very large deformation of the subsoil, whereby the strength of the soil or rock is decisive for resistance.
d) UPL: loss of equilibrium of the structure or ground due to uplift or water pressure.
e) HYD: hydraulic failure, inner erosion or piping in the ground, caused by a hydraulic gradient.

4. In order to transfer it to the provisions of DIN 1054 the GEO limit state shall be divided into GEO 2 and GEO 3 limit states:

   a) GEO 2: failure or very large deformation of the subsoil in conjunction with identification of the action effects and dimensions; i.e. when utilising the shear strength for passive earth pressure, sliding resistance and bearing resistance and when analysing lower failure plane.
   b) GEO 3: failure or very large deformation of the ground in conjunction with analysis of overall stability, i.e. when utilising the shear strength for analysis of the safety against slope failure and global failure and, generally, when analysing the stability of engineered slope stabilisation measures.

5. The previous limit states are replaced as follows:

   a) The previous limit state GZ 1A now corresponds without restrictions to the EQU, UPL and HYD limit states.
   b) The previous limit state GZ 1B corresponds without restrictions to the STR limit state. In addition, the GEO 2 limit state applies in conjunction with external design, i.e. when utilising the shear strength for passive earth pressure, sliding resistance and bearing capacity and when analysing lower failure plane.
   c) The previous GZ 1C limit state corresponds to the GEO 3 limit state, in conjunction with analysis of overall stability, i.e. when utilising the shear strength for analysis of safety against slope failure and overall stability.

   Analysis of the stability of engineered slope stabilisation measures is always allocated to the GEO limit state. Depending on the specific design and function they may be dealt with:

   – either in the sense of the previous limit state GZ 1B adopting the provisions of the GEO B limit state;
   – or in the sense of the previous limit state GZ 1C adopting the provisions of the GEO C limit state.

6. The EQU, UPL and HYD limit states describe the loss of static equilibrium:

   – analysis of safety against overturning EQU;
   – analysis of safety against uplift UPL;
   – analysis of hydraulic heave safety HYD.

   Only actions are associated with these limit states, no resistances. The governing limit state condition is:

   $F_d = F_k \cdot \gamma_{dst} \leq G_k \cdot \gamma_{stb} = G_d$

   i.e. the destabilising action F_k, multiplied by the partial safety factor $\gamma_{dst} \geq 1$, may only be as large as the stabilising action G_k, multiplied by the partial safety factor $\gamma_{stb} < 1$.

7. The STR and GEO 2 limit states describe the failure of structures and structural elements or the failure of the ground. They include:

   – analysis of the bearing capacity of structures and structural elements subjected to soil loads or supported by the soil;
   – verification that the bearing capacity of the soil is not exceeded, e.g. by passive earth pressure, bearing capacity or sliding resistance.

   Verification that the bearing capacity of the ground is not exceeded is performed exactly as for any other construction material. The limit state condition is always the governing condition:

   $E_d = E_k \cdot \gamma_F \leq R_k/\gamma_R = R_d$

   i.e. the characteristic action effect E_k, multiplied by the partial safety factor γ_F for actions or γ_E for effects, may only become as large as the characteristic resistance R_k, divided by the partial safety factor γ_R.

8. The GEO 3 limit state is peculiar to geotechnical and ground engineering. It describes the loss of overall stability. They include:

   – analysis of safety against slope failure;
   – analysis of safety against global failure of retaining structures.

   The limit state condition is always the governing condition:

   $E_d \leq R_d$

   i.e. the design value E_d of the effects may only become as large as the design value of the resistances R_d. The geotechnical actions and resistances are determined using the design values for shear strength:

   $\tan \varphi'_d = \tan \varphi'_k/\gamma\varphi'$ and $c'_d = c'_k/\gamma_c'$ or
   $\tan \varphi_{u,d} = \tan \varphi_{u,k}/\gamma\varphi'$ and $c_{u,d} = c'_k/\gamma_{cu}$

i.e. the tangent of the angle of internal friction φ and the cohesion c are reduced by applying the partial safety factors γ_φ' and γ_c'.

9. The serviceability limit state describes the state of a structure at which the conditions specified for its use are no longer fulfilled, without a loss of bearing capacity. It is based on verification that the anticipated displacements and deformations are compatible with the purpose of the structure. For excavations, the SLS includes the serviceability of neighbouring buildings or structures.

1.5 Support of retaining walls (R 67)

1. Retaining walls are called unsupported if they are neither braced nor anchored and their stability is based solely on their restraint in the ground.
2. Retaining walls are called yieldingly supported if the wall support points can yield with increasing load, e.g. in cases where the supports are heavily inclined toward the excavation base and when using non-prestressed or only slightly prestressed anchors.
3. Retaining wall supports are called slightly yielding in the following cases:
   a) Struts are at least tightly connected by frictional contact (e.g. by wedges).
   b) Grouted anchors are prestressed and locked off to at least 80 % of the computed characteristic effect required for the next construction stage, see Section 7.
   c) A tight connection via frictional contact is established with piles, which verifiably display only a small head deflection under load.
4. Retaining wall supports are known as nearly inflexible if designed according to R 22, Paragraph 1 (Section 9.5), utilising increased active earth pressure, and the struts and anchors are prestressed and locked off according to R 22, Paragraph 10.
5. Retaining wall supports are defined as inflexible only if they are designed either for reduced or for the full at-rest earth pressure according to R 23 (Section 9.6) and the supports are prestressed accordingly. Furthermore, the anchors of anchored retaining walls shall be socketed in non-yielding rock strata or be designed substantially longer than required by calculations.

   If the requirements of Paragraphs 4 or 5 are fulfilled and, in addition:

   – a rigid retaining wall is installed and;
   – excessive toe deflections are avoided;

an excavation structure may be regarded as a low-deflection and low-deformation structure.

1.6 Planning and examination of excavations (R 106)

1. If the planner is not in possession of sufficient expertise and experience, a suitable planner shall be contracted for the geotechnical design of the excavation in line with the Eurocode 7 Handbook, Volume 1, Paragraph 1.3, A 3.
2. The term “geotechnical expert” used in the Recommendations is understood as it is used in the Eurocode 7 Handbook, Volume 2, Paragraph A 2.2.2.
3. Excavations are classified as Geotechnical Category GC 1, GC 2 or GC 3. Annex A5 lists criteria for classifying excavations based on the Eurocode 7 Handbook, Volume 1, Paragraph A 2.1.2.
4. A Geotechnical Design Report in line with the Eurocode 7 Handbook, Volume 1, Paragraph 2.8 shall be compiled for excavations.

   With regard to Geotechnical Categories GC 2 and GC 3, the Geotechnical Design Report for the excavation should contain the following points:

   - Description of the plot and its environs, in particular adjacent buildings;
   - Description of ground conditions with reference to the Geotechnical Report in accordance with the Eurocode Handbook, Volume 2, Paragraph A 7;
   - Description of the proposed excavation structure;
   - Description of the actions from adjacent structures;
   - Description of the impacts on adjacent areas and structures;
   - Characteristic values of soil and rock properties, and of water levels and flows;
   - Proposal for excavation structure and identification os possible risks;
   - Design situation and partial factors;
   - Where necessary, an explanation of the necessity, suitability and sufficiency of the observational method;
   - Analyses, including information on the analysis method and plans;
   - Specifications for manufacturing controls, e.g. load tests;
   - Specifications for measurements and monitoring.
5. Where excavations are classified as Geotechnical Category GC 3 a geotechnical expert shall be consulted.
6. When executing excavations classified as Geotechnical Category GC 2 or GC 3, it is recommended to employ a suitable site supervisor in possession of the appropriate experience and excavation knowledge. For excavations classified as Geotechnical Category GC 3 it is recommended to also employ the geotechnical expert discussed in Section 5 to check the detailed design and to assess the results of measurements and monitoring.

2 Analysis principles

2.1 Actions (R 24)

1. DIN EN 1990, including DIN EN 1990/NA and DIN 1054, differentiates between permanent and variable actions. In excavation structures the permanent actions include:

   – self-weight of the excavation structure, if necessary taking provisional bridges and excavation covers into consideration;
   – earth pressure as a result of the self-weight of the soil, if necessary taking cohesion into consideration;
   – earth pressure as a result of the self-weight of adjacent structures;
   – horizontal shear forces created by vaults, and shear forces from retaining walls and frame-like structures;
   – water pressure as a result of the contractually agreed upon reference water level of groundwater or open water.

   The Eurocode 7 Handbook, Volume 1, Section 9.5.1, A(10), states that, in simplification, the earth pressure resulting from a variable, unbounded distributed load $p_k \leq 10\ kN/m^2$ is adopted as a permanent action. Also see Paragraph 2.

2. According to Recommendations R 55 to R 57 (Sections 2.6 to 2.8), the variable actions are differentiated into a component adopted as an unbounded distributed load $p_k = 10\ kN/m^2$ and a component adopted either as a distributed load q_k in excess of this or as a strip load, line load or point load on a small contact area. While the unbounded distributed load $p_k = 10\ kN/m^2$ according to Paragraph 1 is treated as a permanent load, the other variable actions are differentiated for the cases described below as a function of the duration and frequency of the action based on DIN 1054.

3. Beside the permanent actions it is generally sufficient to base the stability analysis on the following, regularly occurring variable actions:

   – live loads acting directly on provisional bridges and excavation covers according to R 3, Paragraph 1 (Section 2.5);
   – earth pressure from live loads according to R 3, Paragraph 1 (Section 2.5);
   – earth pressure from live loads in conjunction with structures adjacent to the excavation.

4. In special cases it may be necessary to consider the following actions, beside the typical case loads:

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- centrifugal, brake and nosing forces, e.g. for excavations adjacent to or below railway or tram lines;
- exceptional loads and improbable or rarely occurring combinations of loads or points of application of loads;
- water pressure resulting from water levels that may exceed the agreed design water levels, e.g. water levels that will flood the excavation if they occur or at which the excavation shall be intentionally flooded;
- the impact of temperature on struts.

The impact of temperature changes on the remaining excavation structure need not be investigated for flexible walls.

5. In unusual cases it may be necessary to consider exceptional loads, beside the loads of the typical case, e.g.:

- impact of construction machinery against the supports of provisional bridges or excavation covers or against the intermediate supports of buckling protection devices;
- loads caused by the failure of operating or stabilising installations, if the effects cannot be countered by appropriate measures;
- loads caused by the failure of particularly susceptible bearing members, e.g. struts or anchors;
- loads resulting from scour in front of the retaining wall.

Short-term exceptional loads, e.g. such as those occurring when testing, overstressing, or loosening anchors or struts, may be treated as exceptional loads.

6. The actions specified in Paragraphs 3 to 5 are allocated to design situations corresponding to the different safety requirements. Also see R 79 (Section 2.4).

7. When determining representative values the following combination factors Ψ may be adopted to determine loads:

- For excavations adjacent to old buildings $\Psi = 1.0$ is adopted for the foundation loads. For new builds the representative values given in the structural engineer's analysis are adopted.
- If the vertical loads resulting from road and rail traffic corresponding to R 55, Section 2.6 are adopted, combination factors $\Psi = 1.0$ are applied. However, it is also possible to adopt different values if the analyses are based on regulations issued by the respective transport authorities.
- In general, when using simplified load assumptions in accordance with R 56, Section 2.7 for live loads from site traffic and site operations, and in accordance with R 57, Section 2.8 for live loads from excavators and lifting equipment, the combination factors are adopted at $\Psi = 1.0$.

2.2 Determination of soil properties (R 2)

1. In principle, the soil properties required for stability analyses are specified as the immediate result of geotechnical investigations based on DIN EN 1997-2, including DIN EN 1997-2/NA and DIN 4020 "Geotechnical Investigations for Civil Engineering Purposes". To take the heterogeneity of the subsurface and the inaccuracy of sampling and testing into due consideration, surcharges and allowances shall be applied to the values identified during testing before they are adopted as characteristic values in an analysis. This applies particularly to shear strength. Also see Paragraph 3.

2. Two cases are differentiated when specifying characteristic values for the unit weight:

   a) For stability analyses in the GEO 2, STR and GEO 3 limit states, i.e. in particular when analysing the embedment depth, when determining the action effects and when analysing the safety against global failure, the mean value may be adopted as the characteristic value.
   b) When analysing safety against uplift UPL, safety against hydraulic failure HYD and safety against heave EQU, the lower characteristic values are the governing values.

3. Characteristic values for shear strength should be selected as conservative estimates of the statistical mean value. Minor deviations from the mean value may be acceptable if the available samples are sufficiently representative of the soil in the region of the excavation structure being analysed. A larger deviation shall be assumed for a small data pool and heterogeneous subsoil.

4. The capillary cohesion of cohesionless soil, in particular of sand, may be taken into consideration if it cannot be lost by drying or flooding or due to rising groundwater or water ingress from above during construction work.

5. The cohesion of a cohesive soil may only be considered if the soil does not become pulpy when kneaded and if it is certain that the soil state will not change unfavourably compared to its original condition, e.g. when thawing following a period of frost.

6. The following restrictions shall be considered when transferring the shear strength determined by testing laboratory samples to the behaviour of the in-situ ground:

   a) The shear strength of cohesive and rock-like soils can be greatly reduced by hair cracks, slickensides and cracks or intercalations of slightly cohesive or cohesionless soils.

b) Certain slip surfaces may be predetermined by faulting and inclined bedding planes. For example, Opalinus Clay (*Opalinuston*, a Middle-Jurassic (Dogger alpha, Aalenium) clay (*Al (1) Clay*)), Nodular Marl (*Knollenmergel*, a marly claystone containing carbonate nodules; Upper Triassic, Carvian) and *Tarras* (a type of Puzzolan) are all considered especially prone to sliding.
c) In fine-grained soils, e.g. kaolin clay, and in soils with a governing proportion of montmorillonite, the residual shear strength may be the governing factor.

7. If the results of appropriate soil mechanics laboratory tests are not available, the characteristic soil properties may be specified as follows:

   a) As far as it is sufficiently known from local experience that similar subsurface conditions are prevalent, the soil properties identified from previous investigations carried out in the immediate vicinity may be adopted. This requires expertise and experience in the geotechnical field.
   b) If the type and quality of in-situ soils can be assigned to the soil groups specified in DIN 18196 based on drilling or soundings, and further laboratory and manual testing, analysis may be based on the soil properties given in Appendices A 3 and A 4, taking the respective restrictions into consideration.

8. The empirical values for cohesionless soils given in:

   – Table 3.1 for the unit weight based on Appendix A 3 or;
   – Table 3.2 for the shear strength based on Appendix A 3;

   may be adopted, if the following requirements are met:

   a) It shall be possible to allocate the soils to the tables in terms of grain size distribution, uniformity coefficient and relative density. See Appendix A 1 for classification of soils in terms of relative density.
   b) The given empirical values apply to both natural ground and made, cohesionless soils. The density of the soil may be improved in both cases by compaction.

   The table values may not be applied to soils with porous grains, such as pumice gravel and tuff sand.

9. The empirical values for cohesive soils given in:

   – Table 4.1 for the unit weight based on Appendix A 4 or;
   – Table 4.2 for the shear strength based on Appendix A 4;

   may be adopted if the soils can be allocated to the soil groups according to DIN 18 196 in terms of their plasticity and can be differentiated in terms of

their consistency. See Appendix A 2 for classification in terms of consistency.
The table values may not be adopted in any of the following cases:

a) They may not be adopted for mixed-grain soils where the type of fines on the one hand and the proportion of grain > 0.4 mm on the other do not allow the degree of plasticity to be reliably described, e.g. for sandy boulder clay.
b) They may not be adopted for the soils described in Paragraph 6.
c) They may not be adopted if a sudden collapse of the grain skeleton is possible, e.g. in loess (aeolian silt deposit).

2.3 Earth pressure angle (R 89)

1. The angles $\delta_{a,k}$ and $\delta_{p,k}$ between the direction of acting of the earth pressure or the passive earth pressure and the normal on the rear face of the wall depend on:

   – the characteristic wall friction angle δ_k;
   – the relative movement between wall and soil;
   – the selection of slip surface type;
   – the degree of mobilisation.

2. The characteristic wall friction angle δ_k is a measure of the largest possible physical friction between the wall and the ground. It is primarily dependent on the:

   – shear strength of the soil and;
   – surface roughness of the wall.

3. The following cases are differentiated in terms of the roughness of the wall:

   a) A rear wall face is known as "toothed" if, due to its shape, it displays such a convolute surface that the wall friction acting immediately between the wall and the ground is not decisive, but instead the friction in a planar failure surface in the ground, which only partly contacts the wall. This is always the case in pile walls. Even cut-off walls manufactured using a hardening cement-bentonite slurry with inserted sheet pile walls or soldier piles may be classified as toothed [123]. This also applies approximately for driven, vibrated or pressed sheet pile walls.
   b) The untreated surfaces of steel, concrete and wood can generally be considered as "rough", in particular the surfaces of soldier piles and infill walls.
   c) The surface of a diaphragm wall may be classified as "slightly rough" if filter cake development is low, e.g. for diaphragm walls in cohesive

soils. Empiricism indicates that this is also the case for diaphragm walls in cohesionless soils. However, if the formation of a filter cake during diaphragm wall construction can be avoided using suitable measures, or a heavily uneven wall surface is achieved, a higher absolute earth pressure angle than $|\delta_p| = ½\ \varphi$ may be adopted [148, 149].

d) All rear wall faces should be classified as “smooth” if the ground displays “smeary” properties due to its clay content and consistency.

4. Only if:

- earth pressure or passive earth pressure calculations are based on a curved or a non-circular slip surface and;
- it can be demonstrated according to R 9, Paragraph 1 (Section 4.8) that the sum of the characteristic actions directed downwards is at least as large as the upwards directed vertical component $B_{v,k}$ of the characteristic support force B_k;

may the physically possible wall friction be considered according to Paragraph 5 a).

If approximately planar slip surfaces are used, the earth pressure angle according to Paragraph 5 b) shall be reduced to compensate for the error occurring due to overestimation of the passive earth pressure coefficient K_p or underestimation of the earth pressure coefficient K_a.

5. The following wall friction angles and maximum earth pressure angles shall be adopted as a function of the friction angle φ'_k:

Wall texture	Curved slip surfaces	Planar slip surfaces
Toothed wall	$\|\delta_k\| = \varphi'_k$	$\|\delta_k\| \leq {}^2/_3 \cdot \varphi'_k$
Rough wall	$\|\delta_k\| \leq 27{,}5°$ $\|\delta_k\| \leq \varphi'_k - 2{,}5°$	$\|\delta_k\| \leq {}^2/_3 \cdot \varphi'_k$
Slightly rough wall	$\|\delta_k\| \leq {}^1/_2 \cdot \varphi'_k$	$\|\delta_k\| \leq {}^1/_2 \cdot \varphi'_k$
Smooth wall	$\|\delta_k\| = 0$	$\|\delta_k\| = 0$

a) The values in the middle column are wall friction angles, which may be adopted for curved or non-circular slip surfaces as the maximum angle of inclination for the active and the passive earth pressure.

b) The figures in the right column serve to compensate for the modelling error when planar slip surfaces are used. Planar slip surfaces may be adopted for active earth pressure regardless of the friction angle φ'_k, for passive earth pressure only for $\varphi'_k \leq 35°$.

c) If, during analysis of the vertical component of the mobilised passive earth pressure, correction of the earth pressure angle to R 9, Paragraph 2 d) (Section 4.7) is dispensed with, determination of passive earth pressure may only be based on curved slip surfaces.

6. The sign of the earth pressure angle is dependent on the relative displacement between the wall and the ground:

   a) For active earth pressure the earth pressure angle is positive if the earth wedge moves downwards more than the wall as shown in Figure R 89-1 a).

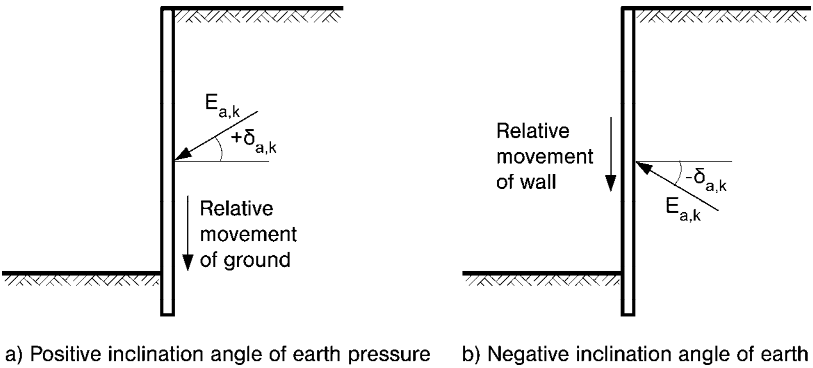


Figure R 89-1. Angle for active earth pressure

   b) For active earth pressure the earth pressure angle is negative if the wall moves downwards more than the ground as shown in Figure R 89-1 b).

   The same applies in principle for determination of the passive earth pressure. Also see Figure R 19-1 (Section 6.3).

2.4 Partial safety factors (R 79)

1. In principle, the value of the partial safety factor depends on the design situations, as specified in DIN EN 1990, including DIN EN 1990/NA and DIN 1054. Excavation structures are included in design situation DS-T (transient situation), in conjunction with the loads for the accidental situation in the design situation DS-A. Based on this the actions in accordance with R 24 (Section 2.1) are allocated as follows:

   a) The standard case according to R 24, Paragraph 3 corresponds to the design situation DS-T.

b) The special case according to R 24, Paragraph 4 corresponds to the design situation DS-T/A.
a) The exceptional case according to R 24, Paragraph 5 corresponds to the design situation DS-A.

2. The partial safety factors for actions for design situations DS-T and DS-A are based on DIN 1054. The partial safety factors for actions for the intermediate design situations DS-T/A are interpolated. This provides the partial safety factors for actions according to Table 6.1 in Appendix A 6.

3. Favourable variable actions may not be adopted for either of the limit states ULS or SLS.

4. In the serviceability limit state (SLS) the partial safety factors for permanent actions $\gamma_G = 1.00$ and for variable actions $\gamma_Q = 1.00$ are adopted. See R 83 for further details (Section 4.11).

5. The partial safety factors according to DIN 1054 for geotechnical resistances are summarised in Appendix A 6:

   – in Table 6.2 for resistances in the GEO 2 limit states;
   – in Table 6.3 for resistances in the GEO 3 limit state.

   The partial safety factors for the design situations DS-T/A are interpolated between those of design situations DS-T and DS-A, similar to those of the actions.

6. The numerical values for design situation DS-P in Appendix A 6 have been adopted as orientation values, but are placed in brackets because they generally do not govern excavation structures. Exceptions include:

   – analysis of deep-seated stability according to R 44, Paragraph 10 (Section 7.3), in excavations adjacent to structures;
   – analysis of global stability according to R 45, Paragraph 7 (Section 7.4), in excavations adjacent to structures;
   – design of struts according to R 52, Paragraph 14 (Section 13.7);
   – design of anchorages for walls in the fully excavated state.

2.5 General requirements for adopting live loads (R 3)

1. The following variable actions are described as live loads:

   – loads from road and rail traffic according to R 55 (Section 2.6);
   – loads from site traffic and site operations according to R 56 (Section 2.7);
   – loads from excavators and lifting equipment according to R 57 (Section 2.8).

See R 24 (Section 2.1) for classification of these loads into standard and exceptional loads.

2. If no precise investigations are carried out, the individual tyre contact widths of rubber-tyred vehicles and construction equipment are assumed as follows:

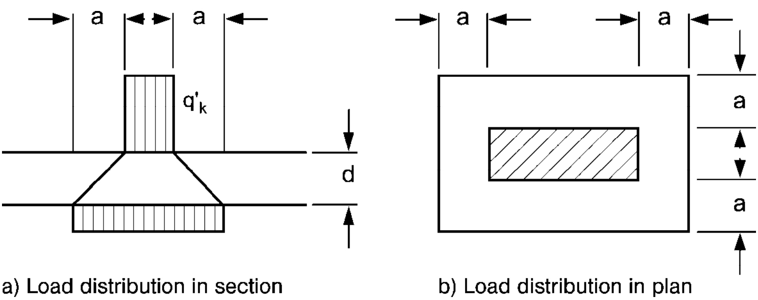
   – 0.60 m for wheel loads of 100 kN (10.0 t);
   – 0.46 m for wheel loads of 65 kN (6.5 t);
   – 0.40 m for wheel loads of 50 kN (5.0 t);
   – 0.30 m for wheel loads of 40 kN (4.0 t);
   – 0.26 m for wheel loads of 30 kN (3.0 t).

   Where required, these values may be linearly interpolated. The contact length in travel direction is always 0.20 m.

3. A load distribution in all directions within the upper road layers may be assumed as shown in Figure R 3-1 as follows, independent of the properties and thickness d of the load distributing layers:

   a) Transfer with a = d for the top/binder course and base courses of bituminous layers, concrete or tight stone pavement.
   b) Transfer with a = 0.75 · d for hydraulically stabilised gravel or crushed stone base courses.
   c) Transfer with a = 0.50 · d for non-stabilised gravel or crushed stone base courses.

   See the *Zusätzlichen Technischen Vertragsbedingungen und Richtlinien für den Bau von Verkehrsflächen (ZTV Beton-StB, ZTV Asphalt-StB, ZTV Pflaster-StB, ZTV SoB-StB)* for base course quality requirements.



a) Load distribution in section b) Load distribution in plan

Figure R 3-1. Load distribution in the upper road layers

4. If no road pavement is installed, the contact areas of rubber-tyred vehicles and construction equipment increase as a result of sinking into the surface. As an approximation, the contact area lengths and widths that apply to

paved roads in Paragraph 2 may be increased by 15 cm, if no precise investigations are carried out.

5. In order to determine the earth pressure, a point load or a bounded distributed load as shown in Figure R 3-2 a) may be converted to an equivalent strip load and the load projection be assumed at approximately 45° to the horizontal. If the effects of neighbouring loads overlap, a simplified approach with a common contact area for both loads may be applied as shown in Figure R 3-2b).
6. If, in strutted excavations, only one wall is loaded by earth pressure from live loads, the opposite wall shall be designed for the same action effects unless, for elastic retaining structures, the resulting earth pressure on the support points is analysed.

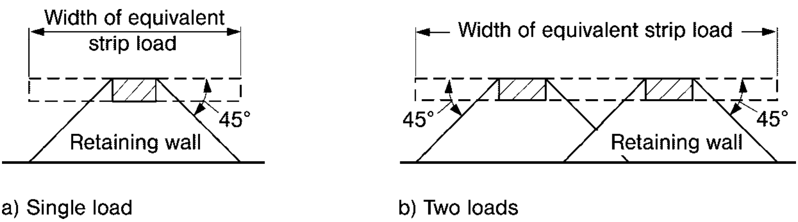


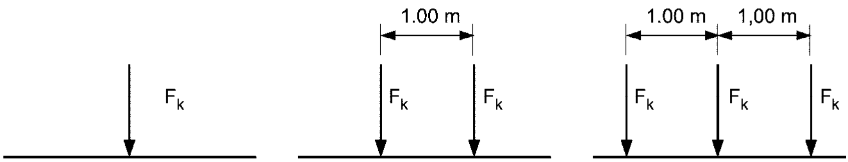
Figure R 3-2. Conversion of bounded distributed loads to strip loads

2.6 Live loads from road and rail traffic (R 55)

1. According to the German Road Transport Licensing Regulations (*StVZO*) of 29 April 2009, the allowable axle loads of commonly licensed road vehicles depend on the number and spacing of the axles. When analysing the stability of excavation structures it is sufficient to investigate the following load combinations:

   - Single axle loads
     of $1 \cdot F_k = 1 \cdot 115$ kN (11.5 t) = 115 kN (11.5 t) as shown in Figure R 55-1 a).
   - Double axle loads
     of $2 \cdot F_k = 2 \cdot 80$ kN (8.0 t) = 160 kN (16.0 t) as shown in Figure R 55-1 b).
   - Triple axle loads
     of $3 \cdot F_k = 3 \cdot 70$ kN (7.0 t) = 210 kN (21.0 t) as shown in Figure R 55-1 c).

The axle loads may be evenly distributed across all wheels of one axle or an axle group. An impact surcharge need not be taken into consideration.



a) Single axle load b) Double axle load c) Triple axle load

Figure R 55-1. Governing axle loads

2. The following recommendations apply to the determination of earth pressure acting on the retaining wall due to wheel loads according to Paragraph 1:

   – R 3, Paragraph 2 (Section 2.5), for the contact area;
   – R 3, Paragraph 3, for load distribution in the upper road layers;
   – R 3, Paragraph 5, for load distribution in the ground.

   The influence of vehicle wheels on the side of the vehicle away from the retaining wall, and the influence of vehicles in more distant lanes, need not be individually investigated. Instead, an unbounded distributed load $p_k = 10$ kN/m^2 is applied immediately adjacent to the wheel loads nearest to the retaining wall.

3. If it is certain that:

   – the loads according to Paragraph 1 will not be exceeded;
   – the road pavement including the bituminous base course layers consists of concrete or tight stone pavement and is at least 15 cm thick and;
   – a distance of at least 1.0 m remains between the wheel contact areas and the rear of the retaining wall;

   a specific investigation according to Paragraph 2 may be dispensed with and an unbounded distributed load of $p_k = 10$ kN/m^2 be adopted as equivalent load beginning at the rear edge of the wall. For lesser distances, the distributed load shall be located in a strip 1.50 m wide directly adjacent to the retaining wall and increased as follows:

   – by $q'_k = 10$ kN/m^2, if the contact areas remain at a distance of at least 0.60 m;
   – by $q'_k = 40$ kN/m^2, if no spacing is adhered to, e.g. in the region of provisional bridges.

Also see Figure R 55-2. The load transfer in the road pavement is already considered in these approaches.

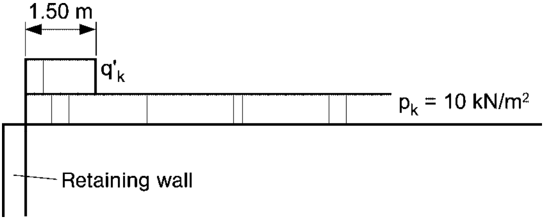


Figure R 55-2. Equivalent loads for road traffic at less than 1.00 m from the retaining wall

4. If, when applying the equivalent loads, vehicles heavier than those given in Paragraph 1 shall be taken into consideration, the equivalent strip loads q'_k given in Paragraph 3 may be converted in a ratio corresponding to the axle loads if the individual vehicles, tractors and trailers do not have more than three axles. Special investigations shall be carried out for vehicles with more than three axles, e.g. wagon-carrying trailers.

5. If a kerb is supported directly by the retaining wall, a horizontal nosing force shall be applied. When designing the kerb the nosing force is generally allocated according to R 24, Paragraph 3; when designing the excavation structure it is a special case according to R 24, Paragraph 4 (Section 2.1).

6. If the retaining wall lies within a rail vehicle load projection, the live loads or equivalent loads are adopted on the basis of the regulations of the transport service provider concerned. A dynamic coefficient need not be taken into consideration. It is sufficient to apply an unbounded distributed load of $p_k = 10$ kN/m^2 for tramlines if a minimum distance of 0.6 m between the ends of the sleepers and the retaining wall is adhered to. Centrifugal and nosing forces shall be taken into consideration as actions in the standard case where necessary.

7. When designing provisional bridges and excavation covers the live loads for road and rail traffic as given in DIN EN 1991-2 and DIN Technical Report 101 generally apply. The relevant regulations of the respective transport service provider apply to special rail traffic (e.g. tram lines). Depending on the local situation, and in agreement with the approving agencies, it may be possible to adapt the traffic load model (LM 1 in accordance with DIN Technical Report 101) using modified calibration factors (α_{Qi}, α_{gi}, α_{gr})

or to agree on loads to DIN 1072:1985-12. If analysis is based on DIN 1072 the loads given there are adopted as characteristic actions.

2.7 Live loads from site traffic and site operations (R 56)

1. Construction materials normally stored in the open or in a site hut are generally taken into consideration by means of an unbounded distributed load of $p_k = 10$ kN/m^2. If large earth masses or large quantities of steel, stones and similar materials are stored in the immediate vicinity of the excavation, more precise investigations in accordance with DIN 1055-1 or DIN EN 1991-1-1 shall be carried out. The same applies to silo loads.

2. When applying equivalent loads for vehicles licensed for general public roads, such as heavy goods vehicles, tractors and trailers, R 55, Paragraph 3 (Section 2.6) also applies when no road pavement is installed. If construction vehicles cannot be associated with the loads given in R 55, Paragraph 1, due to their axle loads or the number of axles, R 55, Paragraph 4 applies accordingly.

   It is not necessary to adopt live loads from site traffic if the influence of excavators and lifting equipment according to R 57, Paragraph 2 (Section 2.8) has already been taken into consideration for the same area. Excavators and lifting equipment that only travel along the outside of the excavation shall be taken into consideration as road vehicles.

3. If the earth pressure from construction vehicles is not determined with the help of equivalent loads according to Paragraph 2, the following recommendations apply:

   - R 3, Paragraph 2 (Section 2.5) for the contact areas of rubber-tyred vehicles;
   - R 3, Paragraph 3, for load transfer in the upper road layers;
   - R 3, Paragraph 4 for the increase in contact area where no pavement is present;
   - R 3, Paragraph 5, for load transfer in the ground.

   The influence of vehicle wheels on the side of the vehicle away from the retaining wall, and the influence of vehicles in more distant lanes, need not be individually investigated. Instead, an unbounded distributed load $p_k = 10$ kN/m^2 is applied immediately adjacent to the wheel loads nearest to the retaining wall.

4. When designing excavation covers, which will serve as working areas or storage areas for formwork, reinforced concrete and similar work, Paragraph 1 applies accordingly. The anticipated loads shall be adopted for

provisional bridges and excavation covers for site traffic. The same applies to non-rubber-tyred site traffic, e.g. roller compactors or crawler excavators. DIN 1072:1985-12, DIN EN 1991-2 and DIN Technical Report 101 apply accordingly with regard to dynamic coefficients, surcharges and exceptional loads. If several loaded vehicles, e.g. ready-mixed concrete vehicles, can simultaneously travel successively or park in one lane, or adjacent to each other in neighbouring lanes, this shall be taken into consideration. R 55, Paragraph 7 (Section 2.6) applies accordingly to traffic regulated by the German Road Transport Licensing Regulations (*StVZO*).

5. When designing struts, a vertical live load of at least $\overline{q}_k$ = 1.0 kN/m shall be applied to cover unavoidable loads caused by site operations, light covers, gantries, bracing and similar loads where larger vertical loads are not envisaged, beside self-weight and the normal force. Horizontal loads, e.g. resulting from bracing or formwork supports, shall be taken into consideration in strut design. Struts may not be loaded with live loads in utility trench construction with vertical or horizontal bracing or soldier pile walls lined with a plank curtain. Otherwise, also see R 52, Paragraph 5 (Section 13.7).
6. If no structural protection against the impact of construction machinery is installed, a point load P = 100 kN in all directions at a height of 1.20 m above the excavation level shall be taken into consideration when designing the supports of provisional bridges or excavation covers, and the intermediate supports of buckling protection devices.

2.8 Live loads from excavators and lifting equipment (R 57)

1. Excavators and lifting equipment operating at short distances from the excavation impose large stresses on the retaining wall structure. Separate investigation of the influence of earth pressure magnitude and distribution may only be dispensed with if the following distances to the retaining wall are adhered to:

   1.50 m for a gross weight of 10 t or a total load of 100 kN;
   2.50 m for a gross weight of 30 t or a total load of 300 kN;
   3.50 m for a gross weight of 50 t or a total load of 500 kN;
   4.50 m for a gross weight of 70 t or a total load of 700 kN.

   Intermediate values may be linearly interpolated. If the distances given here are adhered to it is sufficient to apply an unbounded distributed load of p_k = 10 kN/m^2.

2. If excavators or lifting equipment operate adjacent to the retaining wall at distances smaller than those given in Paragraph 1, the resulting earth pressure magnitude and distribution shall be determined. If this is based on the excavators or lifting equipment point loads, the following apply:

   a) The contact areas of tracked equipment are taken from the manufacturer's specifications.
   b) The contact areas of rubber-tyred equipment are adopted according to R 3, Paragraph 2 (Section 2.5).
   c) For information on load transfer in the upper road layers, see R 3, Paragraph 3.
   d) For information on the increase in contact area where no pavement is installed see R 3, Paragraph 4.
   e) For load transfer in the ground, see R 3, Paragraph 5.

   Where applicable, the effect of load distributing sub-bases such as excavator mattresses, timber packing or rails supported by sleepers may be taken into consideration.

3. When determining earth pressure according to Paragraph 2, all governing excavator and lifting equipment distances from the retaining wall and all governing positions of the crane chassis and the boom shall be taken into consideration. As an approximation, analysis may be based on the following load distribution in the standard case according to R 24, Paragraph 3 (Section 2.1):

   a) With the boom pointing in the direction of equipment travel:
   40 % of the total load at each of the two more heavily loaded wheels or half of the length of both tracks on tracked vehicles.
   b) With the boom positioned diagonally:
   50 % of the total load at the more heavily loaded wheel or half of the length of the more heavily loaded track on tracked vehicles.
   c) With the boom perpendicular to the direction of travel:
   40 % of the total load at the two more heavily loaded wheels or 80 % of the total load at the more heavily loaded track on tracked vehicles.

   The influence of loads acting on the respectively lower stressed wheels or tracks need not be individually investigated. Instead, an unbounded distributed load $p_k = 10$ kN/m^2 is applied immediately adjacent to the wheel loads nearest to the retaining wall.

4. As an approximation, the point loads of excavators and lifting equipment can be substituted by an unbounded distributed load $p_k = 10$ kN/m^2 and an additional strip load q'_k, which begins directly adjacent to the retaining wall as shown in Figure R 57-1 and covers the complete length travelled by the vehicle. For construction machinery on tracks, rubber-tyred construction

machinery with not more than two axles, and construction machinery running on rails supported by sleepers, the magnitude and width may be assumed as follows for transient design situation DS-T (principal loads), according to R 24 (Section 2.1), as a function of the distance to the retaining wall:

Total load (gross weight) of equipment	Additional strip load q'_k		Width of strip load q'_k
	Adjacent to wall	0.60 m from wall	
100 kN (10 t)	50 kN/m^2	20 kN/m^2	1.50 m
300 kN (30 t)	110 kN/m^2	40 kN/m^2	2.00 m
500 kN (50 t)	140 kN/m^2	50 kN/m^2	2.50 m
700 kN (70 t)	150 kN/m^2	60 kN/m^2	3.00 m

Intermediate values may be inserted linearly; weights below 10 t may be linearly extrapolated. Additionally, the following apply:

a) Supporting devices (outriggers) must have a floor contact area of at least 0.25 m² or be placed on an appropriate load distributing structure.
b) In principle, the distance between the retaining wall and the equipment refers to the floor contact area. However, if the equipment travels perpendicular to the side of the excavation, the vertical projection of the wheels or the tracks may not intersect the rear edge of the retaining wall. Where equipment travels on rails and sleepers, the distance to the sleeper ends represents the governing distance.
c) If the road surface is metalled, load distribution at 45° from the rear edge of the equivalent load may be assumed.

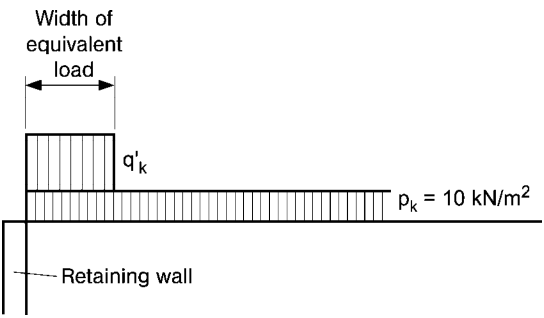


Figure R 57-1. Equivalent load for excavators and lifting equipment

5. The gross weight of excavators and lifting equipment consists of:

   – the operating weight of the equipment based on the manufacturer's specifications and;
   – the weight of the carried soil or any lifted loads.

6. If, in exceptional cases, a conceivable extreme load distribution case is investigated as a special case according to R 24 (Section 2.1), the values given in Paragraph 3 shall be increased as follows:

   – from 40 % to 50 %;
   – from 50 % to 70 %;
   – from 80 % to 100 %.

   The strip loads q'_k given in Paragraph 4 shall be increased by 30 %.

7. When designing provisional bridges and excavation covers which will also serve as work areas for excavators or lifting equipment, the following apply:

   a) The applicable loads are determined according to Paragraphs 3, 5 and 6.
   b) The contact areas of tracked equipment are taken from the manufacturer's specifications; R 3, Paragraph 2 (Section 2.5) applies for determination of the contact areas of rubber-tyred equipment.
   c) The dynamic coefficient is assumed to be $\varphi = 1.20$, independent of the span.
   d) For loads resulting from deceleration or acceleration and nosing forces, a horizontal point load of 1/7th of the vertical load given in Paragraph 5 shall be adopted at the governing location and in the governing direction at the height of the contact area. Additional investigations may be required for backhoe excavators.
   e) Further surcharges and exceptional loads are adopted according to DIN EN 1991-2 and DIN Technical Report 101.

3 Magnitude and distribution of earth pressure

3.1 Magnitude of earth pressure as a function of the selected construction method (R 8)

1. The magnitude of the earth pressure load is highly dependent on the amount of deflection and deformation of the retaining wall as a result of material excavation. The governing factors here are:

   – the flexibility of the support, see R 67 (Section 1.5);
   – the flexibility of the earth support, see R 14 (Section 5.3) and R 19 (Section 6.3);
   – the spacing of the support points and the flexural stiffness of the retaining wall.

   With regard to flexural stiffness, in-situ concrete walls, in particular diaphragm walls and pile walls, can generally be viewed as flexurally stiff and low-deformation walls, sheet pile and soldier pile walls as flexurally soft.

2. If a theoretical excavation case is considered in which any deflection or unloading of the ground is avoided when installing sheet pile walls or in-situ concrete walls, wall loading from at-rest earth pressure shall be taken into consideration. However, because it is not possible in practice to keep retaining walls completely free of deformation and deflection, the effective earth pressure load is generally smaller than the at-rest earth pressure load E_0.

3. For multiple-braced sheet pile walls with relatively small support point centres and slightly yielding supports, and for braced in-situ concrete walls in general, an earth pressure value shall be assumed which lies between the at-rest pressure and the active earth pressure, if the struts are prestressed with a force greater than 30 % of that projected for the fully excavated condition. This also applies to multiple-braced soldier pile walls, if the struts are prestressed with a characteristic force more than 60 % of that projected for the fully excavated condition.

4. If the struts are prestressed with forces smaller than those given in Paragraph 3, it may be assumed that the wall will be deformed or displaced by a value corresponding to 1 % of the wall height in medium-dense to dense, cohesionless soil or at least stiff, cohesive soil. This generally suffices to reduce the earth pressure from the at-rest earth pressure to the active earth pressure. This is generally the case for unsupported retaining walls with fixed-earth supports, regardless of the types of soil present.

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5. The magnitude of the anticipated earth pressure acting on anchored retaining walls is primarily dependent on the prestressing load of the anchors. Also see R 42 (Section 7.1).

6. See R 68 (Section 3.8) for earth pressure during retreating states.

3.2 Magnitude of total active earth pressure lead without surcharge loads (R 4)

1. The characteristic value of the active earth pressure load E_a from soil self-weight and, where applicable, cohesion, may be determined using planar slip surfaces based on classical earth pressure theory, where the limits given in DIN 4085 for wall inclination, ground inclination and earth pressure inclination are adhered to. Otherwise, rigid failure bodies with curved slip surfaces shall be used. This also applies to stratified soils.

2. The characteristic earth pressure inclination angle $\delta_{a,k}$ is dependent on R 89 (Section 2.3). It may be adopted for soldier pile walls, sheet pile walls and in-situ concrete walls with a positive angle if the resulting vertical forces can be completely transmitted into the ground. Otherwise a smaller, or negative, earth pressure inclination angle shall be introduced into the earth pressure analysis according to R 84 (Section 4.9). This may be necessary if large vertical forces are transmitted to the retaining wall, e.g. for provisional bridges or raked anchors.

3. For unsupported or yieldingly supported retaining walls, which rotate around the toe of the wall or a deeper point, the horizontal earth pressure load from soil self-weight and cohesion shall be determined in two alternative ways for homogeneous cohesive soils:

   a) Using the characteristic shear strengths according to R 2 (Section 2.2), whereby the computed tensile stresses resulting from cohesion as shown in Figure R 4-1 c) may not be taken into consideration.
   b) Using the equivalent friction angle $\varphi'_{Equiv,k} = 40°$ as shown in Figure R 4-1 e), where the ratio δ_k/φ'_k in accordance with R 89, Paragraph 5 (Section 2.3) is transferred to $\delta_{a,k}/\varphi'_{Equiv,k}$.

   The governing minimum earth pressure is the larger of the earth pressure loads.

   If the magnitude of the anticipated earth pressure is sufficiently well known from long-term measurements in similar conditions, and is checked in individual cases on the lining being installed, the equivalent friction angle may be increased to $\varphi'_{Equiv,k} = 45°$.

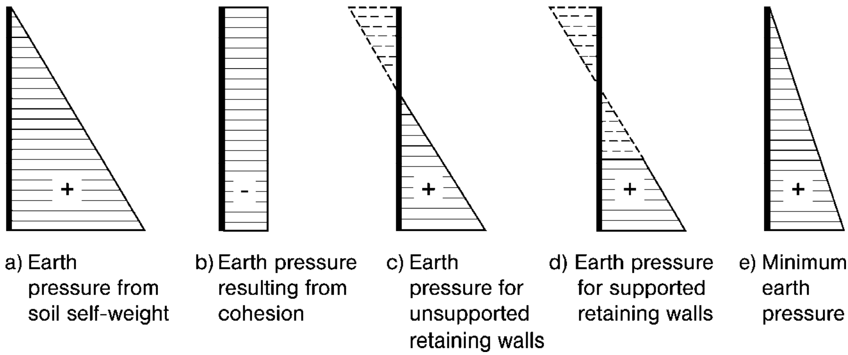


Figure R 4-1. Determination of active earth pressure load for homogeneous cohesive soil

4. For stratified soil the procedure is as follows:
   a) The earth pressure ordinates of the cohesionless strata are always determined with the characteristic shear strengths according to R 2 (Section 2.2). They are the governing values for determining the earth pressure of the stratum in question.
   b) The earth pressure ordinates of the cohesive strata are determined according to the instructions in Paragraph 3 using both the characteristic shear strengths according to R 2 (Section 2.2), as shown in Figure R 4-2 b), and the equivalent friction angle $\varphi'_{Equiv,k}$ as shown in Figure R 4-2 c).

   The governing minimum earth pressure is the larger of the earth pressure loads of the respective stratum. The total load is obtained by adding the governing earth pressure loads of the individual strata.

5. For slightly yieldingly supported retaining walls, where earth pressure redistribution is anticipated due to the prevailing conditions, the computed tensile stresses resulting from the characteristic shear strengths according to R 2 (Section 2.2) due to cohesion may be taken completely into consideration and balanced against any compressive stresses. This results in the following.
   a) In homogeneous, cohesive soil the earth pressure is determined as shown in Figure R 4-1 d) from:

      $E_{ah} = E_{agh} + E_{ach}$

   In addition, the earth pressure load shall be determined with the equivalent friction angle according to Paragraph 3 b). The larger value is the governing minimum earth pressure.

b) In stratified soil the earth pressure load is determined from both the earth pressure ordinates as shown in Figure R 4-2 b) and from the earth pressure ordinates as shown in Figure R 4-2 c). The governing earth pressure is the larger of the earth pressure loads of the respective stratum. The total load corresponds to the minimum earth pressure as shown in Figure R 4-2 d).

6. The vertical earth pressure component is determined from the horizontal component and the inclination angle selected in accordance with Paragraph 2, in conjunction with the characteristic friction angle φ'_k. Even if a minimum pressure in accordance with Paragraph 3b) becomes the governing pressure for the horizontal component, analysis of the vertical component is still based on the characteristic friction angle φ'_k.

7. In cohesive and rocky ground, local experience should be scrutinised for indications that the earth pressure may increase with time due to the swelling capacity of the ground, by frost action, by thawing after a period of frost or for other reasons, above that determined for the respective soil properties. In addition, where rocky ground is involved, it should be established whether bedding planes or joints predetermine certain slip surfaces, which influence the magnitude of the earth pressure load. Also see R 38 (Section 11.1).

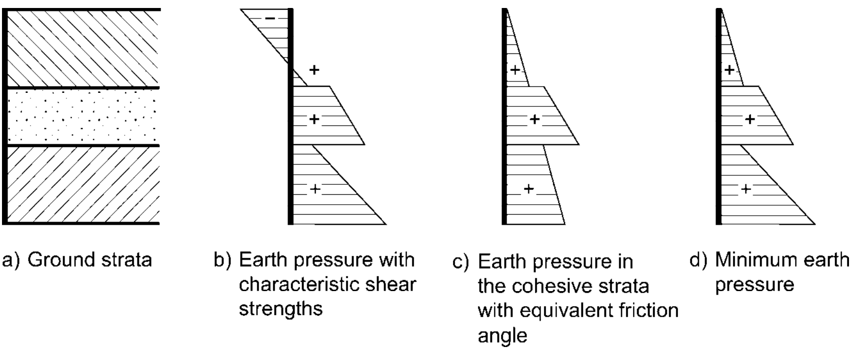


Figure R 4-2. Determination of active earth pressure for partially cohesive soils

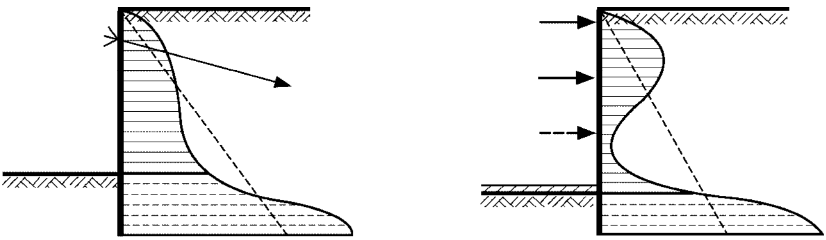
8. Based on theoretical considerations, a larger earth pressure load than computed using classical earth pressure theory is anticipated for rotation around the wall top or a higher point. Despite this, it is not necessary, in fitting with measurements on previously executed excavations, to increase the earth pressure load determined on the basis of Paragraphs 1 to 4.

9. By applying model tests and taking measurements on previously executed excavations (see [69] and [73]) it has been demonstrated that under certain circumstances a portion of the earth pressure from soil self-weight can be redistributed to below the excavation level when using flexible retaining walls, with the result that the effective earth pressure load acting above the excavation level is smaller than the mathematical active earth pressure load $E_{a,k}$. This can be the case for example:

   a) for a yieldingly anchored, flexible retaining wall (Figure R 4-3 a);
   b) when removing the lowest row of struts on a flexible, multiple-braced wall (Figure R 4-3 b).

   However, a corresponding earth pressure reduction may only be adopted at a maximum 20 % for stability analysis of soldier pile walls or 10 % for sheet pile walls, and only when confirmed by measurements in comparable conditions, or if these approaches have been checked against measurements on previously installed bracing.

   See R 68 (Section 3.8) for earth pressure reductions in retreating states. For soldier pile walls, the effects on the analysis of the equilibrium of vertical forces below the excavation level in accordance with R 15 (Section 9.5) shall also be examined.



a) Flexibly anchored soldier pile wall

b) Sheet pile wall strut removal

Figure R 4-3. Possible earth pressure redistribution in the region below the excavation level for flexible retaining walls

The stresses on the earth support increase due to earth pressure redistribution in the zone below the excavation level, and thus also the support force B_h. Generally, this shall be taken into consideration when analysing the embedment depth. For soldier pile walls, the effects of this on the analysis of the horizontal forces below the excavation level shall also be examined.

3.3 Distribution of active earth pressure without surcharges (R 5)

1. Unsupported retaining walls restrained in the soil, or yieldingly supported walls, rotate around a point at depth. Accordingly, classical earth pressure distribution shall be anticipated in such cases. See also R 4, Paragraphs 3 and 4 (Section 3.2) for cohesive soil.

2. Slightly yieldingly supported retaining walls rotate around higher, alternating pivots, associated with parallel deflection and bending, during excavation. Earth pressure distribution varies based on the precise interaction of these influences. Influencing factors include:

   - the type of retaining wall and the method of installation and/or infilling;
   - the flexural stiffness of the retaining wall;
   - the number and configuration of the struts and/or anchors;
   - the size of the respective excavation stage before installation of the struts and/or anchors;
   - the prestressing of the struts and/or anchors.

   Furthermore:

   - the site morphology and;
   - the type and stratification of the ground;

   may play a role.

   In contrast to classical earth pressure distribution, the earth pressure is generally concentrated at the wall supports. The regions between the support points are unloaded if the wall bends correspondingly. The previously recorded deformation at each respective construction stage is governing here (see [5, 6, 32]). Redistribution is generally smaller for flexible supports. In some circumstances no earth pressure redistribution takes place.

3. For braced retaining walls in cohesionless soils and non-yielding supports according to R 67, Paragraph 3 (Section 1.5), the following rules can be assumed in principle, based on theoretical considerations and available measurements (see [3–9, 11–14, 32, 46, 52, 67, 73, 89, 90]):

   a) Earth pressure distribution always commences at ground level with the ordinate at zero and then increases much faster with depth than when based on classical earth pressure theory.
   b) Due to the sequence of excavation and installation of infilling it may generally be assumed that the effective earth pressure ends at the excavation level at the zero ordinate for soldier pile walls. For supported soldier pile walls the earth pressure redistribution is therefore generally restricted to the height H from ground level to the excavation level. However, see also R 15 (Section 5.5).

c) For sheet pile walls, diaphragm walls and pile walls the wall height H′, over which the upward earth pressure redistribution is anticipated, is a function of the stiffness of the wall and the deflection of the wall toe. It is also a function of any structural measures that may also promote upward earth pressure redistribution, in particular slight prestressing of struts. The redistribution zone can be selected if the corresponding pressure diagram is compatible with the wall deformations and the deflections at the wall toe. It is generally acceptable for earth pressure redistribution to be assumed for the height H from ground level to the excavation level, if there is no reason to anticipate an especially large earth pressure redistribution from the zone below the excavation level.
d) The largest load ordinate can be found in the earth pressure redistribution zone at the height of the support in single-propped walls, if this is installed sufficiently low. In double-propped walls it is at the height of the upper support, if this is installed very low; it is at the height of the lower support, on the other hand, if the upper support is installed near ground level. In multiple-propped walls it is generally located at a support level within the central third of the excavation depth.
e) For supported soldier pile walls the earth pressure resultant from soil self-weight and unbounded distributed loads is almost always higher than half of the excavation depth in the earth pressure redistribution zone. The resultant of the redistributed earth pressure load for sheet pile walls, diaphragm walls and pile walls is generally below half of the distance from ground level to the selected end of the earth pressure redistribution.
f) This applies to medium-dense to dense soils. Loose cohesionless soil is also subject to earth pressure redistribution, although only to a minor extent. The earth pressure resultant for sheet pile walls and in-situ concrete walls is lower than that for soldier pile walls, all else being equal.

4. Paragraph 3 applies accordingly for braced retaining walls in cohesive soils (see [10, 15, 16, 47, 90]). However, considering the influence of soil consistency, the following shall be observed:

   a) In semi-solid to stiff cohesive soils, earth pressure redistribution similar to that in medium-dense to dense, cohesionless soils can be assumed. Nevertheless, in the case of stiff cohesive soils the preconditions for applying Recommendations R 38 to R 41 (Sections 11.1 to 11.4) should be examined.
   b) In individual cases in stiff cohesive soils, the earth pressure distribution may either resemble more that of a medium-dense or a loose cohesionless soil. The clay content and sensitivity are governing in this respect.
   c) In soft cohesive soils earth pressure redistribution is at most equal to that of loose cohesionless soil, but is often either lower or does not occur at all. See Section 12.

This information applies with reservations only for:

– soils, the behaviour of which can be impaired by hair cracks, slickensides, joints or intercalations of slightly cohesive or cohesionless soils;
– soils in which certain slip surfaces, which may lead to sliding, may be predetermined by faulting and inclined bedding planes, e.g. Opalinus Clay, Nodular Marl and Tarras.

Assessment of these soils requires geotechnical expertise and experience in the field.

5. Paragraphs 3 and 4 apply without restriction for anchored retaining walls, if the anchors are prestressed so that wall deflection is similar to that for bracing. However, as this is generally not the case, and because correspondingly larger or smaller prestressing imposes different earth pressure distributions and, furthermore, because the ground not only acts as a load but also accepts anchor forces, different regulations and additional requirements may also apply to anchored retaining walls. See Section 7.

6. Because of the numerous possible impacts, the actual earth pressure distribution can only be approximately determined. Determination of the embedment depth and action effects should therefore be based on as simple a pressure diagram as possible, bounded by straight lines, e.g. one of the pressure diagrams shown in Figure R 5-1. To simplify analysis the bending points and load increments of the selected pressure diagrams may be located at the support points. If the preconditions given there are fulfilled, the

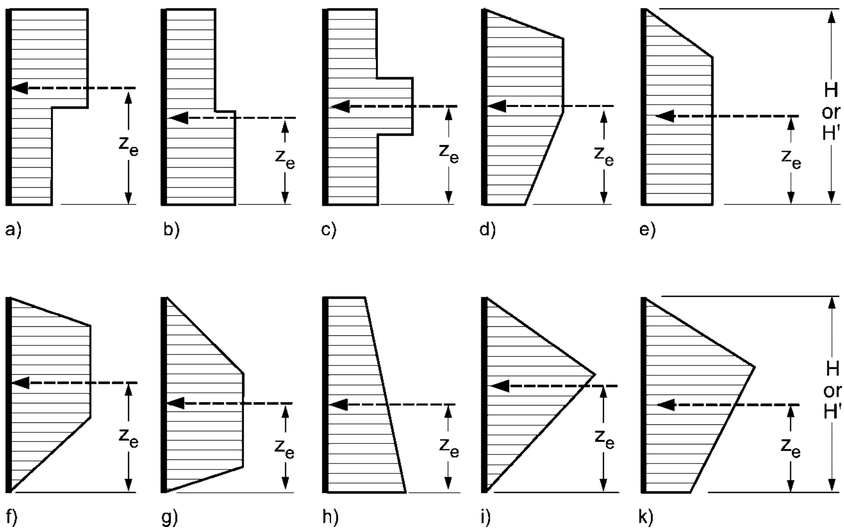


Figure R 5-1. Pressure diagrams for supported retaining walls (examples) pressure diagrams may be adopted according to R 69 (Section 5.2) or R 70 (Section 6.2).

7. If the anticipated earth pressure distribution cannot be estimated with sufficient precision due to unusual circumstances, e.g. layers of soft ground, organic ground or the simultaneous use of struts and anchors, the selected approaches shall be checked by measurements on the lining based on the observational method described in DIN 1054, in order to allow initiation of special structural measures before a critical stage is reached. If this is not possible it may be necessary to perform the analysis using two pressure diagrams, which restrict the possible earth pressure distributions. The most unfavourable action effects are always critical for designing individual components.

3.4 Magnitude of total active earth pressure lead from live loads (R 6)

1. Determination of the earth pressure load E_a from vertical, variable loads may generally be based on the same earth pressure inclination angle $\delta_{a,k}$ as for determination of the earth pressure from soil self-weight. Also see R 4, Paragraph 2 (Section 3.2).
2. The magnitude of the earth pressure load from unbounded, vertical, distributed loads p_k according to R 55 to R 57 (Sections 2.6 to 2.8) or q_k according to R 7 (Section 3.5) may generally be determined using the same slip surface as for the earth pressure load from soil self-weight.
3. The slip surfaces shown in Figure R 6-1 a), originating at the rear edge of the load area or at the line load, and running parallel to the slip surface at

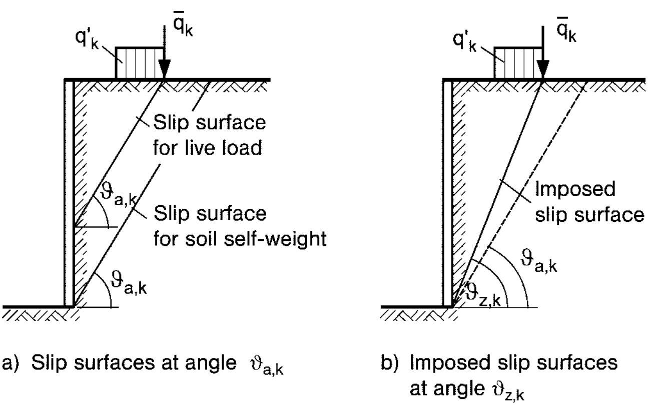


Figure R 6-1. Assumed slip surfaces for determination of the total active earth pressure from soil self-weight and live loads an angle $\vartheta_{a,k}$, and which are critical for determination of the earth pressure load from soil self-weight, may be used to approximately determine the earth pressure load from vertical line or strip loads according to R 3 (Section 2.5) or R 55 to R 57 (Sections 2.6 to 2.8). However, also see Paragraph 5.

4. Even if the earth pressure load from soil self-weight and cohesion according to R 4, Paragraph 3 b) (Section 3.2) is determined for cohesive strata with the aid of an equivalent friction angle, the earth pressure from unbounded, vertical, distributed loads, including those up to $p_k = 10 \text{ kN/m}^2$, and those from line and strip loads, are always determined in accordance with Paragraph 3 a) using the characteristic friction angle φ'_k. In exceptional cases with a cohesion value $c'_k > 30 \text{ kN/m}^2$, the thus determined earth pressures from surcharges may be suitably balanced against numerical tensile stresses from soil self-weight and cohesion, if more precise investigations have been performed and sufficient local experience is available.

5. To determine the active earth pressure from line or strip loads for unsupported retaining walls with a fixed-earth support only, or for yieldingly supported retaining walls, an imposed slip surface shall also be investigated. It runs from the line load or from the rear edge of the strip load to the intersection with the rear of the wall and the excavation level for soldier pile walls, or to the actual or theoretical wall toe (Figure R 6-1 b). The combined earth pressure from soil self-weight and live loads determined in this way governs further analysis if it is greater than that determined using the slip surface angle $\vartheta_{a,k}$. The effective proportion of the earth pressure from live loads is then given by the difference between the determined total load and the active earth pressure from soil self-weight and, where applicable, cohesion for the slip surface angle $\vartheta_{a,k}$. Splitting in the ratio of the loads involved is possible, but not expedient. A numerical determination of the effective proportion of the earth pressure from live loads is unnecessary if the action effects from changeable actions according to R 82, Paragraphs 3 to 5 (Section 4.4) are determined as the difference between the action effects for permanent and changeable actions on the one hand and the action effects for permanent actions on the other.

6. For slightly yielding walls in particular, the earth pressure load E_{aHh} from horizontal line or strip loads H is adopted at:

   $E_{aHh} = H$.

   For unsupported or yieldingly supported walls the earth pressure load E_{aHh} may also be determined using the approach given in DIN 4085. That is:

$$E_{aHh} = H \cdot \frac{\cos(\vartheta_a - \varphi_k) \cdot \cos\delta_{a,k}}{\cos(\vartheta_a - \varphi_k - \delta_{a,k})}.$$

   Depending on the situation $\vartheta_{a,k}$ or $\vartheta_{z,k}$ is adopted for the slip surface angle ϑ_a.

7. See R 71 (Section 3.6) for determining the earth pressure with surcharges under various boundary conditions.
8. See R 21 to R 23 and R 28 to R 29 (Sections 9.2 to 9.6) for determining earth pressure from building loads.

3.5 Distribution of active earth pressure from live loads (R 7)

1. When determining the earth pressure from an unbounded, vertical, distributed load the following shall be differentiated:
   - a load component $p_k \leq 10\ kN/m^2$, which is allocated to the permanent actions and;
   - if applicable, a load component q_k, in excess of $p_k = 10\ kN/m^2$ and which is allocated to the changeable actions.

   The following applies for the earth pressure distribution:

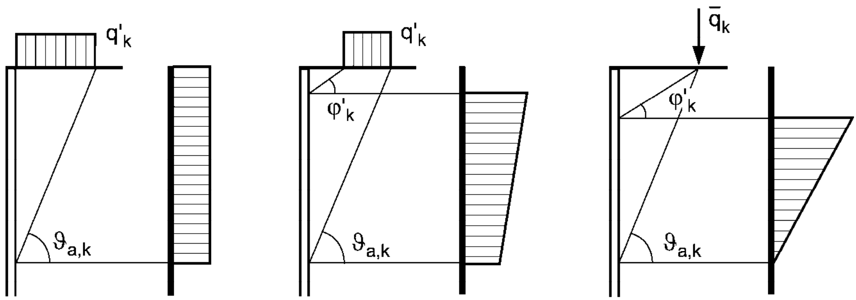
   a) For unsupported or yieldingly supported walls, the earth pressure from an unbounded distributed load is adopted as a rectangle over the whole wall height, based on classical earth pressure theory. This applies equally for a permanent action $p_k \leq 10\ kN/m^2$ and for any changeable action q_k, if applicable.
   b) For slightly yielding walls the earth pressure resulting from an unbounded distributed load $p_k \leq 10\ kN/m^2$ is incorporated in the pressure diagram according to R 5, Paragraph 6 (Section 3.3). The earth pressure resulting from the changeable action q_k is adopted as a rectangle over the wall height based on classical earth pressure theory.
2. The earth pressure from vertical strip loads q_k' or from line loads $\overline{q}_k$ may be adopted as a simple pressure diagram, bounded at the top and bottom as follows:

   a) According to classical earth pressure theory, the pressure diagram for unsupported or yieldingly supported retaining walls begins at the height at which a straight line at an angle φ_k' to the horizontal, originating at the front edge of the strip load or at the line load, intersects the rear of the wall. For slightly yielding retaining walls the pressure diagram may be adopted starting at ground level.
   b) The pressure diagram generally ends at the height at which a straight line at an angle $\vartheta_{a,k}$ to the horizontal, originating at the rear edge of the strip load or at the line load, intersects the rear of the wall. When the earth pressure is determined using imposed slip surfaces according to R 6, Paragraph 5 (Section 3.4), the pressure diagram ends at the intersection of the imposed slip surface with the rear of the wall.

3. The shape of the pressure diagram can be specified as follows for unsupported or flexibly supported retaining walls:

   a) In the case of strip loads adjoining the wall, a rectangular pressure diagram based on classical earth pressure theory results as shown in Figure R 7-1 a).
   b) In the case of vertical line loads and classical earth pressure theory, an earth pressure distribution results which can be substituted, as a conservative approximation, by a triangular pressure diagram based on Figure R 7-1 c).
   c) The earth pressure distribution for vertical strip loads not adjoining the wall shall be determined using an appropriate approximation method investigation. Using a straight-line interpolation as a function of the distance-to-width ratio of the load, the result is a trapezoidal pressure diagram based on Figure R 7-1 b).

   Generally $\vartheta = \vartheta_{a,k}$ is adopted, or $\vartheta = \vartheta_{z,k}$ for imposed slip surfaces (Figure R 6-1).

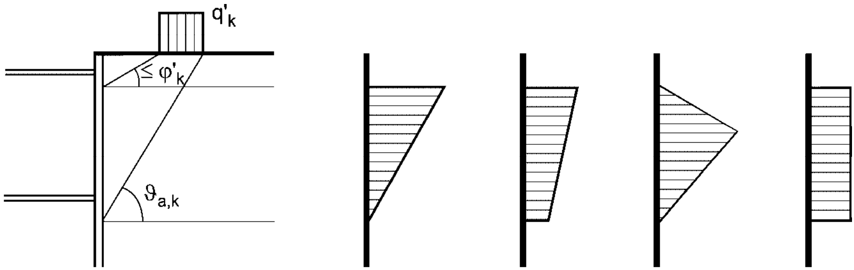


a) Strip load adjoining the wall b) Strip load at a distance from the wall c) Line load

Figure R 7-1. Pressure diagrams for the earth pressure from vertical live loads for unsupported or flexibly supported walls

4. For moderately flexibly supported retaining walls the shape of the pressure diagram as shown in Figure R 7-2 b) may generally be freely selected. Adjustment of the start and end pressure diagram to the support points is also permissible; however, the resultant may not be below the point at which a straight line originating at the rear edge of the strip load or at the line load and running at an angle of 45° from the horizontal, meets the rear of the wall.

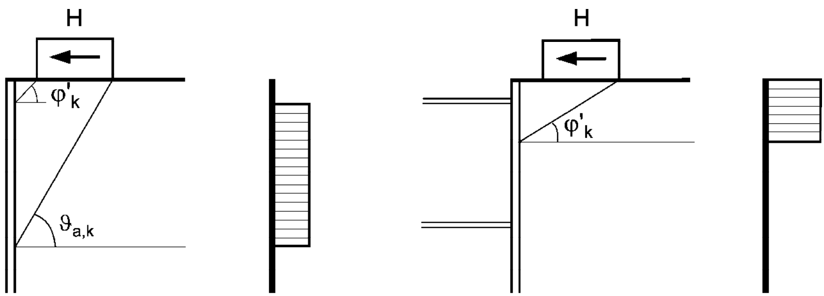
5. In principle, the earth pressure distribution from horizontal line and strip loads may be adopted in the same manner as for the corresponding vertical load. This produces the pressure diagrams shown in Figure R 7-3 for a bounded strip load. The procedure for imposed slip planes is analogous to vertical surcharges.



a) Retaining wall, live load and load distribution

b) Examples of simple pressure diagrams

Figure R 7-2. Pressure diagrams for the earth pressure from vertical live loads for moderately flexibly supported walls



a) Unsupported or flexibly supported wall

b) Moderately flexibly supported wall

Figure R 7-3. Pressure diagrams for the earth pressure from horizontal live loads

6. See R 71 (Section 3.6) for determining the earth pressure with surcharges under various boundary conditions.
7. See R 28 and R 29 (Sections 9.3 and 9.4) for the distribution of earth pressure from building loads.

3.6 Superimposing earth pressure components with surcharges (R 71)

1. For slightly yieldingly supported retaining walls the magnitude and distribution of the earth pressure from soil self-weight, unbounded distributed loads p_k and, where applicable, cohesion on the one hand, and locally acting strip loads q'_k or line loads $\overline{q}_k$ on the other, may be determined separately and used to determine the action effects. In contrast, for unsupported retaining walls with a fixed-earth support, and for yieldingly supported retaining walls, these two components may be subject to mutual influence. Here, the principal differentiation is between:

   a) earth pressure determination using slip surfaces at an angle $\vartheta_{a,k}$ as shown in Figure R 6-1 a) (Section 3.4);
   b) earth pressure determination with imposed slip surfaces at an angle $\vartheta_{z,k}$ as shown in Figure R 6-l b) (Section 3.4).

   Below, the cases that can occur for unsupported or yieldingly supported retaining walls in homogeneous ground are described.

2. The following pressure diagrams result for homogeneous, cohesionless soils, taking R 7, Paragraph 1 (Section 3.5) into consideration:

   a) the earth pressure components and pressure diagrams as shown in Figure R 71-1, adopting slip surfaces at an angle $\vartheta_{a,k}$;
   b) the earth pressure components and pressure diagrams as shown in Figure R 71-2, adopting slip surfaces at an angle $\vartheta_{z,k}$.

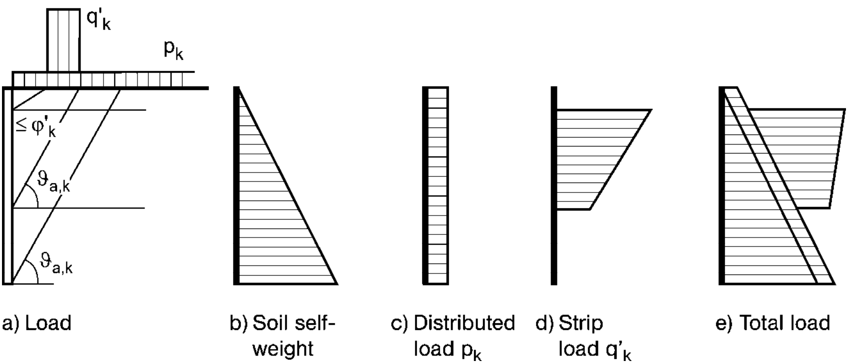


Figure R 71-1. Earth pressure distribution for an unsupported retaining wall, with a fixed earth support in cohesionless soil, assuming slip surfaces at an angle $\vartheta_{a,k}$ (example)

3. The following pressure diagrams result for homogeneous cohesive soils, taking R 4, Paragraph 3 (Section 3.2) and R 7, Paragraph 1 (Section 3.5) into consideration:

   a) the earth pressure components and pressure diagrams shown in Figure R 71-3 with shear strength according to R 2 (Section 2.2), assuming slip surfaces at an angle $\vartheta_{a,k}$;

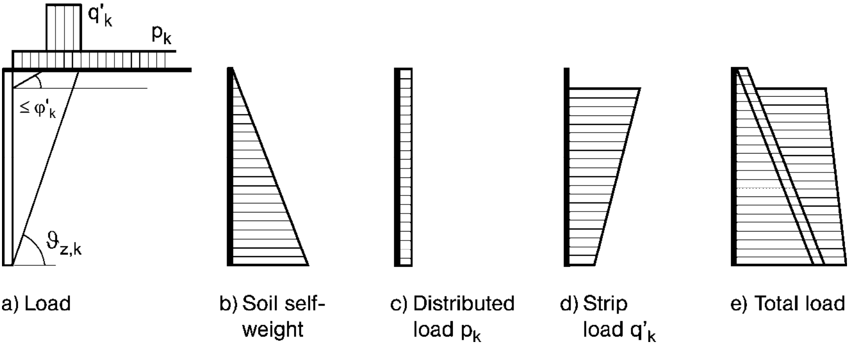


Figure R 71-2. Earth pressure distribution for an unsupported retaining wall, with a fixed earth support in cohesionless soil, assuming slip surfaces at an angle $\vartheta_{z,k}$ (example)

   b) the earth pressure components and pressure diagrams shown in Figure R 71-4 with shear strength according to R 2 (Section 2.2), assuming slip surfaces at an angle $\vartheta_{z,k}$;
   c) the earth pressure components and pressure diagrams shown in Figure R 71-5, assuming an equivalent friction angle according to R 4, Paragraph 3 b) (Section 3.2).

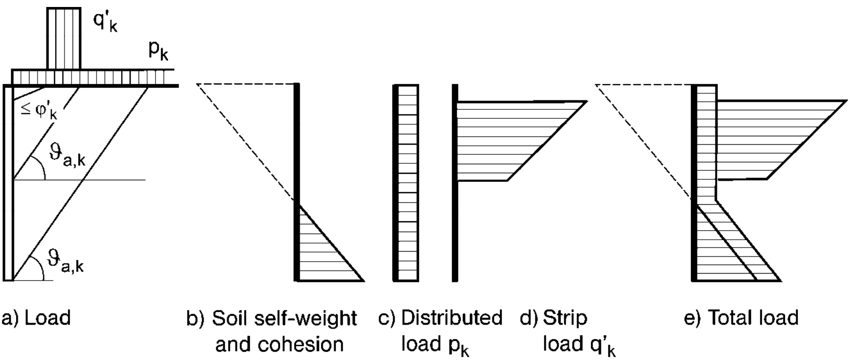


Figure R 71-3. Earth pressure distribution for an unsupported retaining wall, with a fixed earth support in cohesive soil, assuming slip surfaces at an angle $\vartheta_{a,k}$ (example)

4. If the most critical load approach cannot be established, all possible pressure diagrams shall be determined for individual cases, together with the corresponding action effects and embedment depths. The design should be based on the case with the largest bending moment and the largest embedment depth, even if these were not determined using the same approach.

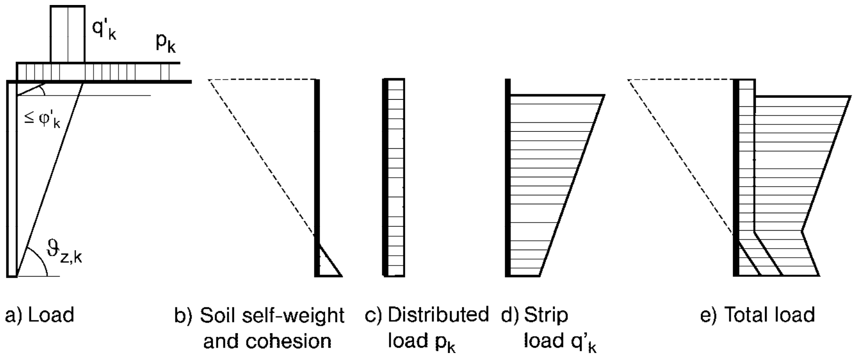


Figure R 71-4. Earth pressure distribution for an unsupported retaining wall, with a fixed earth support in cohesive soil, assuming slip surfaces at an angle $\vartheta_{z,k}$ (example)

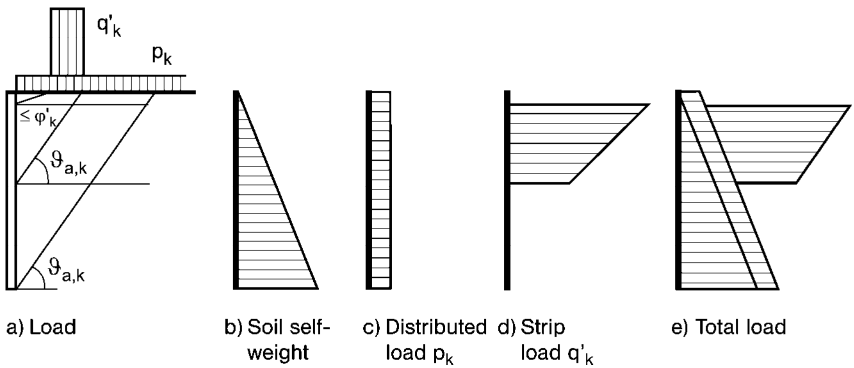


Figure R 71-5. Earth pressure distribution for an unsupported retaining wall, with a fixed earth support in cohesive soil, assuming a minimum earth pressure (example)

3.7 Determination of at-rest earth pressure (R 18)

1. The at-rest earth pressure represents a component for determination of the increased active earth pressure according to R 22 (Section 9.5). The following information on determination of at-rest earth pressure therefore serves

primarily to determine this component calculation value. Only in exceptional cases may it be expedient to base the design of the excavation structure on the actual at-rest earth pressure according to R 23 (Section 9.6).

2. Because the at-rest earth pressure does not describe a limit state in the sense of the partial safety factor approach, but merely occurs as an external action in the design of structural components, all structural analyses are based on the characteristic at-rest earth pressure $E_{0,k}$. Here, the friction angle φ'_k merely represents a control variable. The following cases are differentiated:

   – at-rest earth pressure from soil self-weight;
   – at-rest earth pressure from unbounded distributed loads;
   – at-rest earth pressure from vertical or horizontal building loads.

3. The magnitude of the characteristic at-rest earth pressure from soil self-weight can only be determined approximately. The following approaches are precise enough for determining the at-rest earth pressure coefficient K_0 for practical purposes:

   a) For horizontal ground the at-rest earth pressure coefficient may be determined using the
   $K_0 = K_{0h} = 1 - \sin \varphi'_k$
   approach.
   b) The at-rest earth pressure may be determined for a ground surface climbing at an angle $\beta = \varphi'_k$ using the
   $K_0 = \cos \varphi'_k$ or
   $K_{0h} = \cos^2 \varphi'_k$
   approaches and be linearly interpolated in approximation for $0 < \beta < \varphi'_k$ [40].
   The at-rest earth pressure is always assumed to act parallel to the ground surface.
   c) Generally speaking, these approaches may also be adopted for overconsolidated soils. Only in exceptional cases may it be expedient to increase the at-rest earth pressure coefficients determined according to Paragraph a) or Paragraph b) by the factor:

   $$f_{\ddot{U}} = \sqrt{\frac{\sigma_{v\ddot{u}}}{\sigma_v}} \; .$$

   Where:

   σ_{vo} the vertical stress from a previous surcharge;
   σ_v the current vertical stress.

   In the limit case the increased at-rest earth pressure may not exceed the passive earth pressure.

d) The approaches mentioned also apply in approximation to cohesive soils. Cohesion is thus not taken into consideration.

4. The characteristic at-rest earth pressure from an unbounded distributed load may be approximately determined using

   $e_{0h,k} = K_{0h} \cdot p_k$

   and be assumed to act horizontally, independent of ground inclination. The ordinate remains the same over the complete height of the wall.

5. The characteristic at-rest earth pressure from vertical or horizontal building loads may generally be determined and adopted according to elastic half-space theory. Generally, for the concentration factor after *Fröhlich*:

   $\nu = 4$ for normally consolidated soils;
   $\nu = 3$ for overconsolidated soils.

   Stiff and very stiff cohesive soils are generally regarded as overconsolidated.

   In the $\nu = 4$ case, the characteristic horizontal at-rest earth pressure $E_{OBh,k}$ may be assumed to be approximately 25 %, and in the $\nu = 3$ case 30 % of the total vertical load. The vertical component $E_{0Bv,k}$ of the at-rest earth pressure is introduced in both cases as 50 % of the total vertical load, if it has not been precisely determined, e.g. according to [41] or [46].

6. In principle, the earth pressure load $E_{0Bh,k}$ from building loads shall be divided into a permanent component $E_{0Bgh,k}$ from building self-weight and a variable component $E_{0Bqh,k}$ from building live loads. With regard to determination of the magnitude and distribution of the earth pressure from the variable component of the action, the same rules apply as for the permanent component of the actions according to Paragraph 2. According to R 4, Paragraph 5 (Section 4.12), however, it is generally permissible to increase the building live load by the factor f_q and then to treat it as a permanent action together with the building self-weight.

3.8 Earth pressure in retreating states (R 68)

1. Retreating conditions arise in supported retaining wall systems when, after manufacturing parts of the building and/or after partial backfilling of the excavation or the work space, a row of struts is removed or a set of anchors is unloaded.

2. If no considerable deflections or deformations of the retaining wall are anticipated during removal of struts and/or unloading of anchors, the earth pressure diagram selected for the largest excavation depth must also be maintained in the retreating state.

3. If a deflection of more than 0.2 ‰ is associated with the new span when removing struts or unloading anchors in dense, cohesionless soil, or at least plastic, cohesive soil, earth pressure redistribution over the remaining excavation depth shall be anticipated corresponding to the new support conditions. The earth pressure in the region of the removed supports is reduced; it is partially redistributed to the supports above and partially to those below. With a more precise definition of the pressure diagram based on [89] and [90], substantially more favourable design action effects can result as a function of additional deflection than in a case where the pressure diagram from the previous construction stage is retained or a new pressure diagram with the same earth pressure load is selected.
4. If the span increases by at least 30 %, or the additional deflection is shown to be larger than 0.2 ‰ of the new span, triple- or multiple-propped soldier pile walls and sheet pile walls may be analysed using the following load approaches:

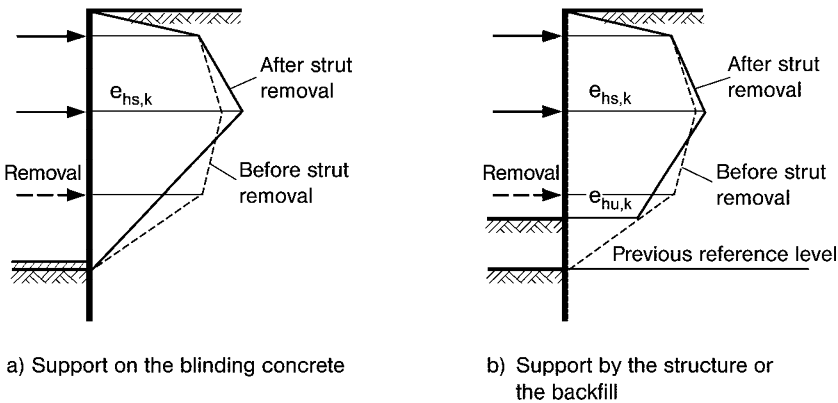


Figure R 68-1. Pressure diagrams for soldier pile walls in retreating states

a) If, after removal, the lowest set of struts or anchors is replaced by a support on the blinding concrete, the load ordinate $e_{hs,k}$ at the height of the new lowest support shall be increased by 15 % as shown in Figure R 68-1 a) and allowed to decrease to zero at the excavation level.
b) If, after removal, the lowest set of struts or anchors is replaced by a support on part of the structure or on the backfill, the load ordinate $e_{hs,k}$ at the height of the new lowest support shall be increased by 5 % as shown in Figure R 68-1 b) and allowed to decrease to $e_{hu,k} = \frac{1}{2} \cdot e_{hs,k}$ at the level of the top of the backfill.

If only one row of struts or anchors is present in the retreating state, then the pressure diagram shall be selected on the basis of the regulations for single-propped soldier pile walls, if more precise stipulations are not made in Paragraph 3.

5. The wall deformations associated with the removal of the highest row of struts and/or unloading of the highest row of anchors are usually sufficient to reduce the upwardly redistributed earth pressure to the classical active earth pressure.

4 General stipulations for analysis

4.1 Stability analysis (R 81)

For stability analyses in limit states STR and GEO 2 according to R 78 (Section 1.4), the following procedure is employed for analysis method 2 according to R 77, Paragraph 7 (Section 1.3), if no other procedure is expedient in individual cases:

1. The excavation structure is designed, the dimensions selected and the structural system defined.
2. The characteristic or representative values of the actions are identified, e.g. the loads imposed by self-weight, active earth pressure, increased active earth pressure, surcharges and, if applicable, the characteristic deformations. See R 63 (Section 10.6) for how to deal with water pressure.
3. The characteristic or representative stresses E_k or E_{rep} are determined on the specified system as action effects, e.g. shear forces, support forces, ground reactions and bending moments. This applies to all sections through the structure and in those soil-structure interfaces that govern design.
4. The design values of the effects are determined for each governing section through the structure and in the soil-structure interfaces. They are obtained from:

   $E_d = E_{G,d} + E_{Q,d}$

   where

   $E_{G,d} = E_{G,k} \cdot \gamma_G$ and $E_{Q,d} = E_{Q,k} \cdot \gamma_Q$ or $E_{Q,d} = E_{Qrep} \cdot \gamma_Q$

   by multiplying the characteristic or representative action effects E_k or E_{rep} by the partial safety factors γ_G or γ_Q.
5. The characteristic resistances $R_{k,i}$ are determined. Here, the resistances of the structural elements and the resistances of the ground are differentiated:
   a) For example, resistances of the structural elements include: resistances against compressive forces, tensile forces, shear forces and bending moments, generally determined from the characteristic material parameters and the material cross-section.
   b) For example, resistances of the ground include passive earth pressure, base resistance and skin resistance of soldier piles, sheet pile walls and in-situ concrete walls, pull-out resistance of grouted anchors, soil nails and tension piles, each determined by means of either analysis, load tests or based on empirical data.

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The resistance design values are obtained using

$R_{d,i} = R_{k,i}/\gamma_R$

by dividing the characteristic resistances $R_{k,i}$ by the partial safety factors γ_R for the respective material, e.g. steel, reinforced concrete, wood or soil.

6. Using the thus determined design values for effects and resistances, adherence to the limit state condition

   $\Sigma E_{d,i} \leq \Sigma R_{d,i}$

   is analysed for every possible section and, where applicable, for every governing action combination.

7. The following procedure may be adopted for non-linear systems or when using numerical methods to determine the characteristic or representative effects from variable actions for the respective investigated load combinations:

   – Determine the total effects $E_{k,i}$ resulting from the characteristic or representative, permanent and variable actions;
   – Determine the effect $E_{Gk,i}$ resulting from the characteristic, permanent actions;
   – Determine the effect $E_{Qrep,i}$ resulting from the representative, variable actions using the approach

   $E_{Qrep,i} = E_{k,i} - E_{Gk,i}$

8. In contrast to Paragraph 7, all variable actions above an unbounded distributed load $p_k = 10 \text{ kN/m}^2$ may be multiplied by the factor $f_q = \gamma_Q/\gamma_G$. This also applies to non-linear structural systems and for numerical methods.

   This procedure replaces the subdivision of the characteristic effects for the respective investigated load combination into permanent and variable actions. To determine the design effect, the characteristic total effect needs only be multiplied by the partial safety factor γ_G.

9. If a governing limit state condition is not satisfied for the investigated section, the dimensions shall be increased appropriately. If excess safety needs to be reduced to satisfy economical considerations, the dimensions may be reduced appropriately. Analysis is repeated in both cases or completed by iteration.

10. According to R 83 (Section 4.11), serviceability can be examined or analysed using the determined deformations, together with the characteristic action effects.

11. Details are given in further recommendations.

4.2 General information on analysis methods (R 11)

1. All advancing and retreating states for excavating and refilling shall be investigated. Excavating states refer to all construction stages until reaching the final excavation level; retreating states refers to all construction stages during backfilling of the excavation and during removal or repositioning of struts, or when unloading anchors.

2. If only the stability analysis is governing, the following simplified approaches may be adopted to analyse embedment depth and to determine the action effects:

   a) The structural system may be based on a beam on inflexible supports.
   b) The deformations in the various construction stages and the effects on subsequent construction stages need not generally be investigated. The advancing states and the fully excavated state may therefore be analysed assuming that they were not preceded by any other construction stage.
   c) In conjunction with a free earth support, a partial restraint or a restraint after *Blum*, the base of the earth support may be assumed to be immovable [168].
   d) For a free earth support the ground reactions actually distributed over the embedment depth in the embedment zone of the wall may be substituted by a fixed support at the height of the resultant regardless of the number of supports, if the following points are adhered to.

3. By replacing the ground reactions by a fixed support at the height of the resultant, erroneous bending moments and incorrect deflections are necessarily obtained (see Figure R 11-1). They should be dealt with as follows:

   a) An incorrect cantilever moment occurs at the height of the assumed support. It may be disregarded for design and reinforcement. In particular, this cantilever moment should not lead to the reinforcement of diaphragm walls being located on the incorrect side.
   b) A backward rotating deflection incorrectly occurs at the wall toe. The deflection curve may be corrected for the region between the excavation level and the wall toe such that it ends at the wall toe with a deflection $s = 0$.

   If additional loads act below this assumed support, in particular a substantial earth pressure, building loads or positive water pressure, the resulting errors are generally no longer acceptable.

4. If a serviceability analysis is necessary or if a realistic, economical design is aimed for, it is generally necessary to adhere to all or at least to some of the following requirements:

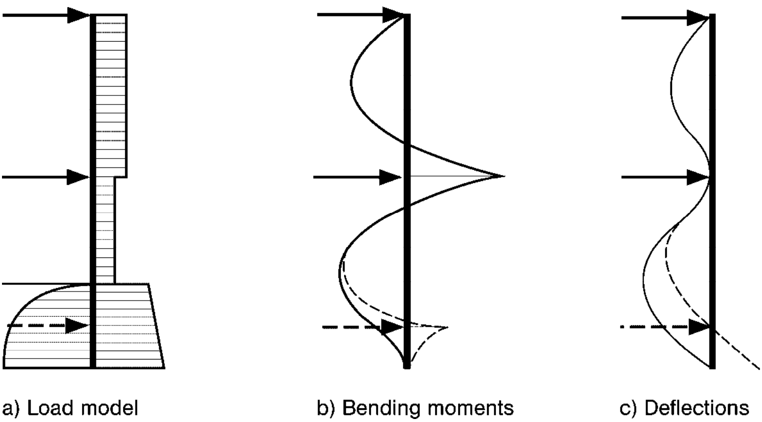

Figure R 11-1. Impact of replacing the ground reaction distributed over the embedment depth by a fixed support

a) The structural system shall be based on a beam on elastic supports.
b) The deformations occurring before installation of the respective subsequent support and their impact on the respective subsequent construction stage shall be taken into consideration.
c) The ground reactions may not be replaced by their resultant according to Paragraph 2 c).
d) An approach for the distribution of the ground reaction is selected and a fixed support arranged at the wall toe, with a support force equal to zero for a free earth support.
e) The flexibility of the earth support shall be identified with the aid of mobilisation functions, using the modulus of subgrade reaction method or the finite-element method (FEM).

5. Any analysis method may be adopted to determine the characteristic action effects and design the sections. For multiple propped soldier piles, sheet pile walls, waling and similar elements of temporary construction aids the elastic-plastic analysis method may also be adopted, in addition to the elastic-elastic method. The plastic-plastic method, referred to as the load limit design method in the second English edition of the EAB, shall not be adopted for excavation structures, with the exception of analyses for design situation DS-A. See R 27 (Section 4.5) in the second edition and [169] for details of the load limit design method.

   In conjunction with the peculiarities of the toe support in the ground, the following methods may generally be adopted in principle:

a) The classical method involving elastic theory can be combined with a fixed or a flexible toe support, if necessary including a geotechnical restraint. In addition, it is possible to employ the modification described in Paragraph 6.
b) Adoption of the modulus of subgrade reaction method according to R 102 (Section 4.5) and the finite-element method (FEM) according to R 103 (Section 4.6) allows identification of the soil-structure interaction in the embedment zone.
c) In addition, under certain conditions, special geometrical boundary conditions and complex ground conditions can also be identified using the finite-element method (FEM) according to R 103 (Section 4.6).

Proceed according to R 80 (Section 4.3) to specify the embedment depth and select the analysis method.

6. The following moment redistribution is permissible for statically indeterminate systems using a linear-elastic analysis:

a) If mathematical overloading of the soldier piles or the waling occurs at a single support point, that component of the design value of the bending moment exceeding the design value of the bending resistance may be redistributed using the elastic-plastic method described in DIN EN 1993-1-1, Paragraph 4.1 (5), as shown in Figure R 11-2. This may be done if the action effects according to R 12, Paragraph 3 (Section 5.1) or R 16, Paragraph 3 (Section 6.1) were determined on the basis of a realistic pressure diagram.
b) The effects on the bending moments in the adjacent fields and at the adjacent supports must be estimated; however, the shear and support forces at the investigated support may not be reduced.
c) The support moments determined using elastic theory may be reduced or increased by a maximum of 15 % of their maximum values according to DIN EN 1993-1-1. Following moment redistribution, the characteristic material parameters reduced by applying the appropriate partial safety factors may not be exceeded at any point, taking the design values of the normal and shear forces into consideration. In addition, lateral torsional buckling shall be prevented. This moment distribution is only permissible for Class 1 and 2 cross-sections, see the description in R 48 (Section 13.3).
d) The moments may also be redistributed based on DIN EN 1992-1-1, Section 5.5 for in-situ concrete walls and bored pile walls. However, the support moment reduction may not be greater than given in DIN EN 1992-1-1, Section 5.5 as a function of the ductility of the steel, the strength of the concrete and the ratio of the height of the compression zone to the effective structural height of the section. If an in-situ-

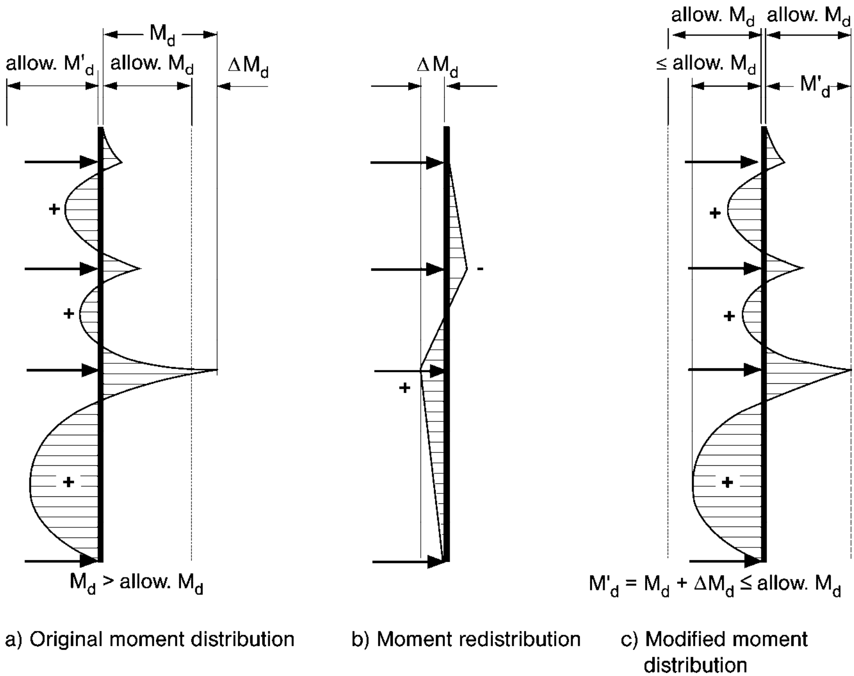

Figure R 11-2. Redistribution of bending moments

concrete wall is subsequently utilised as a load-bearing member in a permanent structure, it may prove expedient to forego reduction of the support moment for the construction stage.

7. Application of the partial safety concept requires strict differentiation of actions and resistances. The previously common practice of superimposing earth pressure and ground reaction in the zone below the excavation level when utilising the global safety factor approach, and the specification of a point of zero stress, is thus no longer permissible for any of the methods discussed. If superimposing is expedient for programming purposes, the actions and the ground reactions shall be subsequently separated again.

4.3 Determination and analysis of embedment depth (R 80)

1. The GEO 2 limit state governs analysis of the actual embedment depth of retaining walls or the embedment depth selected according to Paragraph 9. Accordingly, the analysis is based on the characteristic earth pressure and the corresponding characteristic ground reactions, see Paragraphs 2 to 8.

Alternatively, the embedment depth may be determined as described in Paragraph 11.

2. The characteristic or the representative earth pressure value is obtained from the characteristic soil properties according to R 2 (Section 2.2):

   – from soil self-weight and cohesion according to R 4 (Section 3.2);
   – from characteristic or representative surcharges according to R 6 (Section 3.4).

   The earth pressure from an unbounded distributed load $p_k \leq 10$ kN/m^2 may be superimposed with the earth pressure from soil self-weight and, if applicable, cohesion, according to R 7, Paragraph 1 (Section 3.5). All other earth pressure components from variable actions shall be dealt with separately. However, also see R 105, Paragraph 5 (Section 4.12).

3. The distribution of earth pressure:

   a) from soil self-weight and, if applicable, cohesion, is obtained according to R 5 (Section 3.3);
   b) from surcharges caused by live loads is obtained according to R 7 (Section 3.5).

4. The following applies to determination of the characteristic ground reactions of a wall with a free-earth support:

   a) As long as analysis is not carried out using a continuous elastic support or the finite-element method, the distribution of the ground reaction with embedment depth in the serviceability state may be adopted. In addition to the conservative triangular distribution, a bilinear or parabolic distribution may be expedient in individual cases. See Figure R 80-1 for more details, as well as R 14, Paragraph 4 (Section 5.3) for soldier pile walls and R 19, Paragraph 4 (Section 6.3) for sheet pile walls and in-situ concrete walls.
   b) If, initially, only sufficient embedment depth needs to be demonstrated, a support in the centroid of the anticipated ground reaction may be assumed according to R 11, Paragraph 2 (Section 4.2). If the anticipated ground reactions are subsequently required to determine the action effects according to R 82 (Section 4.4), they may be determined for the distribution adopted from the support force.
   c) If the actual anticipated ground reaction is adopted from the outset, the governing ordinate $\sigma_{h,k}$ of this ground reaction is obtained iteratively from the condition that the support force becomes zero at an assumed support at the height of the wall toe. The characteristic values of the partial support forces are obtained by integration of the ground reaction stresses over the embedment depth t_0.

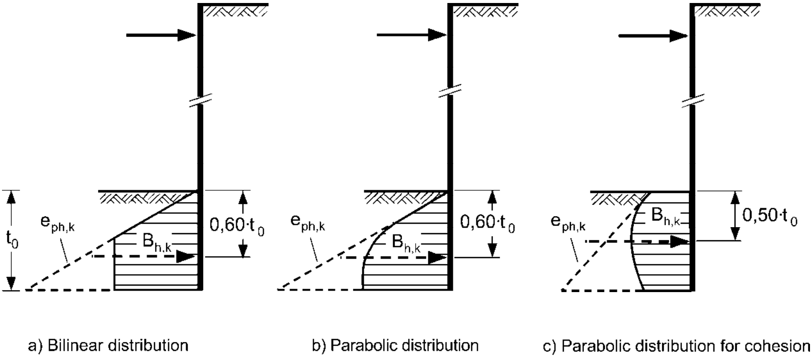

Figure R 80-1. Examples for adopting the ground reaction for free earth support

d) Regardless of whether the procedure according to Paragraph b) or according to Paragraph c) is followed, it may be necessary to take the deflection anticipated for the projected utilisation of the passive earth pressure into consideration at the height of the assumed support or at the height of the wall toe according to R 11, Paragraph 4 d) (Section 4.2). Also see R 14, Paragraph 6 (Section 5.3) for soldier pile walls and R 19, Paragraph 6 (Section 6.3) for sheet pile walls and in-situ concrete walls.

5. For fixed earth support the ground reaction may be assumed according to *Blum's* load approach [23]. This assumes a linear increase in the ground reaction with depth as far as the theoretical toe, see Figures R 25-1 and R 252 (Section 5.4), and R 26-1 and R 262 (Section 6.4). The following apply:

a) A vertical tangent to the deflection curve is required at the assumed theoretical toe for a full geotechnical restraint of supported walls. The corresponding ground reaction ordinate $\sigma_{ph,k}$ is obtained iteratively using a framework analysis application and under the condition that

– either the tangent to the deflection curve contacts the nearest support point for an assumed hinged support at the height of the theoretical toe;
– or the restraint moment becomes zero at an assumed fixed restraint at the height of the theoretical toe.

The minimum required embedment depth is obtained from an additional iteration according to Paragraph c).

b) The vertical tangent condition does not apply for partially restrained, supported walls. Accordingly, a hinged support is assumed at the theoretical toe. The ordinate $\sigma_{ph,k}$ at the height of the theoretical toe is obtained from the condition that the design support force is not greater than the design resistance. This is approximately the case for cohesionless soils if $\sigma_{ph,k} \leq e_{ph}/(\gamma_{GQ} \cdot \gamma_{Ep})$ is used to determine the action effects. The following may be adopted as divisors:

– $(\gamma_{GQ} \cdot \gamma_{Ep}) \approx 1.20 \cdot 1.30 = 1.56 \Rightarrow 1.60$
  for the design situation DS-T:
– $(\gamma_{GQ} \cdot \gamma_{Ep}) \approx 1.10 \cdot 1.25 = 1.37 \Rightarrow 1.40$
  for the design situation DS-T/A:
– $(\gamma_{GQ} \cdot \gamma_{Ep})$ $1.00 \cdot 1.20 = 1.20 \Rightarrow 1.20$
  for the design situation DS-A.

An exact method is described in [168]. The required embedment depth can be determined directly by adopting the design values for actions and resistances, and with a given angle of inclination at the hinged support point.

c) The characteristic values of the support forces in the ground can be determined from the ordinates $\sigma_{ph,k}$ of the ground reaction and the embedment depth t_1 or t'_1 down to the theoretical toe. The governing embedment depth is obtained, adopting the design values according to Paragraph 6 and Paragraph 7, from the additional condition that the limit equilibrium condition according to Paragraph 8 given by

$$B_{h,d} = E_{ph,d}$$

is fulfilled.

6. The partial support force design values are obtained

a) as a result of the earth pressure from soil self-weight, unbounded distributed load $p_k \leq 10$ kN/m^2 and, if applicable, cohesion according to R 4 (Section 3.2) by multiplying the characteristic values by the partial safety factor γ_G;
b) as a result of the proportion of earth pressure surcharges due to unbounded distributed loads q_k or q_{rep} above $p_k = 10$ kN/m^2, or from strip or linear live loads q'_k or q'_{rep}, by multiplying the characteristic values by the partial safety factor γ_Q.

The governing design value of the support force $B_{h,d}$ is the sum of the design values of the partial support forces. Otherwise, attention is drawn to the possible simplifications described in R 105, Paragraphs 3 to 5 (Section 4.11).

7. The following rules apply for determination of the design value of the passive earth pressure:
   - Section 5 for soldier pile walls and bored pile walls;
   - Section 6 for sheet pile walls and in-situ concrete walls.
8. It shall be demonstrated that the design value of the support force is only as large as the passive earth pressure design value

   $B_{h,d} \leq E_{ph,d}$

   If applicable, the selected embedment depth may be reduced until the support force design value is exactly as great as the passive earth pressure design value.
   Generally, the ground reaction in the failure state of the soil is distributed differently to that in the serviceability state (see Paragraph 4). The resulting eccentricity moment may be disregarded during analysis.
9. Generally, only the embedment depth corresponding to the selected structural system need be taken into consideration in the individual advancing states, e.g. a free earth support, a partial restraint or a full geotechnical restraint. It is permissible to adopt the most suitable respective procedure for each construction stage, e.g. according to R 102 (Section 4.5) or R 103 (Section 4.6).

   If that part of the wall not taken into consideration structurally is subject to water pressure, the effects shall be investigated in a stability analysis and, if applicable, the serviceability investigated.
10. The partial safety factors γ_G and γ_Q are summarised in Table 6.1 of Annex A 6.
11. In linear-elastic systems the embedment depth may also be determined directly by adopting the design values of actions and resistances. However, an analysis compliant with Paragraph 8 must always be performed.

4.4 Determination of action effects (R 82)

1. In principle the characteristic action effects are determined similar to R 80 (Section 4.3). The following also applies:
   a) For single-propped walls with a low support and double-propped walls with a high support it shall be taken into consideration that the actions from dredging and lifting equipment operating only a short distance from the edge of the excavation may have a different impact in individual cases in terms of favourable or unfavourable actions when determining the embedment depth than when determining the action effects.

b) If

   - at least medium-dense, cohesionless soil or at least stiff, cohesive soil is present below the excavation level and;
   - a distribution increasing linearly with depth is assumed when applying the ground reactions;

   a higher utilisation of the passive earth pressure may be assumed when determining bending moments, shear forces and support forces at the supports, in contrast to analysis of embedment depth. Also see:

   - R 14, Paragraph 5 (Section 5.3) and R 25, Paragraph 9 (Section 5.4) for soldier pile walls or;
   - R 19, Paragraph 5 (Section 6.3) and R 26, Paragraph 10 (Section 6.4) for sheet pile walls and in-situ concrete walls.

2. Generally, linear-elastic behaviour of the structure may be assumed. However, it may be necessary in individual cases to assume non-linear behaviour, e.g. when considering deformations, or when using the modulus of subgrade reaction method or the finite-element method.

3. For linear-elastic behaviour the characteristic action effects may be determined individually for each action. If the largest field moments from permanent actions and the largest field moments from variable actions are located at different positions, the respectively largest field moments max $M_{G,k}$ and max $M_{Q,k}$ may be regarded as governing in simplification. The following procedure is used for a more precise analysis:

   a) the maximum value max $M_{G,k}$ of the characteristic field moment $M_{G,k}$ from permanent actions $S_{G,k}$, e.g. from earth pressure and water pressure, is determined separately;
   b) the maximum values max $M_{i,k}$ of the field moments $M_{i,k}$ are determined for each variable action $S_{Qi,k}$ or representative action $S_{rep,i}$, together with the permanent actions $S_{G,k}$;
   c) the maximum values max $M_{qi,k}$ of the field moments for the respective variable action $S_{Qi,k}$ or representative action $S_{rep,i}$ are obtained as a difference

   $\max M_{Qi,k} = \max M_{i,k} - \max M_{G,k}$ or
   $\max M_{rep,i} = \max M_{i,k} - \max M_{G,k}$

   The determined characteristic or representative action effects are converted to design values according to R 81, Paragraph 4 (Section 4.1). See R 105, Paragraph 5 (Section 4.11) for possible simplifications when determining action effects from permanent and variable actions.

4. The following applies to non-linear system behaviour for all action effects and for all governing action combinations within the respective design situations DS-T, DS-T/A and DS-A:

   a) the characteristic action effects $E_{G,k}$ from permanent actions $S_{G,k}$ are determined separately;
   b) together with the permanent actions $S_{G,k}$, the action effects E_k or E_{rep} are determined for every possible combination of variable actions $S_{Qi,k}$ or representative actions $S_{rep,i}$;
   c) the action effects for the respective combination of variable actions $S_{Qi,k}$ are obtained as differences.

   $E_{Qk} = E_k - E_{G,k}$ or

   $E_{rep} = E_k - E_{G,k}$.

   See Paragraph 3 for determination of the maximum field moment and for converting the characteristic action effects to design action effects.

5. If the largest field moment from permanent actions and the largest field moment from variable actions are not located at the same position, it is permissible, according to Paragraph 3 or Paragraph 4, to adopt the position at which the field moment M_k exhibits its greatest value. The location at which the field moment M_d exhibits its greatest value governs a precise analysis. The moment diagram for

   $M_d = M_{G,d} + M_{Q,d}$

   shall be determined for this purpose. Generally, this more precise investigation may be dispensed with. Otherwise, attention is drawn to the possible simplifications according to R 105, Paragraph 5 (Section 4.12).

6. When changing from one construction stage to the next the action effects of the new construction stage should be determined by superimposing the action effects of the previous construction stage with those caused by the simultaneous and fundamental change in the actions and in the structural system. This is in contrast to R 11, Paragraph 2 b) (Section 4.2), where each construction stage may be analysed separately. This is especially the case if the retaining wall is supported by bracing at the height of the excavation level before removal of the lowest set of struts or before the lowest set of anchors is unloaded, e.g. by blinding concrete or by the bottom slab of a structure. This problem also occurs in excavations in water with an underwater concrete slab. Also see Figure R 63-3 (Section 10.6).

4.5 Modulus of subgrade reaction method (R 102)

1. The modulus of subgrade reaction method may be employed for analysis of embedment depth, for determination of action effects and for serviceability analyses. This allows the soil-wall interaction, the actual structural behaviour and the anticipated deflections and deformations to be more realistically identified than when assuming a predetermined distribution of ground reactions and deflection of the wall toe.

   Adopting the modulus of subgrade reaction method assumes a realistic modulus of subgrade reaction is determined. This requires expertise and experience in geotechnics.

2. It may be assumed in approximation that the original at-rest earth pressure on the excavation side of the wall remains generally unaffected even after soil removal is complete [15]. It is obtained in the general case as shown in Figure R 102-1 from:

   $e_{0g,k} = \gamma \cdot K_0 \cdot (H + z_p)$.

   However, once the excavation is complete only the passive earth pressure limit value:

   $e_{ph,k} = e_{pgh,k} + e_{pch,k}$

   can be effective in the region immediately below the excavation level due to the reversal of the principal stresses following unloading. The same angle of inclination $\delta_{p,k}$ may be adopted for determination of the passive earth pressure as for determination of the embedment depth and the action effects. Figure R 102-1 shows the case where $e_{pch,k} = 0$.

3. The ground reaction over and above the at-rest earth pressure below the intersection of $e_{0g,k}$ and $e_{ph,k}$ may be adopted as a function of the local displacement s_h as a subgrade reactions, see Figures R 102-1 and R 102-2.

   $\sigma_{Bh,k} = k_{sh,k} \cdot s_h$

   Also see Paragraphs 4 to 8 for determining and adopting the modulus of subgrade reaction $k_{sh,k}$. The sum of the stresses from at-rest earth pressure $e_{0g,k}$ and ground reaction $\sigma_{Bh,k}$ may not exceed the passive earth pressure stresses $e_{ph,k}$.
   If the intersection of $e_{0g,k}$ and $e_{ph,k}$ lies below the base of the wall, analysis using the modulus of subgrade reaction method is not possible because the greatest possible ground reaction is already available to accept support forces without noticeable displacement.

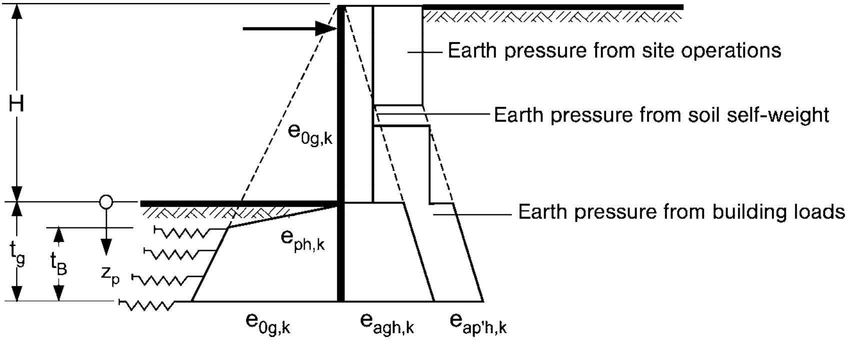

Figure R 102-1. Load model for elastic support in cohesionless soil

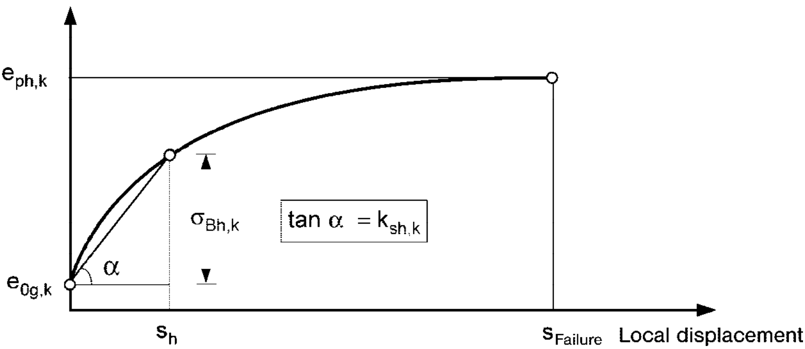

Figure R 102-2. Determination of the modulus of subgrade reaction

For programming purposes it may be expedient to integrate the subgrade reaction in the initial stress $e_{og,k}$. This then gives the modulus of subgrade reaction:

$$k^*_{sh,k} = \frac{\sigma_{Bh,k} + e_{ogh,k}}{s_h}$$

The following details are with reference to the modulus of subgrade reaction $k_{sh,k}$.

4. In approximation, the modulus of subgrade reaction $k_{sh,k}$ may be derived from the oedometric modulus $E_{Sh,k}$:

   a) The following applies in approximation for in-situ concrete walls and sheet pile walls:

$$k_{sh,k} = \frac{E_{Sh,k}}{t_B}$$

The embedment depth t_B utilised by the subgrade is governing.
Where walls are longer than structurally necessary, the depth t_B utilised by the subgrade may be determined in approximation from the structurally required embedment length.

b) The following applies for soldier piles based on DIN 1054:

$$k_{sh,k} = \frac{E_{Sh,k}}{b}$$

The flange width b is governing for driven soldier piles. For soldier piles installed in pre-drilled boreholes and concreted at the base, the borehole diameter D replaces the flange width b. Otherwise, this approach assumes that a displacement of $s = 0.03 \cdot b$ or $s = 0.03 \cdot D$ or a maximum of 20 mm is not exceeded. According to DIN 1054 the diameter D shall be limited to one meter for analysis purposes. This applies accordingly for the width b in general.

c) The oedometric modulus $E_{Sh,k}$ is derived from the anticipated stress range. If the oedometric modulus E_S is only known in the vertical direction it must be converted, in approximation, to a horizontal direction using a factor of $0.5 \leq f \leq 1.0$.

5. Guide values for mean subgrade reaction moduli applicable to continuous walls in cohesionless soils are given in Table R 102-1. The values depend on the relative density and were determined on an empirical basis. They include the approximate influence of preloading from the weight of the excavated soil and below non-flowing water. The values may be doubled for use above water.

   The values were determined at a passive earth pressure utilisation factor $\mu_a \approx 1$ for the DS-T design situation. For a utilisation factor $\mu_a = B_{h,d}/E_{ph,d} < 1$, higher modulus of subgrade reaction values may also occur due to the non-linearity of the mobilisation curves, see Figure R 102-2.

   Values between 3 MN/m³ and 9 MN/m³ may be adopted for cohesive soils of stiff to semi-solid consistency. However, higher values may also be specified based on regional experience.

Table R 102-1. Ranges of empirical submerged modulus of subgrade reaction values for a passive earth pressure utilisation factor $\mu_a \approx 1$ in the DS-T design situation.

Cohesionless soil Relative density			
Loose	Medium-dense	Dense	Very dense
1 – 4 MN/m³	3 – 10 MN/m³	8 – 15 MN/m³	12 – 20 MN/m³

6. In addition to the linear approach for the modulus of subgrade reaction and simultaneous limitation by the passive earth pressure, the subgrade can also be modelled by local, non-linear mobilisation approaches [1, 126, 131, 150, 168]. Realistic values for the modulus of subgrade reaction can also be determined using finite-element analyses. However, R 103 shall be observed.

7. If the stiffness conditions of the retaining wall and the ground allow a restraint and backrotation of the base of the wall, the following apply below the point of zero displacement on the earth side:

   – the at-rest earth pressure may be adopted in place of the active earth pressure;
   – the determined modulus of subgrade reaction may be as much as doubled without further analysis, if the soil conditions are not impaired.

8. Generally, when using the methods described in Paragraphs 4 and 5, a constant modulus of subgrade reaction may be assumed. It may be expedient to adopt a modulus of subgrade reaction increasing with depth for large embedment depths, or to increase it in stages with depth. If an average, conservative value is not adopted, the modulus of subgrade reaction should be adjusted to the ground conditions where soil layering changes.

9. Generally, a realistic average value of the modulus of subgrade reaction may be adopted for analysis. If in doubt, it may be necessary to perform the analysis using upper and lower limit values in order to study the possible impacts.

10. It shall be ensured in accordance with R 80, Paragraph 8 (Section 4.3) that sufficient safety against failure of the ground in front of the toe of the soldier pile or in front of the wall is given:

    a) It shall be verified for continuous walls that the limit state condition

       $B_{h,d} = B_{Bh,d} + E_{V,d} \leq E_{ph,d}$

       is fulfilled, where

       $B_{h,d}$ the design value of the resultant support force according to Paragraph 11;
       $B_{Bh,d}$ the design value of the resultant from soil stresses $\sigma_{Bh,k}$;
       $E_{V,d}$ the design value of the remaining at-rest earth pressure force;
       $E_{ph,d}$ the design value of the passive earth pressure according to Paragraph 12.

       The same angle of inclination δ_p may be adopted for determination of the characteristic passive earth pressure as for determination of the embedment depth and the action effects.

b) It shall be verified for soldier pile walls that the limit state condition

$$B^*_{h,d} = B_{Bh,d} + b \cdot E_{V,d} \leq E^*_{ph,d}$$

is fulfilled. In addition to the previous information:

$B^*_{h,d}$ the design value of the resultant support force according to Paragraph 11 in terms of the soldier pile;

b the width of the soldier pile or the diameter of the concreted soldier pile;

$E^*_{ph,d}$ the design value of the three-dimensional passive earth pressure in front of the soldier pile according to R 14, Paragraph 1 (Section 5.3).

The same angle of inclination δ_p may be adopted for determination of the characteristic passive earth pressure as for determination of the embedment depth and the action effects.

11. The characteristic value of the ground reaction $B_{Bh,k}$ from soil stresses $\sigma_{Bh,k}$ consists of one component from permanent actions and one from variable actions. When determining the proportions of the support forces from permanent actions $B_{BGh,k}$ and from variable actions $B_{BQh,k}$, the proportion from variable actions $B_{BQh,k}$ may be determined by subtraction of the proportion from permanent actions $B_{BGh,k}$ from the total reaction $B_{Bh,k}$, based on R 82, Paragraph 4 (Section 4.4)

$$B_{BQh,k} = B_{Bh,k} - B_{BGh,k}$$

The design values $B_{BGh,d}$ and $B_{BQh,d}$ are obtained by multiplying the characteristic values by the partial safety factors γ_G and γ_Q. The design value $E_{V,d}$ of the resultant remaining at-rest earth pressure is obtained from the characteristic value $E_{V,k}$ by multiplying by the partial safety factor γ_G.

12. For a free earth support the passive earth pressure $E_{phP,k}$ may be adopted for parallel deflections when carrying out analysis according to Paragraph 10, as shown in Figure R 102-3 a). Given a full or partial restraint effect the passive earth pressure $E_{phF,k}$ governs rotation around the toe as shown in Figure R 1023 b), assuming that rotation around a higher point, and thus a combination of toe rotation and parallel displacement cannot occur in supported walls in the failure state.

In approximation, the following relationship applies to continuous walls in cohesionless soils according to [91] and [131]

$$0.50 \cdot E_{phP,k} \leq E_{phF,k} \leq 0.62 \cdot E_{phP,k}$$

This relationship may also be applied in approximation to cohesive soils.

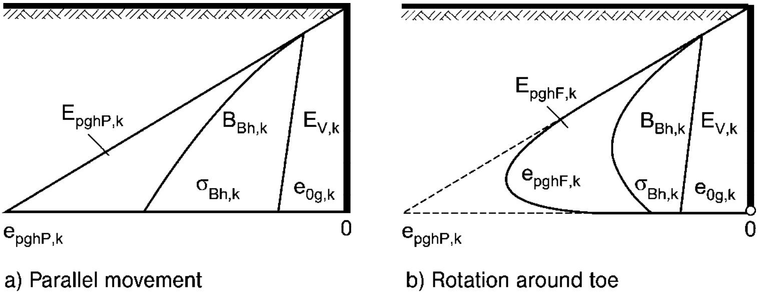

Figure R 102-3. Utilisation of passive earth pressure in cohesionless soil

13. The $E_{V,k}$ component shall be taken into consideration for analysis according to Paragraph 10 even if it is balanced wholly or in part against the loads on the earth side of the wall in a practical analysis for application programming reasons.

14. For analysis according to R 9 (Section 4.8), which guarantees the occurrence of the selected negative angle of passive earth pressure $\delta_B = \delta_p$, $B_{h,k}$ represents the characteristic value of the support force according to Paragraph 11. For continuous walls it is obtained from

    $B_{h,k} = B_{Bh,k} + E_{V,k}$

    For soldier pile walls the support force per unit of length

    $B_{h,k} = (B_{Bh,k} + b \cdot E_{V,k}) / a$

    governs.

15. Alternatively, for analysis according to R 84 (Section 4.9), which guarantees that the downward-acting vertical forces in the embedment zone of the wall can be transferred to the subsurface with sufficient safety, the vertical component of the ground reaction resultant may be replaced by the skin friction.

4.6 Finite-element method (R 103)

1. The finite-element method (FEM) is suitable for:

   – determination of the characteristic stresses in governing sections through the excavation structure and in the soil-structure interfaces;
   – analysis of ground and excavation structure deformations;

- geohydraulic analyses;
- analysis of safety regarding slope failure and overall stability.

Details can be taken from the following paragraphs.

2. Numerical analyses of excavation structures using FEM can be particularly useful if the use of classical beam structural analysis, in association with simplified load approaches, leads to inadequate results due to geometrical boundary conditions or complex ground conditions, or if special demands are placed on the analysis results. It is generally sufficient to investigate the problem using a 2D model. In exceptional cases it may be necessary to use a 3D model. FEM models are used in the following cases, for example:

   a) Retaining walls with support conditions that do not allow confident determination of the magnitude and distribution of earth pressure, for example heavily deforming walls;
   b) Excavations with complex geometrical dimensions, e.g. protruding or recessed corners which do not allow confident determination of earth pressure distribution using classical assumptions;
   c) Staggered retaining walls with a berm width which does not allow confident determination of the magnitude and distribution of earth pressure using classical assumptions;
   d) Excavation structures in which a realistic assessment of the impacts of excavation and strut or anchor prestressing on the earth pressure redistribution and the deflections of the retaining wall is required;
   e) Excavation structures in which a realistic assessment of seepage and the associated water pressures is required;
   f) Excavations adjacent to buildings, pipelines, other structures or traffic areas;
   g) Exceptionally deep excavations.

   Detailed notes and worked examples for applying FEM for the analysis of excavation structures can be taken from the Recommendations of the Working Group "*Numerik in der Geotechnik*" [122] (Numerical Methods in Geotechnics).

3. The use of FEM and specifying the adopted constitutive equations require relevant experience and special care. Because specialised knowledge of soil mechanics is required, especially to determine the material parameters and the state variables, the structures generally belong in Geotechnical Category GC 3 according to DIN 1054. The following points are recommended:

   a) A geotechnical expert in terms of DIN 1054, trained in the required field and in possession of the appropriate experience, should be employed for planning the necessary investigations and monitoring the technically correct execution of exposures, as well as for field and laboratory testing.

b) It is anticipated that the geotechnical expert recommends a constitutive equation allowing realistic determination of the stress and displacement conditions, taking the problem and the local ground conditions into consideration.
c) The Recommendations of the Working Group "Numerical Methods in Geotechnics" [122] shall be observed when determining the material parameters and state variables required for numerical analysis.

4. The following procedure should adhered to for numerical analysis:

   a) A suitable constitutive equation, which allows consideration of excavation loading, unloading and reloading processes, shall be selected for the soil.
   b) The characteristic values of the parameters required for the selected constitutive equation are determined from laboratory and field tests.
   c) If possible, initial numerical analyses shall be performed using measurement data from excavations in similar ground conditions in order to calibrate and check the selected parameters for the constitutive equation.

   In order for the analysis assumptions to be transparent, the numerical analysis should always be preceded by verifiable documentation of the processing steps for points a) to c).

5. Generally realistic upper and lower limit values of the respective soil parameters should be included in the analysis

   a) Conservative values are initially required for determination of the characteristic stresses for analysis of bearing capacity.
   b) For analysis of serviceability it is generally sufficient to adopt the average value of the upper and lower characteristic values as the soil parameter with the highest probability of occurring.

   In both cases it may be necessary to perform the analysis using upper and lower characteristic values, e.g. if the impacts are in part favourable and in part unfavourable or if the possible analysis result range needs to be determined.

6. In particular if the dimensions of the components used in the numerical model cannot be defined using empirical values, e.g. thickness and length of the retaining wall and the prestressing forces of struts and anchors, initial calculations using analytical methods should be performed in order to reduce the iterative optimisation effort for the component dimensions.

   The anchor lengths for tied-back excavations are generally also determined in initial calculations using analytical methods. See [152–154] for further details on using the finite-element method.

7. It is necessary to adopt suitable contact elements to consider the soil-structure interaction between the retaining wall and the ground. Also see R 4, Paragraph 2 (Section 3.2) for adopting the earth pressure angle.

8. According to R 80, Paragraph 8 (Section 4.3), it shall be ensured that sufficient safety against failure of the ground in front of the base of the soldier pile or in front of the wall is given. It shall be verified that the limit state condition

   $B_{h,d} \leq E_{ph,d}$

   is fulfilled, where

   $B_{h,d}$ the design value of the resultant support force according to Paragraph 9;

   $E_{ph,d}$ the design value of the passive earth pressure according to Paragraph 10.

   The same angle of inclination δ_p may be adopted for determination of the characteristic passive earth pressure as for determination of the embedment depth and the action effects.

9. The ground reaction design value $B_{h,d}$ consists of one component from permanent actions and one from variable actions. When determining the proportions of the support forces from permanent actions $B_{Gh,d}$ and from variable actions $B_{Qh,d}$, the proportion from variable actions $B_{Qh,k}$ may be determined by subtraction of the proportion from permanent actions $B_{Gh,k}$ from the total reaction $B_{h,k}$, based on R 82, Paragraph 4 (Section 4.4):

   $B_{Qh,k} = B_{h,k} - B_{Gh,k}$

   The design values $B_{Gh,d}$ and $B_{Qh,d}$ are obtained by multiplying the characteristic values by the partial safety factors γ_G and γ_Q.

   Alternatively, variable actions may also be multiplied by a factor γ_Q/γ_G in order to simplify the analysis discussed above. If the effects of this simplification are not readily discernible on special constructions, comparative analyses shall be performed beforehand.

   If variable actions can act favourably, separate computations shall be performed without adopting variable actions.

10. For wall base deflections corresponding to a free earth support, the passive earth pressure $E_{phP,k}$ may be adopted for parallel deflections when performing an analysis according to Paragraph 8, as shown in Figure R 102-3 a). For deflections of the wall base corresponding to a partial or full restraint effect, the passive earth pressure $E_{phF,k}$ for rotation around the toe as shown in Figure R 102-3 b) is governing, also see R 102, Paragraph 12. In ap-

proximation, the following relationship applies to continuous walls in cohesionless soils according to [91] and [131].

$0.50 \cdot E_{phP,k} \leq E_{phF,k} \leq 0.62 \cdot E_{phP,k}$.

This relationship may also be applied in approximation to cohesive soils.

11. Analysis according to R 9 (Section 4.8), which guarantees the occurrence of the selected negative angle of passive earth pressure $\delta_B = \delta_p$, may be dispensed with, because adherence to the corresponding equilibrium condition is inherently ensured by the numerical equilibrium.

12. The analysis according to R 84 (Section 4.9), which guarantees that the downward-acting vertical forces in the embedment zone of the wall can be transferred to the subsurface with sufficient safety, shall be performed accordingly. The vertical components of the characteristic earth pressure are obtained by integrating the vertical stresses over the rear face of the wall, for example.

13. For homogeneous, cohesive soil and in cohesive soil layers, tensile stresses are excluded in the constitutive equation. In addition, a test for adherence to a minimum earth pressure shall be incorporated in the design. See R 4, Paragraphs 3 to 5 (Section 3.2).

14. The following points shall be observed for additional stability analyses of anchored walls:

    a) Analysis of safety against slope failure and general failure can only be performed using FEM if based on the Fellenius circular-arc method, and the shear strength in the soil and at the soil-structure interface is reduced in stages until no mathematical equilibrium state is possible or a mathematical failure state occurs. This method is known as "φ-c reduction". The limits of this method and notes on defining the necessary convergence criteria can be taken from the Recommendations of the Working Group "Numerical Methods in Geotechnics" [122].
    b) The safety against base heave and deep-seated stability are allocated to the STR, GEO 2 limit state to DIN 1054 and analysis is performed using the corresponding partial safety factors. For analysis of deep-seated stability – used to demonstrate sufficient anchor length – the notes in Paragraph 6 apply.

15. The input values and results of numerical analyses shall be verifiably documented. This is particularly the case for loads on the retaining wall and for action effects, e.g. bending moments, shear forces and support forces, as well as for displacements, e.g. deformations of the wall and the ground. The following parameters are recommended:

a) the horizontal earth pressure components and the water pressure over the height of the retaining wall in the individual excavation stages;
b) the action effects over the height of the retaining wall in the individual excavation stages;
c) the action effects at the governing sections as a function of the construction stage;
d) the horizontal wall displacement at various points of the retaining wall as a function of the construction stage;
e) the surface settlements at various points of the ground surface as a function of the construction stage;
f) the heave at various points of the excavation base;
g) the potential distributions at governing structural elements during geohydraulic analyses of groundwater flow.

In addition, graphical visualisations of plasticised zones, stress trajectories and displacements may be useful as vector or colour plots for result evaluation.

4.7 Analysis of the vertical component of the mobilised passive earth pressure (R 9)

1. It shall be verified that the occurrence of the selected negative angle of inclination is guaranteed for the mobilised passive earth pressure. This is the case if the sum $V_k = \Sigma V_{k,i}$ of all downward directed characteristic actions is equal to or greater than the vertical component $B_{v,k}$ of the characteristic support force B_k

   $V_k \geq B_{v,k}$

   The required analysis is not allocated to a limit state. It comprises only adherence to the equilibrium condition $\Sigma V_k = 0$.

2. The following shall be observed for soldier pile walls, sheet pile walls or in-situ concrete walls with a free earth support according to R 14 (Section 5.3) or R 19 (Section 6.3):

   a) Downward-acting characteristic actions include, for example, the self-weight G_k of the wall, permanent surcharges P_k acting immediately upon the wall, the vertical component E_{av} of the earth pressure determined using a positive earth pressure inclination angle and, if applicable, the vertical component A_v of any anchor force.
   b) The characteristic ground reaction force B_k corresponds to the characteristic mobilised passive earth pressure mob. $E_{p,k}$. The angle of inclina-

tion of the support force B_k and that of the mobilised passive earth pressure are thus identical. In addition, the inclination angles of the characteristic variables and the angle of the design condition may be equated to each other.

c) Taking the assumptions in 2 b) into consideration, the characteristic value of the vertical component $B_{V,k}$ of the support force B_k is obtained from the horizontal component $B_{h,k}$ using

$$B_{v,k} = B_{h,k} \cdot \tan \delta_{p,k}$$

d) The governing value for curved slip surfaces according to R 89, Paragraph 3 (Section 2.3) is adopted for the support force B_k inclination angle $\delta_{p,k}$. This also applies if the passive earth pressure is determined using planar slip surfaces and a reduced angle of inclination in order to obtain realistic K_p values during analysis using planar slip surfaces. This avoids non-conservative analysis of the vertical component.

3. For soldier pile walls, sheet pile walls or in-situ concrete walls restrained in the ground according to R 25 (Section 5.4) or R 26 (Section 6.4), whose restraint was computed using *Blum's* load approach [23], a simplified and a precise analysis are differentiated:

   a) The simplified analysis is

   $$V_k = G_k + E_{av,k} + A_{v,k} + C_{v,k} \geq B_{v,k}$$

   The vertical component $B_{v,k}$ may be determined as described in Paragraph 2 c).

   b) For a more precise analysis the computed support force $B_{h,k}$, as shown in Figure R 9-1, may be reduced by half of the corresponding force $C_{h,k}$

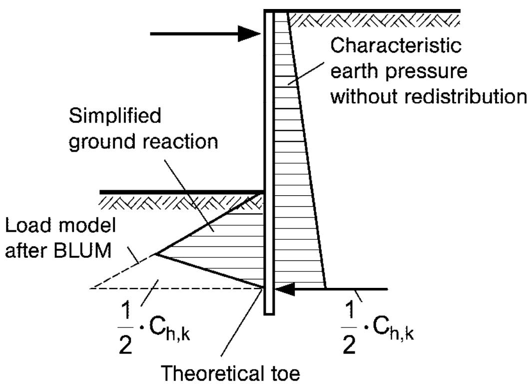

Figure R 9-1. Effective component of the ground reaction for *Blum's* earth restraint in order to determine the actual effective support forces. Accordingly, only half of the downward-acting component of the force C_k may be incorporated in the analysis as a favourable action:

$$V_k = G_k + E_{av,k} + A_{v,k} + ½ \cdot C_{v,k} \geq (B_{h,k} - ½ \cdot C_{h,k}) \cdot \tan \delta_{p,k}.$$

In both the simplified and the more precise analysis, the positive inclination angle of the equivalent force C_k shall generally be limited to $\delta_C \leq 1/3 \cdot \varphi'_k$.

4. The vertical forces from variable actions may not be taken into consideration for either the analysis according to Paragraph 1 or for that according to Paragraph 3, if they favourably impact on the analysis of $\Sigma V_k = 0$.

5. For anchored retaining walls with an average anchor inclination $\alpha_A \geq 15°$, verification that the selected negative angle of inclination is guaranteed for the mobilised passive earth pressure may be dispensed with.

6. If the vertical component of the passive earth pressure cannot be verified, the angle of inclination of the support force B_k shall be reduced. This leads to a reduction in the magnitude of the passive earth pressure. Accordingly, the embedment depth and the design action effects shall be determined once again using the modified data.

7. The analysis described here assumes that the vertical component of the resultant of all actions is relatively small. Regardless of this, analysis of the transfer of vertical forces into the subsurface according to R 84 (Section 4.9) shall be performed. This assumes that the vertical component of the resultant of all actions is relatively large. Generally, only one of the two analyses governs design.

8. For cut-off walls manufactured using a hardening cement-bentonite slurry with an inserted sheet pile wall or inserted soldier piles, it shall be verified that the vertical component $B_{V,k}$ of the characteristic support force B_k can be transferred to the sheet pile wall or the soldier piles via bonding stress. Also see [127].

4.8 Analysis of the transfer of vertical forces into the subsurface (R 84)

1. It shall be guaranteed that the downward directed vertical actions can be transferred from the wall to the subsurface. For this purpose it shall be verified that according to the limit state condition

   $$V_d \leq R_d$$

the sum V_d of the design values of the downward directed components of the actions are at most as great as the sum R_d of the design values of the resistances.

2. The following shall be observed for soldier pile walls, sheet pile walls or in-situ concrete walls with a free earth support according to R 14 (Section 5.3) or R 19 (Section 6.3):

   a) The downward directed characteristic actions, e.g. the self-weight of the wall, permanent surcharges acting immediately on the wall, the vertical component of the earth pressure determined using the positive angle of inclination and, if applicable, the vertical component of the anchor forces, shall be converted to design values using the partial safety factors γ_G and γ_Q, separated into permanent and variable actions. See R 105, Paragraph 5 (Section 4.12) for possible simplifications for determining action effects.
   b) All upward-acting characteristic resistances, e.g. the base resistance and the friction force acting on the excavation side of the wall, shall be converted to design values using the corresponding partial safety factors for resistances.
   c) The characteristic base resistances for driven soldier piles, sheet pile walls, bored piles and soldier piles placed in boreholes and grouted at the base, as well as for in-situ concrete walls, are obtained from R 85 (Section 13.10).
   d) Either a skin resistance or the vertical component of the support force B_k may be adopted as the characteristic friction force $R_{v,k}$ on the excavation side of the wall. The following parameters are obtained:

      – the skin resistance of the developed surface A_s of the area and the skin friction $q_{s,k}$ from

      $$R_{v,k} = A_s \cdot q_{s,k}$$

      – the vertical component of the support force B_k from the horizontal support force $B_{h,k}$ and the friction coefficient tan $\delta_{B,k}$ after R 89 (Section 2.3) from

      $$R_{v,k} = B_{h,k} \cdot \tan \delta_k$$

   See R 85 (Section 13.10) and Appendix A 10 for the skin friction $q_{s,k}$.

3. In addition, the following shall be observed for soldier pile walls, sheet pile walls or in-situ concrete walls restrained in the ground according to R 25 (Section 5.4) or R 26 (Section 6.4):

a) In contrast to the analysis according to R 9 (Section 4.7) the characteristic vertical component C_V of the upward-acting equivalent force C is obtained from

$$C_{V,k} = C_{h,k} \cdot \tan \delta_{C,k}$$

The angle of inclination $\delta_{C,k}$ of the equivalent force C may not be greater than the wall friction angle according to R 89, Paragraph 3 (Section 2.3).

b) According to R 9 (Section 4.7) the characteristic support force $B_{h,k}$ shall be reduced by half of the characteristic equivalent force $C_{h,k}$. The vertical component $B_{v,k}$ is reduced accordingly. The computed equivalent force $C_{h,k}$ may in turn only be adopted at half value. Also see Figure R 9-1 (Section 4.7).

4. Skin friction may be adopted as a resistance for diaphragm walls or sheet pile walls in those regions in which they are extended for the entire length of the excavation or staggered in sections over and above that structurally required. It is not necessary to provide continuous reinforcement of the structurally extended sections for diaphragm walls.

5. If transfer of the vertical forces cannot be analysed using the initially selected approach, the positive earth pressure angle shall be reduced. If necessary, a negative earth pressure angle shall be adopted, assuming a corresponding force transfer is possible at all. The associated earth pressure increase shall be taken into consideration. Accordingly, the embedment depth and the design action effects shall be determined once again using the modified data. When adopting a negative earth pressure angle the upward-acting characteristic vertical component E_{avk} of the earth pressure is adopted as a negative action and is therefore subtracted from the remaining characteristic actions V_k.

6. The following apply for determination of the design resistances:

   a) On the resistance side the partial safety factors for the pile resistances may be adopted for the characteristic toe resistance $R_{b,k}$ and skin resistance $R_{s,k}$, and the partial safety factor for the passive earth pressure for the characteristic friction force.
   b) If the retaining wall settlements need to be kept to a minimum, e.g. for excavations adjacent to structures, the characteristic values of the resistances shall be reduced with the aid of a calibration factor $\eta \leq 0.80$. It may also be necessary to analyse serviceability according to R 83 (Section 4.11).

7. For cut-off walls manufactured using a hardening cement-bentonite slurry with an inserted sheet pile wall or inserted soldier piles, it shall be verified

that the wall or soldier pile self-weight can be transferred to the hardened cut-off wall via bonding stress, together with the vertical component A_V of an anchor force. Also see [127].

4.9 Stability analyses for braced excavations in special cases (R 10)

1. It may be necessary to analyse safety against base heave for soils below the excavation level with a characteristic friction angle of less than $\varphi'_k = 25°$. Also see [25, 26, 52, 130]. This analysis forms part of the GEO 2 limit state. The following procedure is used:
   a) The governing forces are those acting on a soil mass of width b_g. Actions include the weight $G_{B,k}$ of the soil mass and, if applicable, any surcharges G_k and Q_{rep}. Resistances include the lateral vertical force T_k and the bearing capacity $R_{n,k}$ of the load-bearing strip of width b_g (Figure R 10-1).
   b) The limit state condition using the design values:

   $$G_{B,d} + G_d + Q_d \geq T_d + R_{n,d}$$

   shall be fulfilled. The width b_g according to [52, 130] shall be varied until the maximum degree of utilisation:

   $$\mu = \frac{G_{B,d} + G_d + Q_d}{T_d + R_{n,d}}$$

   is obtained.

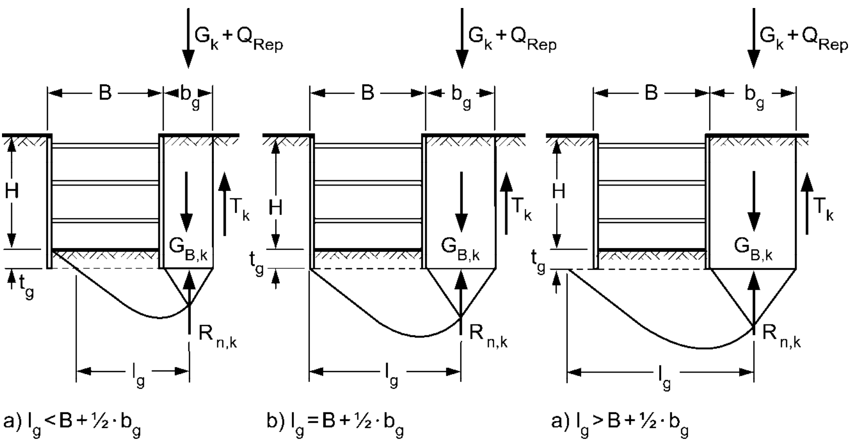

Figure R 10-1. Excavation base heave in homogeneous soil

Only those cases shall be investigated for which the failure prism lies within the excavation (Figure R 10-1 a) or just reaches the opposite side (Figure R 10-1 b). In the case of narrow excavations it is not necessary to vary the width (Figure R 10-1 c), see [52].

c) The limitation of the friction coefficient when determining the friction component of T_k and the peculiarities for narrow excavations [52, 130] shall be observed.

d) The design values T_d and $R_{n,d}$ are obtained from the characteristic values T_k and $R_{n,k}$ by division by the partial safety factor $\gamma_{R,V}$ for bearing capacity.

2. Bearing capacity safety shall be demonstrated regardless of ground conditions if a heavy foundation is present approximately at the excavation level and only a small distance from the outside of the retaining wall (Figure R 10-2).

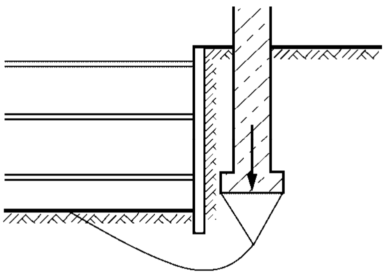

Figure R 10-2. Analysis of bearing capacity for a braced excavation

3. In exceptional cases it may be necessary to analyse general stability in the GEO 3 limit state if large earth pressures are anticipated below the excavation level, e.g. for a very heavy foundation adjacent to the excavation as shown in Figure R 10-3. The effect of strut forces shall be taken into consideration at least when they unfavourably impact on stability due to their position above the center of the slip circle. If they act favourably, e.g. as with the lower set of struts as shown in Figure R 10-3, they may be adopted at the design value of the strut resistance S_d.

4. If any of the cases mentioned in Paragraphs 1 to 3 occur in conjunction with an excavation in water, it may be necessary to take into consideration that the magnitude of the passive earth pressure or the bearing capacity may be impaired. This is particularly the case for low effective vertical stresses below a base liner [96]. Also see R 63, Paragraph 5 (Section 10.6).

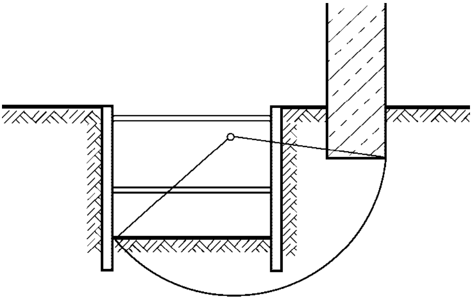

Figure R 10-3. Analysis of general stability for a braced excavation

5. In excavations deeper than 10 m it may be necessary to investigate heave at the excavation level according to R 83 (Section 4.11) and to verify that the associated heave of provisional bridge supports or excavation coverings, or of intermediate supports for buckling protection devices, has no negative impact. Also see [51, 52].

4.10 Serviceability analysis (R 83)

1. The regulations of Sections 5 and 6 ensure that for at least medium-dense, cohesionless soil and at least stiff, cohesive soil, the displacements of the toe support of a multi propped wall remain small and that their magnitude corresponds to the movements and deformations of the rest of the retaining wall. The more detailed regulations in Recommendations R 20 (Section 9.1), R 22 (Section 9.5) and, if applicable, R 23 (Section 9.6) limit the anticipated deformations to such a degree that damage to adjacent structures is generally avoided. Special investigations of the magnitude of deformations and displacements are thus generally unnecessary. However, if, in exceptional cases, there is a danger that the deformations and displacements of the retaining wall will impair the stability or serviceability of adjacent structures despite adhering to the discussed measures, the serviceability limit state shall be analysed in accordance with the Eurocode 7 Handbook, Volume 1.
2. In particular analysis of serviceability may be necessary:
   - for excavations adjacent to very high, poorly founded structures or structures in poor condition;
   - for excavations at a very small distance from, or immediately contacting, existing structures;
   - for excavations adjacent to structures with a simultaneous high water table (also see [96, 97]);

– for excavations adjacent to structures founded in soft, cohesive soils;
– for excavations adjacent to structures with especially exacting demands on adherence to the position of the building, e.g. due to the sensitivity of machines;
– for excavations adjacent to sensitive installations as described in R 20, Paragraph 8 (Section 9.1);
– for excavations with anchors inclined at greater than 35°;
– for excavations without a workspace, where the clear space for the structure could be intolerably restricted.

3. Two cases are differentiated for analysing serviceability:

 a) If the wall deformations need to be more precisely analysed, but the impact on the surroundings are less relevant, the precision of the deformation forecasts can be increased by improving the structural system, e.g. by evaluating the flexibility of anchors, taking pre-deformations in the various construction stages into consideration and applying the subgrade reaction (see Paragraphs 4 to 10).
 b) If both the wall deformations and those of the surrounding soil need to be determined, numerical investigations, e.g. using the finite-element method, taking the initial stress conditions into consideration, are necessary, see R 103 (Section 4.6).

4. Serviceability analysis is performed using the characteristic values for actions. With regard to adopting the active earth pressure or an increased active earth pressure, the same rules apply as for investigation of the GEO 2 or STR limit state. The earth pressure from an unbounded distributed load $p_k \leq 10$ kN/m^2 according to R 24 (Section 2.1) is adopted as a permanent load. Any earth pressure over and above this from an unbounded distributed load q_k or other bounded distributed loads q'_k ensuing from traffic and site operations, including representative loads, need generally only be taken into consideration if the magnitude of the load and the duration of its acting make this necessary.

5. The structural system is defined by the supports above the excavation level. The following points apply for the earth support:

 a) Generally, the actual embedment depth and not the statically required embedment depth shall be assumed, if an embedment depth greater than that statically required has been adopted.
 b) For walls with a free earth support the position of the ground reaction resultant may be assumed according to the information given for the GEO 2 limit state in R 14, Paragraph 4 (Section 5.3) or R 19, Paragraph 4 (Section 6.3), if no elastic support is adopted, e.g. the modulus of subgrade reaction method according to R 102 (Section 4.5).

c) Special investigations according to Paragraph 6 are necessary for walls restrained in the soil.

6. The following approaches are generally available for considering a restraint for flexible retaining walls, in addition to *Blum's* approach:

   a) In approximation the distribution of the ground reactions may be assumed as shown in Figure R 9-1 (Section 4.7), whereby the ordinates of the characteristic passive earth pressure in the region immediately below the excavation level may be adopted, if necessary taking cohesion into consideration.
   b) In the case of cohesionless soils a more precise distribution of the ground reaction from the excavation level to the fulcrum is obtained from [91]. Also see R 102, Paragraph 12 (Section 4.5).
   c) For sufficiently flexible walls the effective restraint is obtained with the aid of the elastic support, e.g. using the modulus of subgrade reaction method according to R 102, Section 4.5.

7. A support force $B_{h,k}$, which shall be accepted by the ground in front of the wall, is obtained using the structural system according to Paragraph 6. In approximation, the corresponding displacement is obtained:

   – according to [20] or [46] for soldier pile walls in cohesionless soil and according to [93] in silty soils;
   – according to [94, 126] for continuous walls in cohesionless soil and according to [95] in cohesive soils. The displacements s_V associated with the remaining at-rest earth pressure as shown in Figure R 102-2 (Section 4.6) may be subtracted.

   Regardless of this, horizontal compression of the soil may occur in excavations in water with a deep base liner, in particular if the water within the excavation is lowered further than is necessary for the respective excavation stage [96].

8. In general, when determining the deformations and displacements of the retaining wall:

   a) the predeformations at the height of the supports before they are installed and;
   b) the strains on anchors resulting from forces over and above the lock-off force;

   shall be taken into consideration. The elastic compression of struts and the movement of the wall towards the ground when prestressing struts or anchors may generally be disregarded.

9. In addition to the horizontal deformations and displacements of the wall, the wall settlements shall also be investigated. Also see R 85 (Section 13.10).

10. The information given above only takes into consideration the behaviour of the wall itself. Movements caused by loosening or compaction of the soil while manufacturing the retaining wall are not identified, e.g.:

    - ground loosening prior to installing the piles of a soldier pile wall;
    - soil removal when drilling, soil collapsing as a result of overcutting;
    - soil unloading due to a pressure drop in the slurry in the trench of a diaphragm wall;
    - soil collapse as a result of soil removal during drilling or anchor installation;
    - ground compaction during driving or anchor casing;
    - ground unloading caused by void formation when drawing sheet piling.

    If these effects cannot be avoided using technical measures, the impacts on wall serviceability shall be approximately estimated. See [155, 156] for additional notes.

11. For anchored walls the movements caused by:

    - tilting of a cofferdam-like soil mass as shown in Figure R 46-1 (Section 7.5);
    - shear deformation of the cofferdam-like soil mass and the soil below it;
    - horizontal displacement of the cofferdam-like soil mass as a result of compression of the soil mass below the excavation level;

    shall also be taken into consideration. These movements and deformations can be estimated according to [72]. More precise investigations based on numerical analysis are possible. Otherwise, see [38] and [39].

12. If the investigation demonstrates that the determined wall deformations and displacements do not fulfil the conditions for serviceability, the following measures may generally be considered:

    - changing the configuration of supports;
    - increasing the embedment depth;
    - installing a toe support at the height of the excavation level before excavation;
    - selecting stronger sections or greater wall thicknesses;
    - for anchored walls, if applicable, the measures described in R 46, Paragraph 3 (Section 7.5).

    If the structural system is considerably altered by one of these measures, a new analysis of the GEO 2 or STR limit state shall be performed.

13. In addition to the deflections of the retaining wall and the deformation of the ground behind it, base heave and heave of the retaining wall may also be taken into account, even in braced excavations. Also see [51] and [52]. The heave is caused by excavation unloading and is later negated either completely or in part by the loads imposed by the structure.

    For excavations in water with a base secured by anchor piles, base heave is anticipated that is considerably greater than that anticipated for dry excavations or excavations in lowered groundwater. Also see [141] and [142]. This is particularly the case if the level of safety prescribed in R 62, Paragraph 3 b) (Section 10.5) is not attained when analysing the UPL limit state, e.g. when adopting the observational method.

4.11 Allowable simplifications in limit states GEO 2 or STR (R 104)

1. The following major changes are associated with the introduction of the partial safety factor approach:

   a) The limit state condition $E_d \leq R_d$ associated with the partial safety factor approach demands strict separation of actions and resistances.
   b) Because of the differing partial safety factors the partial safety factor approach also demands strict separation of permanent and variable actions and representative actions.
   c) Superimposing earth pressure and reduced passive earth pressure is no longer possible. There is therefore no longer a point of zero stress, below which only supporting load ordinates may be adopted.
   d) Incorporation of the earth pressure from loads over and above $p_k = 10\ kN/m^2$, in particular the earth pressure from line or bounded loads caused by construction machinery, in a mutual pressure diagram with the earth pressure from soil self-weight, is no longer possible.
   e) Generally, the anticipated ground reaction stresses may no longer be replaced by a support at the height of their resultant.
   f) Generally, all dimensions shall be estimated beforehand and subsequently optimised by way of iteration.

2. Simplifications that reduce the additional effort imposed by the alterations are described below. Two areas are differentiated:

   a) The number of variable actions is relatively small for excavation structures. In addition, their effects, with a few exceptions, are always unfavourable and are not governing, in contrast to the effects of permanent actions. It is therefore appropriate to allow very general simplifications, if the result is not impaired, or only to a very minor degree. Also see Paragraphs 3 to 5.

b) Transitional regulations are required for the period until new applications based strictly on the partial safety factor approach are available and which provide both the necessary embedment depth and the required action effects. Also see Paragraph 6.

3. All permanent actions may be incorporated in a single action, even if they have different causes. In particular the earth pressure from permanent building loads may be incorporated in a common pressure diagram with the earth pressure from soil self-weight, unbounded surcharge and, if applicable, cohesion according to R 4 (Section 3.2). This also applies in the case of an increased active earth pressure or a reduced or complete at-rest earth pressure. However, when determining the vertical forces it should be noted that the at-rest earth pressure component for a ground surface inclined at $\delta_0 = \beta$ occurs on a horizontal ground surface at $\delta_0 = 0$. The vertical force components should therefore be adopted in the same ratio as the horizontal components of the increased active earth pressure.

4. Because water pressure generally produces unfavourable actions and may be dealt with as a permanent action according to the Eurocode 7 Handbook, Volume 1, it may be incorporated in a mutual pressure diagram with the buoyancy-reduced earth pressure. However, when determining the vertical forces it should be noted that only the earth pressure component with wall friction occurs. The mutual pressure diagram is not expedient if the action effects are determined using classical earth pressure distribution, and earth pressure redistribution according to R 63, Paragraph 3 (Section 10.6) is replaced by surcharges to the determined support forces.

5. All variable actions over and above the unbounded distributed load $p_k = 10$ kN/m^2, in particular equivalent loads q_k from traffic and site operations, as well as the variable component of building loads, may be multiplied by the factor:

   - $f_q = \gamma_Q/\gamma_G = 1.30/1.20 = 1.08$ for the DS-T design situation,
   - $f_q = \gamma_Q/\gamma_G = 1.15/1.10 = 1.05$ for the DS-T/A design situation,
   - $f_q = \gamma_Q/\gamma_G = 1.00/1.00 = 1.00$ for the DS-A design situation,

   and their effects in the shape of earth pressure from live loads be superimposed with the earth pressure from soil self-weight, unbounded distributed load $p_k = 10$ kN/m^2 and, if applicable, cohesion, if they have an unfavourable impact on the embedment depth or on the action effects. The thus determined characteristic action effects then need only be converted to design values using the uniform partial safety factor γ_G.

6. Until new software applications are available it is expedient to determine the required embedment depth using older applications, where the earth

pressure was superimposed with the reduced passive earth pressure. Two routes may be considered:

a) Analysis is based on the global safety factor approach:
   where $\eta_p = 1.50$, if the partial safety factor η_p has a fixed value
   or

   – where $\eta_p = \gamma_{GQ} \cdot \gamma_{R,e} \geq 1.20 \cdot 1.30 = 1.56 \approx 1.6$
     in the DS-T design situation
   – where $\eta_p = \gamma_{GQ} \cdot \gamma_{R,e} \geq 1.10 \cdot 1.25 = 1.38 \approx 1.40$
     in the DS-T/A design situation
   – where $\eta_p = \gamma_{GQ} \cdot \gamma_{R,e} = 1.00 \cdot 1.20 = 1.20$
     in the DS-A design situation

   if η_p can be adopted.

b) Analysis is based on the partial safety factor approach if

   – the characteristic earth pressure is increased using γ_G or γ_Q;
   – the characteristic passive earth pressure is reduced using γ_{Ep} and;
   – the increased earth pressure is superimposed with the reduced passive earth pressure.

   In order to fulfil the formal demands of the partial safety factor approach, the action effects are determined in all cases using the now known embedment depth and analyses of $E_d \leq R_d$ performed in all governing sections.

5 Analysis approaches for soldier pile walls

5.1 Determination of load models for soldier pile walls (R 12)

1. If the conditions given in R 8 (Section 3.1) for reducing the earth pressure from the at-rest earth pressure to the active earth pressure are met, the earth pressure E_a according to R 4 (Section 3.2) and R 6 (Section 3.4) shall be determined from the ground surface to the excavation level, taking into consideration soil self-weight, unbounded distributed load $p_k \leq 10$ kN/m² and, if applicable, cohesion according to R 4 (Section 3.2). The earth pressure below the excavation level is not included in the load model, unless differently stipulated in R 15 (Section 5.5). Figure R 12-1 shows the procedural principle, without consideration of earth pressure from other live loads.

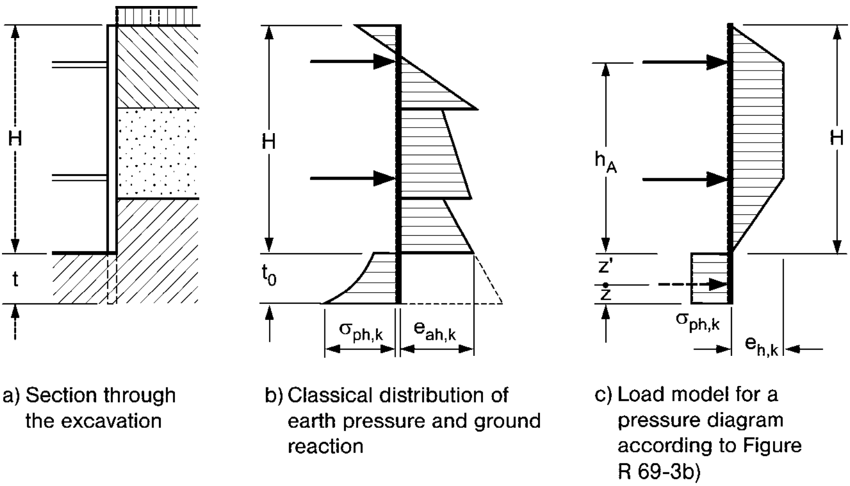

Figure R 12-1. Load model determination for supported soldier pile walls when adopting active earth pressure and a free earth support (example of a double-propped soldier pile wall in stratified ground)

2. For unsupported soldier pile walls with fixed earth support, and for flexibly supported soldier pile walls, the classical earth pressure distribution shall always be adopted for analysis of embedment depth according to R 80 (Section 4.3) and for determination of the action effects according to R 82 (Section 4.4). When investigating forced slip surfaces the starting point is generally assumed to be at the excavation level.

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3. For non-yielding soldier pile walls the load determined according to Paragraph 1 (Figure R 12-1 b) shall be converted to a simple pressure diagram according to R 5 (Section 3.3), corresponding to the anticipated earth pressure redistribution. In the advancing states the selected pressure diagram may generally approach $e_{h,k} = 0$ at the height of the respective excavation state and at the excavation level for the fully excavated condition. If the entire earth pressure from ground level to the base of the soldier pile is incorporated into the redistribution corresponding to R 15, Paragraph 6 c) (Section 5.5) for soldier pile walls with a free earth support, $e_{h,k} \geq 0$ must be defined at the base of the soldier pile. In this case, R 15, Paragraph 7 c) (Section 5.5) also applies to soldier pile walls with fixed earth support. No differentiation need be made between a free earth support and fixed earth supported beams when defining the pressure diagram.

4. The information given in Recommendation R 69 (Section 5.2) may be used as a guide for the choice of a realistic pressure diagram for non-yielding soldier pile walls. A rectangular pressure diagram cannot be regarded as realistic in the majority of cases. If one is adopted the errors associated with this procedure when determining shear and support forces shall be corrected by applying suitable surcharges. Also see R 13 in the 3rd edition [124] and EAB-100 [125].

5. The magnitude and distribution of earth pressure from live loads are determined according to R 6 (Section 3.4) and R 7 (Section 3.5). Due to the differing partial safety factors for permanent and variable actions the earth pressure from unbounded distributed loads over and above $p_k = 10$ kN/m^2, and the earth pressure from strip loads q'_k and line loads $\overline{q}_k$, may not be superimposed on the earth pressure according to Paragraph 1. However, also see R 104, Paragraphs 3 and 5 (Section 4.11).

6. If the soldier piles are embedded sufficiently deep in the ground, the toe support can be adopted as follows:

   a) as a free earth support corresponding to R 14 (Section 5.3) or;
   b) as an fixed earth support or partial fixed earth support according to R 25 (Section 5.4).

   In the case of a free earth support in cohesive soil, a load model according to Figure R 12-1 c) is obtained.

7. Widely spaced bored pile walls are treated as soldier pile walls. However, a smaller earth pressure redistribution should generally be anticipated, corresponding to the ratio of the pile diameters to the pile spacing and depending on the stiffness of the piles. Refer to R 25 (Section 5.4) for details of earth restraints.

5.2 Pressure diagrams for supported soldier pile walls (R 69)

1. If:

   – the ground surface is horizontal;
   – medium-dense or dense, cohesionless soil or at least stiff, cohesive soil is present;
   – a non-yielding support according to R 67, Paragraph 3 (Section 1.5), is present and;
   – excavation does not proceed deeper than shown in Figure R 69-1 before the next row of struts or anchors is installed;

   soldier pile walls according to R 5, Paragraphs 3 and 4 (Section 3.3) in the advancing and fully excavated states may employ the pressure diagrams described below when adopting the active earth pressure from soil self-weight, unbounded distributed load $p_k \leq 10$ kN/m^2 and, if applicable, cohesion according to R 4 (Section 3.2). These pressure diagrams should only be seen as a guide; they do not exclude other realistic pressure diagrams. Also see [32, 52, 69, 89, 90].

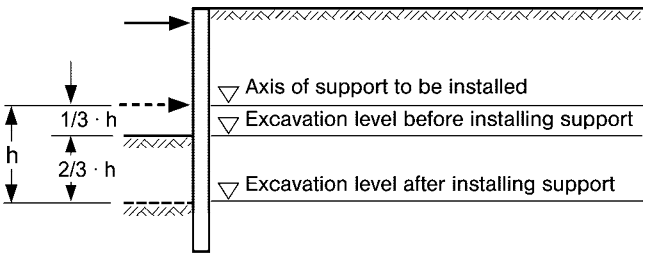


Figure R 69-1. Limit of excavation before installing support

2. The following pressure diagrams may be assumed as realistic for single-propped soldier pile walls:

   a) a continuous rectangle corresponding to Figure R 69-2 a), if the struts or anchors is not lower than $h_k = 0.10 \cdot H$;
   b) a stepped rectangle at halfway with $e_{ho,k}$: $e_{hu,k} = 1.50$ as shown in Figure R 69-2 b), if the struts or anchors are in the range $h_k > 0.10 \cdot H$ to $h_k = 0.20 \cdot H$;
   c) a stepped rectangle at halfway with $e_{ho,k}$: $e_{hu,k} = 2.00$ as shown in Figure R 69-2 c), if the struts or anchors are in the range $h_k > 0.20 \cdot H$ to $h_k = 0.30 \cdot H$.

A triangle with the largest ordinate at the height of the support, as shown in Figure R 5-1 i) (Section 3.3), is recommended as the realistic pressure diagram if $h_k > 0.30 \cdot H$.

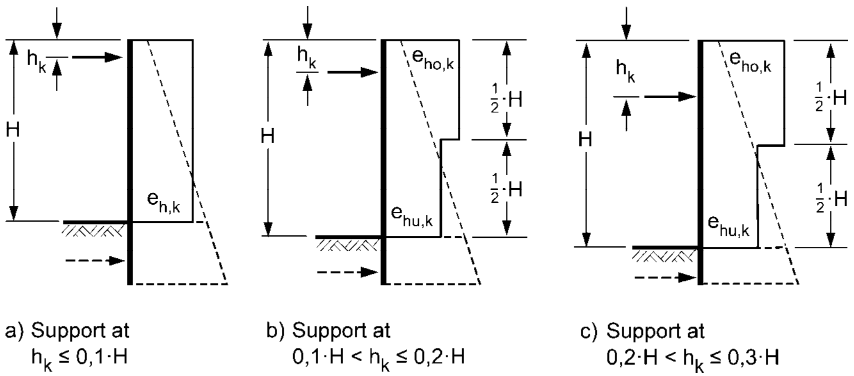


Figure R 69-2. Pressure diagrams for single-propped soldier pile walls

3. The following pressure diagrams may be regarded as realistic for double-propped soldier pile walls:
   a) a stepped rectangle with the load increment at the height of the lower row of struts or anchors and $e_{h0,k}$:$e_{hu,k}$ = 2.00 as shown in Figure R 69-3 a), if the upper row of struts or anchors is approximately at ground level and the lower row is in the upper half of the excavation height H;
   b) a trapezoid as shown in Figure R 69-3 b), if the upper row of struts or anchors is below ground level and the lower row is at approximately half of the height H of the excavation;
   c) a trapezoid as shown in Figure R 69-3 c), if both rows of struts or anchors are installed very low.

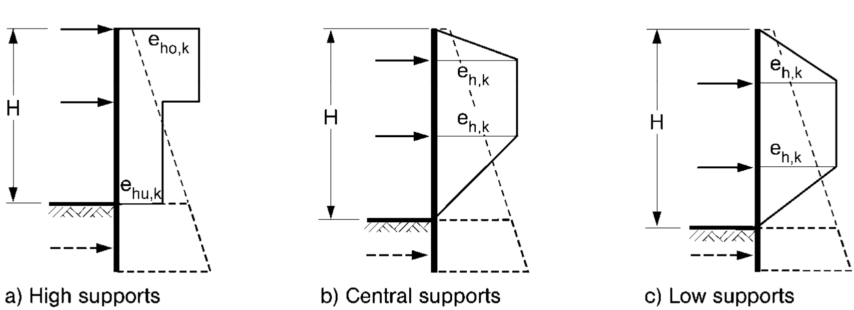


Figure R 69-3. Pressure diagrams for double-propped soldier pile walls

4. The trapezoid as shown in Figure R 69-4 may be regarded as a realistic pressure diagram for triple- or multiple-propped soldier pile walls with approximately the same spans. The earth pressure resultant should be in the range $z_e = 0.50 \cdot H$ to $z_e = 0.55 \cdot H$.

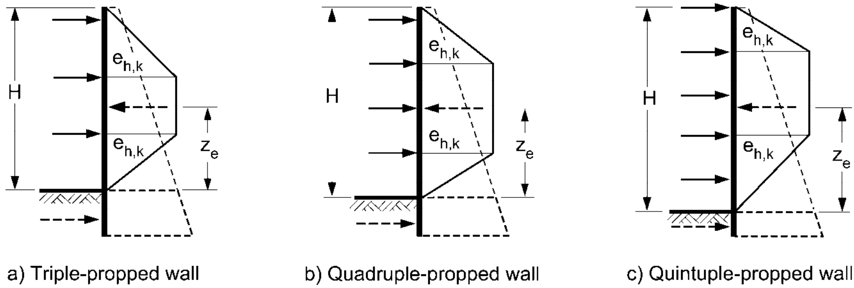


Figure R 69-4. Pressure diagrams for triple- or multiple-propped soldier pile walls

5. The pressure diagrams recommended here do not take the previous construction stage into consideration. More precise definitions take the pressure diagram of the previous construction stage and the earth pressure increase from the additional excavation phase into consideration for the pressure diagram of the current construction stage. This earth pressure increase acts primarily at the last installed support [89, 90]. This is particularly important in stratified ground. Supports that are lower than 30 % of the wall height H have no appreciable impact on the shape of the pressure diagram.

5.3 Soil reactions and passive earth pressure for soldier pile walls with free earth supports (R 14)

1. The characteristic passive earth pressure in front of soldier piles can be determined for cohesionless soils in line with the calculation procedure proposed in [20] and developed further in [52] and [168]. If the soldier piles are so closely spaced that the passive earth pressure influences overlap, the computed passive earth pressure forces shall be reduced accordingly. Passive earth pressure shall be determined with and without overlapping for this purpose. The smaller respective value then governs analysis. Also see [52]. If the analysis proposal derived for cohesionless soils is adopted for cohesive soils without additional studies, the proportion of the passive earth pressure resulting from cohesion shall be reduced to half the computed value. Also see [21] and [93].

2. The design passive earth pressure is obtained from the characteristic passive earth pressure given by the shear parameters φ'_k and c'_k by dividing by the partial safety factor $\gamma_{R,e}$ in accordance with R 79 (Section 2.4).

3. When applying the partial safety factors given in R 79 (Section 2.4) to determine the passive earth pressure design values for transferring the support force in the ground, considerable toe displacements shall generally be anticipated. Only if the design passive earth pressure is reduced using the calibration factor $\eta_{R,e} = 0.80$ may it be assumed for cohesionless soils and at least stiff, cohesive soils that the toe support displacements are of the same magnitude as the deflections and deformations of the remainder of the retaining wall. However, if it can be demonstrated that:

   a) the toe support deflections do not impair the serviceability of single-propped walls, or;
   b) for multiple-propped walls these deflections are not greater than the deflections and deformations of the rest of the retaining wall, e.g. in dense, cohesionless soils or very stiff, cohesive soils at the embedment depth;

   a calibration factor may be dispensed with when determining the embedment depth.

4. In approximation, a parabolic or bilinear approach as shown in Figures R 80-1 b) and c) or 80-1 a) (Section 4.3), with the centroid of the ground reaction at $z' = 0.60 \cdot t_0$, may be assumed for either cohesionless soil or at least stiff, cohesive soil when determining the action effects. If equilibrium of the horizontal forces according to R 15, Paragraph 1 (Section 5.5) can be demonstrated, a support at the centroid of the ground reaction may be assumed not only for stability analysis according to R 80, Paragraph 4 (Section 4.3), but also for serviceability analysis according to R 83, Paragraph 5 (Section 4.10). The cantilever moment described in R 11, Paragraph 3 a) (Section 4.2) does not generally occur in this case, if no earth pressure is applied to the soldier piles below the excavation level. The computed toe displacement may be corrected to $s = 0$ according to R 11, Paragraph 3 b) (Section 4.2).

5. If at least medium-dense, cohesionless or at least stiff, cohesive soil is present below the excavation level and a ground reaction increasing linearly with depth is adopted, determination of bending moments, shear forces and support forces according to R 82, Paragraph 1 b) (Section 4.4) may be based on:

   – either a reduced embedment depth t_0;
   – or a partial fixed earth support at depth t'_1 according to R 25, Paragraph 6 (Section 5.4).

The following stipulations apply:

a) The reduced embedment depth t_0 or the depth t'_1 may be determined or verified using the reduced partial safety factor $\gamma_{R,e,red} = 1.00$.
b) Any calibration factor $\eta_{R,e} = 0.80$ necessary according to Paragraph 3 remains unaffected.

6. If the serviceability according to R 11, Paragraph 4 (Section 4.2) is relevant, it may be necessary to take the displacement required to mobilise the ground reaction into consideration. To do this, the anticipated toe displacements may be estimated with the aid of the information given in [20], [93] and DIN 4085, or simple relationships between ground reaction and displacement derived. The resultant $E_{V,k}$ in Figure R 102-3 (Section 4.5) is obtained from the remaining earth pressure stresses in the excavated condition as shown in Figure R 102-1 (Section 4.5), taking preconsolidation into consideration, whereby the stresses are only applied to the actual pile width. If necessary, iteration shall be performed until the ground reaction and the deflection approximately match, or either the modulus of subgrade reaction method in accordance with R 102 (Section 4.5) or the finite element method in accordance with R 103 (Section 4.6) are adopted.

5.4 Fixed earth support for soldier pile walls (R 25)

1. If the soldier piles of a soldier pile wall embed deeply enough in the ground below the excavation level, a fixed earth support can be adopted for determination of action effects. This fixed earth support of the soldier pile can be identified with the aid of the *Blum* approach [23]. Supported and unsupported soldier pile walls are differentiated:

   a) For unsupported walls in load-bearing ground the full geotechnical fixed earth support always occurs, because the soldier piles may rotate around a point above the wall toe until equilibrium is achieved.
   b) For supported walls the degree of fixed earth support depends on the deformation behaviour of the soldier piles and the ground. In this case, a full geotechnical fixed earth support assumes that neither displacement nor rotation occurs at the theoretical toe.

   Generally, the soldier pile sections of supported walls are sufficiently flexible, so that a full fixed earth support forms in at least medium-dense, cohesionless soils and at least stiff, cohesive soils. Only under certain circumstances for very stiff sections and small support spans, may the backward rotation of the wall required for mobilisation of the equivalent force C below the theoretical toe not occur or may only partially occur.

2. The magnitude of the passive earth pressure in front of the soldier piles can be determined according to R 14 (Section 5.3). It is generally appropriate to distribute the effective passive earth pressure in front of the individual soldier piles uniformly across the whole length of the retaining wall being investigated in order to apply the analysis methods derived for steel sheet pile walls. The result is the same passive earth pressure as in front of a steel sheet pile wall if the failure bodies in front of the individual soldier piles overlap and the wall friction angle is adopted at $\delta_{p,k} = 0$. In all other cases the passive earth pressure in front of a row of soldier piles determined using the negative angle of inclination is smaller than the passive earth pressure in front of a steel sheet pile wall [19, 20].

3. If the passive earth pressure failure bodies in cohesionless soils in front of the individual soldier piles do not overlap, the result in the failure state is a parabolic ground reaction increasing with depth [68] based on the analysis proposal given in [20]. The ensuing distribution diagram can be transformed to an equal area triangle. The error resulting from the displacement of the resultant shall be compensated for, in approximation, by a reduction of the computed passive earth pressure by 15 %, if no more precise analysis is performed. For cohesive soils the computed passive earth pressure may be increased by up to 10 %, if the passive earth pressure failure bodies in front of the individual soldier piles overlap [123].

4. A load model as shown in Figure R 25-1 b) results for unsupported soldier pile walls with fixed earth support. When analysing the embedment depth the passive earth pressure design value shall be determined using the partial safety factors according to R 79 (Section 2.4). If the head deflections anticipated using this approach are incompatible with sensitive facilities behind the wall, e.g. pipelines, road pavements, masts or railway facilities, a greater embedment depth shall be selected, thus reducing utilisation of the ground reaction, or a stronger soldier pile than determined by calculations selected. This is particularly the case if loose, cohesionless soils or only nearly stiff, cohesive soils are present in the area of the fixed earth support. For soldier pile walls close to foundation loads and for excavations in soft, cohesive soils, an unsupported wall with only a fixed earth support is generally not permissible due to the large anticipated deformations, see R 20 (Section 9.1) or R 92, Paragraph 1 (Section 12.3).

5. A load model as shown in Figure R 25-2 b) results for supported soldier pile walls. For medium-dense or dense, cohesionless soil, or at least stiff, cohesive soil, it may generally be accepted that the deformation conditions associated with a full fixed earth support after *Blum* are approximately fulfilled if the passive earth pressure design value was determined using the partial safety factors according to R 79 (Section 2.4) when verifying the

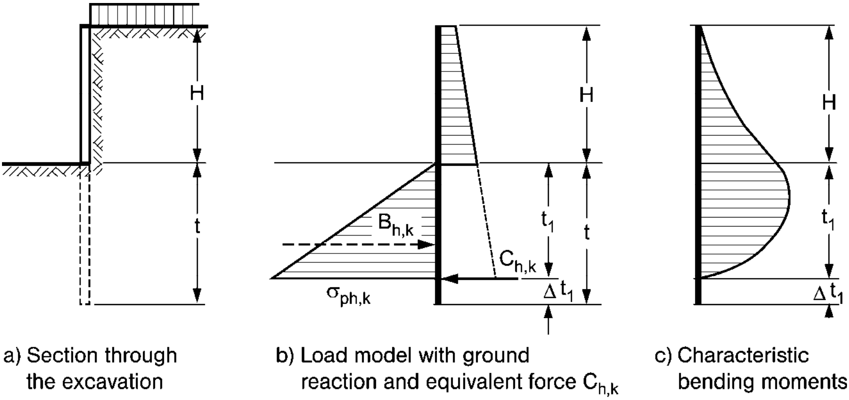


Figure R 25-1. System, loading and moment distribution for an unsupported soldier pile wall with fixed earth support

embedment depth. For loose soils and for stiff soldier piles the dissimilar deformation behaviour of soldier piles and the ground can be taken into consideration in the analysis by introducing a suitable passive earth pressure reduction using a calibration factor $\eta_{R,e} = 0.80$ according to R 14 (Section 5.3). A fixed earth support effect may not generally be applied for soft, cohesive soils or soils with high organic content, see R 96 (Section 12.7).

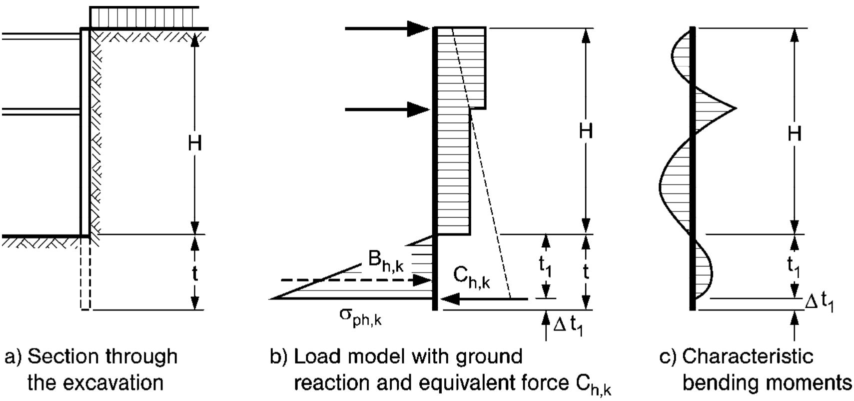


Figure R 25-2. System, loading and moment distribution for a double-propped soldier pile wall with fixed earth support

6. Intermediate cases with partial fixed earth support are possible between the limit cases of full fixed earth support and free earth support, and may be

adopted for supported soldier pile walls with an embedment depth $t'_1 < t_1$. In this case there are no restrictions on the angle of the end tangent. Also see R 80, Paragraph 5 b) (Section 4.3).

7. The embedment depth t_1, which is required for a fixed earth support of an unsupported soldier pile wall as shown in Figure R 25-1, shall generally be increased by at least $\Delta t_1 = 0.20 \cdot t_1$ in order to accept the structurally required equivalent design force $C_{h,d}$. The same applies to supported soldier pile walls as shown in Figure R 25-2, if the full fixed earth support can develop in the ground. In approximation, for a partial fixed earth support the surcharge Δt_1 may be linearly interpolated between the governing full fixed earth support value Δt_1 and the free earth support value $\Delta t_1 = 0$ as a function of the ratio $t'_1 : t_1$.
8. If necessary, determination of the fixed earth support can also be based on subgrade reaction using a deformation resistance. Also see R 102 (Section 4.5).
9. If at least medium-dense, cohesionless or at least stiff, cohesive soil is present below the excavation level, determination of bending moments, shear forces and support forces according to R 82, Paragraph 1 b) (Section 4.4) may be based on:
   – either a reduced embedment depth t_1;
   – or an increased partial fixed earth support at the specified depth t'_1.

   The following stipulations apply:

   a) The reduced embedment depth t_1 or the depth t'_1 may be determined or verified using the reduced partial safety factor $\gamma_{R,e,red} = 1.00$.
   b) Any calibration factor $\eta_{R,e} = 0.80$ necessary according to Paragraph 5 remains unaffected.
10. Analysis of the vertical component of the support force B_k shall be performed according to R 9 (Section 4.7), that of the transfer of vertical forces to the subsurface according to R 84 (Section 4.8).

5.5 Equilibrium of horizontal forces for soldier pile walls (R 15)

1. The characteristic earth pressure $\Delta E_{ah,k}$ below the excavation level may be neglected for analysis of the embedment depth and for determination of the action effects of soldier pile walls, if it can be demonstrated that the design value of the earth pressure $\Delta E_{ah,d}$, together with the design support force $B_{h,d}$ from the soldier piles, is completely transferred by the available passive earth pressure design value $E_{ph,d}$:

   $B_{h,d} + \Delta E_{ah,d} \leq E_{ph,d}$.

This analysis shall be regarded as a supplement to analysis of the embedment depth. The design earth pressure and the design passive earth pressure are obtained from the characteristic variables using the shear parameters φ'_k and c'_k by multiplying by the partial safety factors γ_G and γ_Q, and by dividing by the partial safety factor $\gamma_{R,e}$ according to R 79 (Section 2.4). The calibration factor $\eta_{R,e} = 0.60$, as specified in R 22, Paragraph 6 (Section 9.5), shall be taken into consideration for excavations adjacent to buildings.

Analysis may only be dispensed with if the preconditions laid out in Paragraph 9 are fulfilled.

2. The magnitude of the neglected characteristic earth pressure $\Delta E_{ah,k}$ for soldier piles with a free earth support is obtained from the difference introduced into the analysis between the earth pressure to the soldier pile toe and the earth pressure to the excavation level. In the case of cohesive soil layers, determination of the neglected earth pressure shall be according to R 4, Paragraph 3 a), and R 4, Paragraph 3 b) (both Section 3.2). The larger is the governing value. The theoretical support point for soldier piles with fixed earth support replaces the actual toe point of the soldier pile.

3. The magnitude of the characteristic soldier pile support force is obtained directly from analysis of the embedment depth of the soldier piles for walls with a free earth support. The characteristic support force for walls with fixed earth support is equal to the mathematically required ground reaction from the excavation level to the theoretical toe based on *Blum's* load approach; see Figures R 25-1 and R 25-2 (Section 5.4). However, the support force $B_{h,k}$ determined on the basis of *Blum's* load approach may be approximately reduced by half of the computed equivalent force $C_{h,k}$, as shown in Figure R 9-1 (Section 4.7), considering the magnitude of the actual anticipated ground reaction required for soldier pile with **fixed earth support**. If applicable, R 25, Paragraph 8 (Section 5.4) shall be taken into consideration when adopting a subgrade reaction support.

4. The characteristic passive earth pressure may be determined using the wall friction angle $\delta_{p,k} = -\varphi_k$, if based on curved or non-circular slip surfaces. Also see R 19, Paragraph 1 (Section 6.3).

5. If analysis using the selected embedment depth according to Paragraph 1 is not possible for unsupported soldier pile walls with fixed earth support then either:

   a) the embedment depth shall be increased or;
   b) the soldier pile wall shall be treated as a steel sheet pile wall.

6. If analysis using the initially selected embedment depth and the earth pressure adopted according to Paragraph 1 (Figure R 15-1 a) is not possible for supported soldier pile walls with a free earth support, then either:

   a) the embedment depth shall be increased (Figure R 15-1 b) or;
   b) mathematical embedment shall be dispensed with (Figure R 15-l c) or;
   c) the complete earth pressure from the surface to the base of the soldier pile shall be incorporated in the redistribution (Figure R 15-1 d).

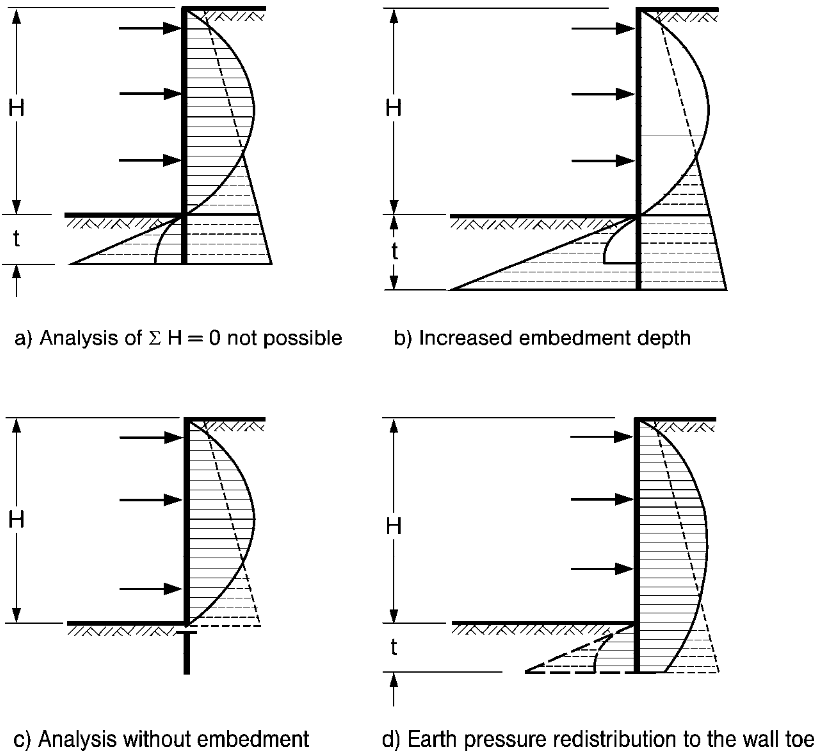


Figure R 15-1. Analysis of Σ H = 0 for soldier pile walls

7. If analysis using the selected embedment depth according to Paragraph 1 is not possible for supported soldier pile walls with fixed earth support then either:

   a) the embedment depth shall be increased or;
   b) full fixed earth support shall be dispensed with and analysis performed with a partial fixed earth support or with a free earth support, or;

c) the complete earth pressure from the surface to the theoretical toe shall be incorporated in the redistribution, or;
d) the soldier pile wall shall be treated as a steel sheet pile wall.

8. The following additional analyses are required for the solutions discussed in Paragraphs 5, 6 and 7:

a) If the embedment depth is increased according to Paragraph 5 a), Paragraph 6 a) or Paragraph 7 a), analysis according to Paragraph 1 shall be performed again. Renewed determination of the action effects is not necessary, see Figure R 15-1 b).
b) For a soldier pile wall without sufficient embedment according to Paragraph 6 b), it shall be demonstrated that the soldier piles and struts or anchors are capable of transferring the horizontally acting earth pressure forces above the excavation level without soldier pile embedment. Analysis according to Paragraph 1 is dispensed with. An upward and a downward vault effect is then assumed in the region in which the passive earth pressure is insufficient to transfer the active earth pressure. This is the case for cohesionless soils in particular. However, also see R 10, Paragraph 1 (Section 4.9).
c) The action effects shall be determined again for earth pressure redistribution from ground level to the toe according to Paragraph 6 c) or to the theoretical toe according to Paragraph 7 c). Here, only that portion of the earth pressure diagram above the excavation level need be adopted. However, the component lying below the excavation level shall be taken into consideration for analysis according to Paragraph 1, which shall be renewed for the altered conditions.
d) If a greater embedment depth than that ascertained in the original analysis is found for a projected steel sheet pile wall during the additional investigation according to Section 5 b) or Section 7 d), the larger is the governing value. A renewed analysis according to Paragraph 1 is unnecessary. Renewed determination of the action effects is also unnecessary.

9. Analysis according to Paragraph 1 can be dispensed with if, simultaneously:

- cohesionless soil with a friction angle $\varphi'_k \geq 32.5°$ is present below the excavation level and possesses approximately the same self-weight as the soil above the excavation level;
- no earth pressure from building loads needs to be taken into consideration below the excavation level;
- the embedment depth of the soldier piles is not less than one quarter of the excavation depth;
- the width of the soldier piles is not more than one fifth of the soldier pile centres and;

- the passive earth pressure in front of the soldier piles can be determined by applying a negative wall friction angle, given the prevalent conditions.

10. If a layer of loose, cohesionless soil is present below the excavation level, additional investigations shall be carried out to determine the deformation behaviour of the wall and the ground. Instead of this, it may also be expedient to reduce the support point centres and to dispense with the computed toe support. The anticipated ground reaction in the soldier pile embedment zone below the excavation level may be treated as an action.

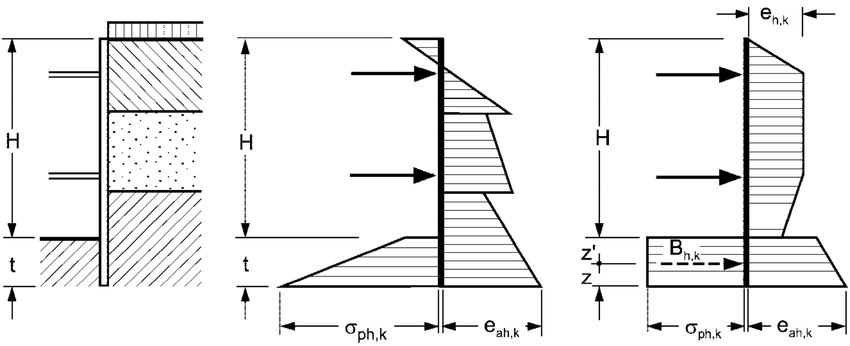
6 Analysis approaches for sheet pile walls and in-situ concrete walls

6.1 Determination of load models for sheet pile walls and in-situ concrete walls (R 16)

1. If the conditions for reducing the earth pressure from the at-rest earth pressure to the active earth pressure given in R 8 (Section 3.1) are met, the earth pressure load E_a according to R 4 (Section 3.2) and R 6 (Section 3.4) shall be determined down to the excavation level using classical earth pressure theory and adopting characteristic soil properties, taking into consideration soil self-weight, unbounded distributed load $p_k \leq 10$ kN/m^2 and, if applicable, cohesion:

   – from the ground surface to the base of the wall for walls with a free earth support;
   – from the ground surface to the theoretical toe for walls with a fixed earth support.

   Figure R 16-1 shows the procedural principle, without consideration of earth pressure from other temporary loads.



a) Section through the excavation b) Classical distribution of earth pressure and ground reaction c) Load model for a pressure diagram according to Figure R 70-2b)

Figure R 16-1. Load model determination for sheet pile walls and in-situ concrete walls with active earth pressure and free earth support

2. For unsupported sheet pile walls and in-situ concrete walls (see R 67, Section 1.5) with a fixed earth support, and for flexibly supported sheet pile

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walls and in-situ concrete walls, the earth pressure distribution determined according to Paragraph 1 shall always be adopted for analysis of embedment depth according to R 80 (Section 4.3) and for determination of the action effects according to R 82 (Section 4.4). When investigating forced slip surfaces, the lower starting point is generally assumed to be at the height of the base of the wall or the theoretical toe.

3. For moderately flexibly supported sheet pile walls and in-situ concrete walls, the earth pressure load determined according to Paragraph 1 (Figure R 16-1 b) shall be converted to a simple pressure diagram according to R 5 (Section 3.3), corresponding to the anticipated earth pressure redistribution. It is generally sufficient to limit the earth pressure redistribution to the region of the height of the wall H extending from ground level to the excavation level. If there are reasons for anticipating upward earth pressure redistribution from the region below the excavationlevel, or if such a redistribution is favoured by structural measures, it may be expedient to extend the earth pressure redistribution to the height $H' > H$ according to R 5, Paragraph 3 c) (Section 3.3), in the most extreme case to the base of the wall or the theoretical toe. With regard to whether the wall has a free earth support or with a fixed earth support, differentiation is only required when defining the pressure diagram if no displacement is anticipated at the theoretical toe.

4. The information given in Recommendation R 70 (Section 6.2) can be used as a guide for specifying a realistic pressure diagram for moderately flexibly supported sheet pile walls and in-situ concrete walls. A rectangular pressure diagram cannot be regarded as realistic in the majority of cases. If one is adopted the errors associated with this procedure when determining the shear and support forces shall be corrected by applying suitable surcharges. Also see R 17 in the 3rd edition of these Recommendations [124] and R 100 [125].

5. The magnitude and distribution of earth pressure from temporary loads are determined according to R 6 (Section 3.4) and R 7 (Section 3.5). Due to the differing partial safety factors for permanent and temporary actions the earth pressure from unbounded distributed loads over $p_k = 10$ kN/m^2, and the earth pressure from strip loads q'_k and line loads $\overline{q}_k$, may not be superimposed on the earth pressure according to Paragraph 1. However, also see R 104, Paragraph 3 (Section 4.11).

6. The toe support can be adopted as a function of the selected embedment depth and the stiffness of the wall as follows:

   a) as a free earth support corresponding to R 19 (Section 6.3) or;
   b) as a fully fixed end support or partial fixed end support according to R 26 (Section 6.4).

In the case of a free earth support in cohesive soil a load model as shown in Figure R 16-1 c) is obtained.

6.2 Pressure diagrams for supported sheet pile walls and in-situ concrete walls (R 70)

1. If:

– the ground surface is horizontal;
– medium-dense or densely compacted, cohesionless soil or at least stiff, cohesive soil is present;
– an inflexible support according to R 67, Paragraph 3 (Section 1.5), is present and;
– excavation does not proceed deeper than shown in Figure 69-1 (Section 5.2) before the next row of struts or anchors is installed;

sheet pile walls and in-situ concrete walls according to R 5, Paragraphs 3 and 4 (Section 3.3) may employ the pressure diagrams described below in the advancing and fully excavated states when adopting the active earth pressure from soil self-weight, unbounded distributed load $p_k \leq 10\ kN/m^2$ and, if applicable, cohesion according to R 4 (Section 3.2). These pressure diagrams should be regarded as a guide only; they do not exclude other realistic pressure diagrams. Also see [52].

The pressure diagrams proposed below assume earth pressure redistribution from the ground surface to the excavationlevel. The classical earth pressure distribution, increasing with depth, remains unchanged from the excavation level to the wall toe.

2. The following pressure diagrams may be regarded as realistic for single-propped sheet pile walls and in-situ concrete walls:

a) a continuous rectangle corresponding to Figure R 70-1 a), if the set of struts or anchors is not lower than $h_k = 0.10 \cdot H$;
b) a stepped rectangle with $e_{ho,k}:e_{hu,k} = 1.20$ as shown in Figure R 70-1 b), if the struts or anchors are in the range $h_k > 0.10 \cdot H$ to $h_k = 0.20 \cdot H$;
b) a stepped rectangle with $e_{ho,k}:e_{hu,k} = 1.50$ as shown in Figure R 70-1 b), if the struts or anchors are in the range $h_k > 0.20 \cdot H$ to $h_k = 0.30 \cdot H$.

Where $h_k > 0.30 \cdot H$ a rectangle is recommended as the realistic pressure diagram as shown in Figure R 5-1 k) (Section 3.3), with the largest ordinate at the height of the support.

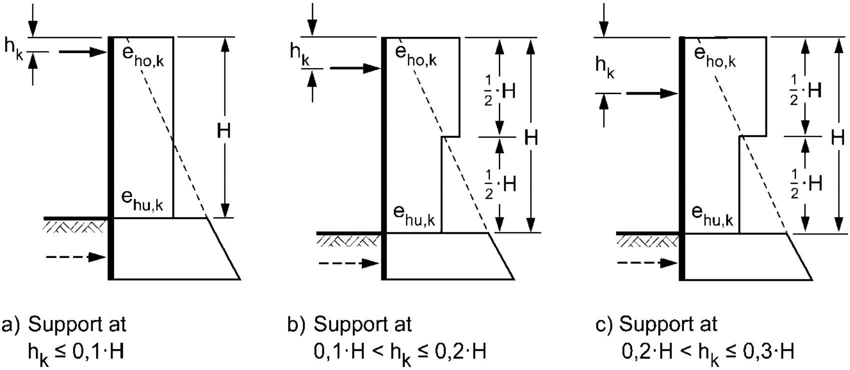


Figure R 70-1. Pressure diagrams for single-proppedsheet pile walls and in-situ concrete walls

3. The following pressure diagrams may be regarded as realistic for double-propped sheet pile walls and in-situ concrete walls:

   a) a stepped rectangle with the load increment at the height of the lower row of struts and an ordinate ratio $e_{ho,k}$:$e_{hu,k}$ = 1.50 as shown in Figure R 70-2 a), if the upper row of struts or anchors is approximately at ground level and the lower row is in the upper half of the height H;
   b) a quadrangular pressure diagram $e_{ho,k}$:$e_{hu,k}$ = 2.00 as shown in Figure R 70-2 b), if the upper row of struts or anchors is approximately at ground level and the lower row approximately at half of the height H;
   c) a tapered rectangle corresponding to Figure R 70-2 c), if both rows of struts or anchors are installed very low.

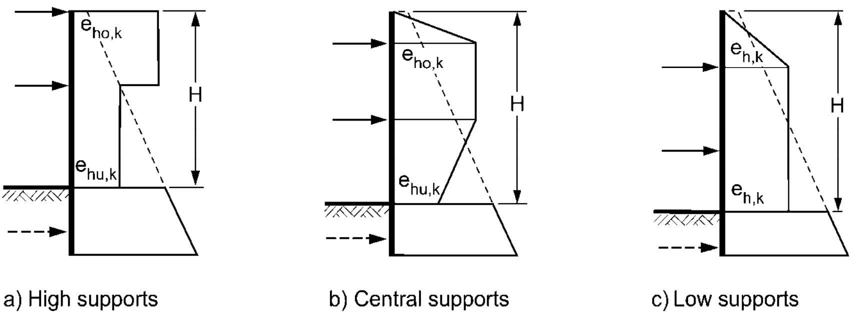
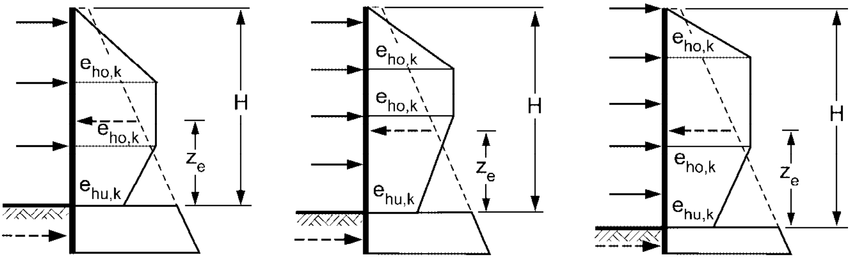


Figure R 70-2. Pressure diagrams for double-propped sheet pile walls and in-situ concrete walls

4. The pressure diagram according to *Lehmann* as shown in Figure R 70-3 may be regarded as realistic for triple- or multiple-propped sheet pile walls or in-situ concrete walls, but only when the bending points are at the height of the support points and in the ratio $e_{ho,k}$:$e_{hu,k}$ = 2.00. The resultant of the mathematical load should be in the range $z_e = 0.40 \cdot H$ to $z_e = 0.50 \cdot H$.



a) Triple-propped wall b) Quadruple-propped wall c) Quintuple-propped wall

Figure R 70-3. Pressure diagrams for triple- and multiple propped sheet pile walls and in-situ concrete walls

5. The pressure diagrams recommended here do not take the previous construction stage into consideration. More precise definitions take the pressure diagram of the previous construction stage and the earth pressure increase from the additional excavation phase into consideration for the pressure diagram of the current construction stage. This earth pressure increase acts primarily at the last installed support [89, 90]. This is particularly important in stratified ground. Supports that are lower than 30 % of the wall height H have no appreciable impact on the shape of the pressure diagram for double- or multiple-propped walls.

6. If upward earth pressure redistribution from the region below the excavation level is anticipated, or if it is favoured by structural measures, the pressure diagram shall be specified corresponding to the stiffness of the wall, the anticipated displacement of the wall toe and the strut prestressing.

6.3 Ground reactions and passive earth pressure for sheet pile walls and in-situ concrete walls with free earth support(R 19)

1. If the $V_k \geq B_{v,k}$ condition in accordance with R 9, Paragraph 1 (Section 4.7) and the relative movement between retaining wall and soil permit, the characteristic passive earth pressure may be determined as follows for sheet pile walls and pile walls with free earth supports:

a) The angle of inclination may be adopted at $\delta_{p,k} = -\varphi'_k$, if curved slip surfaces after *Caquot, Kerisel* and *Absi* [70] or according to DIN 4085, or non-circular slip surfaces based on the approach after *Weißenbach* [71] and *Mao* [91], modified after *Streck*, are used as the basis for analysis.
b) Planar slip surfaces may only be adopted as the basis for analysis if the ground surface does not rise, the friction angle is not greater than $\varphi'_k = 35°$ and the angle of inclination is reduced from $\delta_{p,k} = -\varphi'_k$ to $\delta_{p,k} = -{}^2/_3 \cdot \varphi'_k$ for sheet pile walls and pile walls. In the case of diaphragm walls a smaller inclination angle shall be adopted according to R 89, Paragraph 3 (Section 2.3), generally $\delta_{p,k} = -½ \cdot \varphi'_k$.

See Figure R 19-1 for sign definitions.

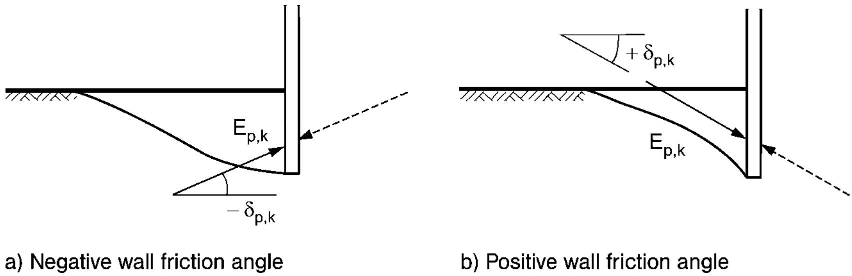


Figure R 19-1. Sign rule for the passive earth pressure inclination angle

2. The design passive earth pressure $E_{ph,d}$ is obtained from the shear parameters φ'_k and c'_k determined using the characteristic passive earth pressure by dividing by the partial safety factor $\gamma_{R,e}$ according to R 79 (Section 2.4).

   It shall be demonstrated that:

   $\Sigma\ B_{h,d} \leq E_{ph,d}$.

   $\Sigma\ B_{h,d}$ is the design value of the characteristic support force components from permanent or temporary actions multiplied by the partial safety factors γ_G or γ_Q.

3. When applying the partial safety factors given in R 79 (Section 2.4) to determine the passive earth pressure design values for accepting the earth support in the ground, it may be assumed that the displacements of the toe support are of the same magnitude as the deflections and deformations of the remainder of the retaining wall in cohesionless soils and at least stiff, cohesive soils. See R 96 (Section 12.7) for details of displacements in soft, cohesive soils.

4. The point of acting of the resultant characteristic support force $B_{h,k} = B_{h,G,K} + B_{h,Q,K}$ from the ground reaction $\sigma_{ph,k}$ for a wall with a free earth support may be assumed to be $z' = 0.6 \cdot t$ in the case of cohesionless soil and $z' = 0.50 \cdot t$ in the case of at least stiff, cohesive soil, if the errors associated with this described in R 11, Paragraph 3 (Section 4.2) are acceptable. In one case, this corresponds to a parabolic or bilinear distribution as described in R 80 (Section 4.3), Figure R 80-1 a) or b), in the other case a rectangular distribution as shown in Figure R 80-1 c). Otherwise, the ground reaction $\sigma_{ph,k}$ is used for analysis.

5. If at least medium-dense, cohesionless soil or at least stiff, cohesive soil is present below the excavation level and a ground reaction distribution increasing linearly with depth is selected, determination of the wall effects, i.e. the bending moments, shear forces, normal forces and support forces may be based on:

   – either a reduced embedment depth t_0 or;
   – a partial fixed end support at depth t'_1 according to R 26, Paragraph 5 (Section 6.4).

   This reduced embedment depth t_0, or the depth t'_1, may be determined or verified according to R 82, Paragraph 1 b) (Section 4.4) using the reduced partial safety factor $\gamma_{R,e,red} = 1.00$.

6. If the serviceability according to R 11, Paragraph 4 (Section 4.2) is relevant, it may be necessary to take the displacement required to mobilise the ground reaction into consideration. The anticipated toe displacements can be estimated with the aid of [94, 95], [126] and DIN 4085. The earth pressure stresses $e_{0g,k}$ remaining from preconsolidation may be taken into consideration as shown in Figure R 102-2 (Section 4.5). Iteration shall be performed where necessary, until ground reaction and displacement match.

7. Generally, the same passive earth pressure as for closed walls may be applied for sheet pile walls and pile walls with staggered toes. However, without analysis, only every second double section or every second pile may be shortened by 20 % of the necessary computed embedment depth t, but by a maximum of 1.0 m. If such shortening is performed on the master (bearing) pile of combined sheet pile walls or on the reinforced piles of a pile wall manufactured using alternating reinforced and unreinforced piles, it shall be demonstrated that the wall can accept the effects and that the passive earth pressure can accept the earth support force.

6.4 Fixed earth support for sheet pile walls and in-situ concrete walls (R 26)

1. Under certain circumstances, if a sheet pile wall or an in-situ concrete wall embeds deeply enough below the excavationlevel, a geotechnical fully fixed earth support can be applied to determine the action effects. This fully fixed earth support can be identified with the aid of the *Blum* approach [23]. Supported and unsupported walls are differentiated:

   a) For unsupported walls in load-bearing ground the full geotechnical fixed earth support always occurs, because the wall may rotate around a point above the wall toe until equilibrium is achieved.
   b) For supported walls the degree of fixed earth support depends on the deformation behaviour of the wall and the ground. In this case, a full geotechnical fixed earth support assumes that neither displacement nor rotation occurs at the theoretical toe.

   Generally, the sheet pile sections of supported walls are sufficiently flexible, so that a full geotechnical fixed earth support occurs in the ground in at least medium-dense, cohesionless soils and at least stiff, cohesive soils. Under certain circumstances only, for very stiff sections and small support spans, the backward rotation of the wall toe required for mobilisation of the equivalent force C may not occur or only occur partially. For supported in-situ concrete walls in unconsolidated rock, a geotechnical fixed end support may only be adopted if the wall support is highly flexible.

2. A load model as shown in Figure R 26-1 b) is obtained for unsupported sheet pile walls and in-situ concrete walls with fully fixed earth support in

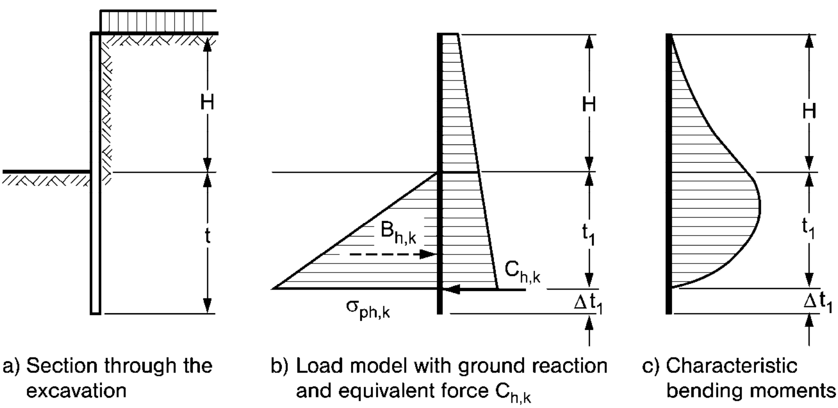


Figure R 26-1. System, load and moment distribution for an unsupported soldier pile wall or in-situ concrete wall fixed in the ground the ground. The passive earth pressure design value is determined according to R 19 (Section 6.3). If the deflections at the top of the wall anticipated using this approach give cause for doubt, e.g. with regard to damage to pipelines or road pavements, danger to road or rail traffic, or with regard to restrictions in the projected workspace, a greater embedment depth should be adopted, thus reducing utilisation of the ground reaction. If necessary, a stronger section than determined by calculations shall also be adopted. This is particularly the case if loosely compacted, cohesionless soil or only nearly stiff, cohesive soil is present in the embedded area. If necessary, serviceability shall be analysed again according to R 83 (Section 4.10) using the new dimensions. For retaining walls close to foundation forces and for excavations in soft, cohesive soils a cantilever wall with only a fully fixed earth support is generally not permissible due to the large anticipated deformations, see R 20 (Section 9.1) or R 101 (Section 12.12).

3. A load model as shown in Figure R 26-2 b) is obtained for supported sheet pile walls. It may generally be accepted for medium-dense or densely compacted, cohesionless soil, or at least stiff, cohesive soil, that the deformation conditions associated with a full fixed earth support after *Blum* are approximately fulfilled. For loosely compacted, cohesionless soils and for very stiff sheet pile walls the dissimilar deformation behaviour of the wall and the ground can be taken into consideration in the analysis by introducing a suitable passive earth pressure reduction using a calibration factor $\eta_{Ep} < 1$. A fixed earth support effect may not generally be applied for soft, cohesive soils or soils with high organic content, see R 96 (Section 12.7).

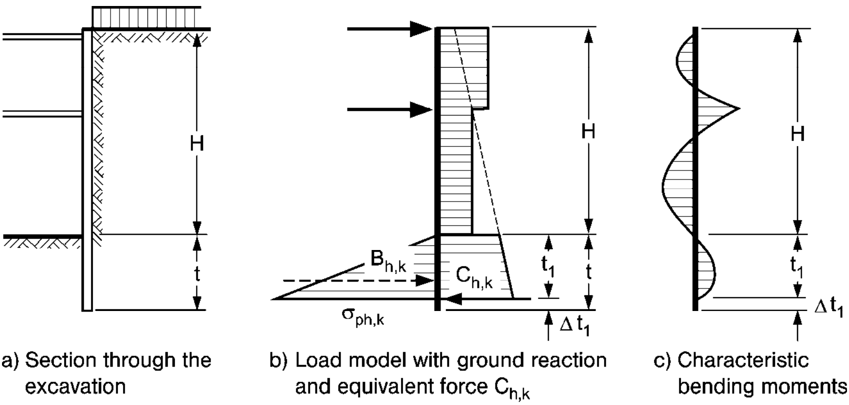


Figure R 26-2. System, load and moment distribution for a double-propped sheet pile wall fixed in the ground

4. Intermediate cases with partial fixed earth support are possible between the limit cases of fully fixed and free earth support, and may be adopted for supported sheet pile walls with an embedment depth $t'_1 < t_1$. In this case there are no restrictions on the angle of the end tangent. Also see R 80, Paragraph 5 b) (Section 4.3).

5. The embedment depth t_{-1}, which is required for the fully fixed earth support of an unsupported sheet pile wall or in-situ concrete wall as shown in Figure R 261, shall generally be increased without analysis by at least $\Delta t_1 = 0.20 \cdot t_1$ in order to accept the structurally required equivalent force $C_{h,d}$. However, if a more precise analysis is performed a surcharge of at le-ast $\Delta t_1 = 0.10 \cdot t_1$ is required. The same applies to supported sheet pile walls as shown in Figure R 26-2, if the fully fixed end support can develop in the ground. In approximation, the embedment depth $\Delta t'_1$ for partial fixed earth support may be linearly interpolated between the governing fully fixed earth support value Δt_1 and the free earth support value $\Delta t_1 = 0$ as a function of the ratio $t'_1 : t_1$.

6. The more precise analysis stipulated in Paragraph 5 can be based on *Lack-ner* [2, 24] and the following is obtained as shown in Figure R 26-3:

   $\Delta t_1 \geq C_{h,d}/2 \cdot e_{phC,d}$.

   Where:

   $C_{h,d} = C_{Gh,k} \cdot \gamma_G + C_{Qh,k} \cdot \gamma_Q$

   $e_{phC,d} = e_{phC,k}/\gamma_{R,e}$

   $e_{phC,k} = (g_k + p_k) \cdot K_{pghC} + c'_k \cdot K_{pch}$.

   The following shall be observed:

   a) The vertical stress g_k at the level of the theoretical toe is determined from the weight of the overlying strata, if necessary taking uplift into consideration.
   b) The value and sign of the angle of inclination $\delta_{C,k}$ are obtained from analysis of the vertical component of the mobilised passive earth pres-sure in accordance with R 9, Paragraph 3 b) (Section 4.7). The angle of inclination is generally restricted to $\delta_{C,k} \leq ⅓ \cdot \varphi'_k$.

   Note: The design value $C_{h,d}$ is determined here from the characteristic values $C_{Gh,k}$ and $C_{Qh,k}$, which are mathematically obtained from *Blum's* load approach and from $\Sigma H_k = 0$ of all characteristic actions and support forces.

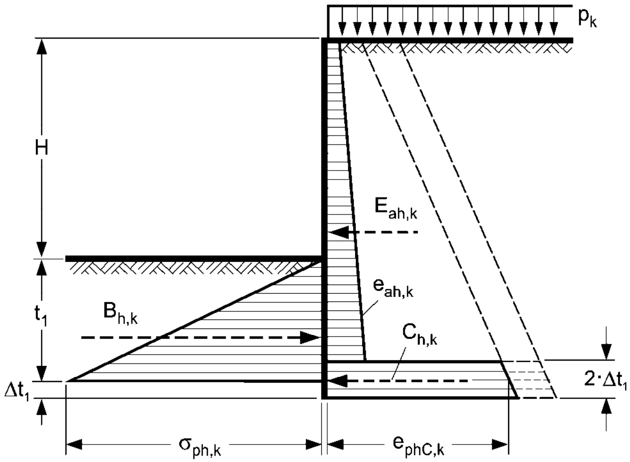


Figure R 26-3. Transfer of force $C_{h,k}$ at the toe of a wall fixed in the ground after *Lackner*

7. If necessary, determination of the degree of fixed earth support can also be based on elastic bedding using a deformation resistance. Also see R 102 (Section 4.5).

8. If at least medium-dense, cohesionless soil or at least stiff, cohesive soil is present below the excavationlevel, determination of bending moments, shear forces and support forces may be based on:

   – either a reduced embedment depth t_1 or;
   – an increased partial fixed earth support at the specified depth t'_1.

   This reduced embedment depth t_1 or the depth t'_1 may be determined or verified according to R 82, Paragraph 1 b) (Section 4.4) using the reduced partial safety factor $\gamma_{R,e,red} = 1.00$.

9. Verification of the vertical component of the mobilised passive earth pressure shall be performed according to R 9 (Section 4.7), that of the vertical failure of embedded walls according to R 84 (Section 4.8).

10. See R 19, Paragraph 7 (Section 6.3) for staggering sheet pile walls and pile walls.

7 Anchored retaining walls

7.1 Magnitude and distribution of earth pressure for anchored retaining walls (R 42)

1. The magnitude and distribution of the earth pressure on anchored retaining walls depend in principle on whether anchors are prestressed and, if so, with what force they are prestressed and locked off. Earth pressure distribution deviating from the classical earth pressure, e.g. the pressure diagram as shown in Figure R 5-1 (Section 3.3), is generally only obtained if the anchors are prestressed to at least 80 % of the active earth pressure design value or 100 % for pressures higher than the active earth pressure for the characteristic effects E_k, computed for the respective subsequent construction stage. When prestressing for substantially lower forces, earth pressure distribution is predominantly dependent on the interaction of local factors such as live loads, building loads, soil type, wall stiffness, length of and strain on the anchors, and flexibility of the toe support, and can no longer be determined with sufficient precision.
2. An earth pressure distribution of choice can be imposed, within certain limits, by the appropriate configuration and prestressing of the anchors, in particular depending on the stiffness of the retaining wall. If a large upward redistribution of earth pressure needs to be achieved, e.g. a pressure diagram with the resultant in the upper half of the excavation, it is also necessary to design the upper anchors longer than the lower ones for retaining walls with more than one row of anchors. Otherwise, the length of the anchors depends on the stability at the lower failure plane according to R 44 (Section 7.3), general overall stability according to R 45 (Section 7.4) and, if necessary, on the results of the investigation of possible wall deflections according to R 46 (Section 7.5).
3. In exceptional cases, anchor configuration, anchor length and the degree of prestressing can be selected in such a way that a flexible supported wall ensues and the classical earth pressure distribution can be adopted, at least for relatively stiff walls. With regard to include cohesion and to investigate the influence of live loads, the same considerations apply as for unsupported retaining walls restrained in the ground and for flexible supported walls. Also see R 4, Paragraph 5 (Section 3.2), R 6, Paragraph 5 (Section 3.4), R 7, Paragraph 1 (Section 3.5), R 12, Paragraph 2 (Section 5.1) and R 16, Paragraph 2 (Section 6.1).
4. If two opposing retaining walls are partially supported by anchors and partially by struts, earth pressure distribution may be selected similar to fully

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braced excavations. The anchors shall be prestressed appropriately. If necessary, the variable flexibility of the support points shall be taken into consideration when determining internal forces or moments.

5. In general it is admissible to prestress all anchors to 80 % of the characteristic effects E_k computed for the fully excavated state for active earth pressure design and to 100 % for design using pressures greater than the active earth pressure, see R 22 (Section 9.5). Only if these measures lead to overloading of the excavation structure or excessive deflection of the top of the retaining wall towards the ground, thereby representing a possible hazard to structures or pipelines, may it be necessary to initially prestress the anchors corresponding to the characteristic anchor forces prevalent in the construction stage following anchor installation and to re-stress at subsequent construction stages.

7.2 Analysis of force transfer from anchors to the ground (R 43)

1. The GEO 2 limit state according to R 78, Paragraph 4 (Section 1.4) governs analysis of force transfer from the anchors to the ground.
2. Sufficient safety of force transfer from the anchors to the ground is given if the limit state condition:

   $P_d \leq R_{a,d}$

   is fulfilled, i.e. if the design value of the anchor load P_d is, at most, as large as the design value of the pull-out resistance $R_{a,d}$.
3. The design value of the anchor load is composed of:
   a) the design value of the force acting on an anchorage, given by the design of the anchored wall;
   b) if applicable, the design value $E_{a,d}$ of the active earth pressure, acting on the rear face of the anchor wall or anchor plate.

   The design value $R_{a,d}$ of the resistance is determined according to Paragraphs 4 to 7 below.
4. For determination of the characteristic passive earth pressure $E_{p,k}$ in front of continuous anchor walls the characteristic value of the angle of earth pressure inclination is assumed at $\delta_{p,k} = 0$, if the only vertical force is that of the self-weight of the wall. Otherwise, the influence of anchor inclination shall be taken into consideration, in particular for anchors that slope down toward the anchor wall. If the anchor wall is covered with earth, the approximate passive earth pressure may be determined similar to a wall be-

ginning at ground level. The passive earth pressure design value is obtained from:

$R_d = E_{p,d} = E_{p,k}/\gamma_{R,e}$.

5. The characteristic three-dimensional passive earth pressure $E^*_{p,k}$ in front of anchor plates may be determined according to [35] or, as for soldier piles, according to [20] or DIN 4085. However, if the distance a between the anchor plates is small, not more than the proportion of the planar passive earth pressure $E_{p,k}$ determined using $\delta_p = 0$ may be adopted:

   $E^*_{p,k} \leq E_{p,k} \cdot a$.

   The three-dimensional passive earth pressure design value $E^*_{pk,d}$ is obtained from:

   $E^*_{p,d} = E^*_{p,k}/\gamma_{R,e}$.

6. The characteristic pull-out resistance $R_{a,k}$ of grouted anchors is obtained according to R 86 (Section 13.11).

7. The design pull-out resistance value is obtained from:

   $R_{a,d} = R_{a,k}/\gamma_a$.

8. Anchorages using tension piles shall be designed in compliance with Section 7, 'Pile Foundations', of 'Handbuch Eurocode 7, Band 1'.

7.3 Verification of stability at the lower failure plane (R 44)

1. Verification of stability at the lower failure plane is required for anchored retaining walls. It serves to determine the necessary anchor length, assuming that the anchors form a contiguous soil prism together with the wall and the surrounding ground, which slides on an upward-curved slip surface in the failure state, and rotates around a deep point (Figure R 44-1). When investigating, first the anchorage length needs to be selected before stability is analysed.

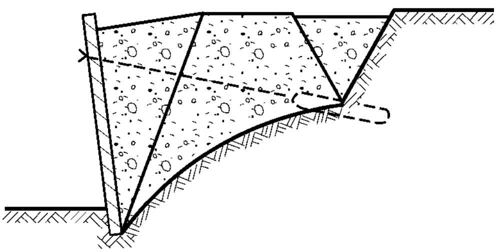


Figure R 44-1. Deep-seated failure after *Kranz* [99]

2. The analysis model described below is based on the *Kranz* method [99], which was originally derived for single-anchored walls utilising a free earth support with anchor walls and untensioned anchors. In addition:
   - this also applies to prestressed anchors designed for active earth pressure or increased active earth pressure;
   - with the *Ranke* and *Ostermayer* [17] extension it is a very good approximation solution for multiple-anchored walls;
   - it can also be transferred to walls restrained in the ground.
3. Using the *Kranz* method [99], the upward-curved slip surface is replaced by a planar slip surface. This can also be regarded as a stability problem for a trapezoidal soil prism separated from the retaining wall by a vertical cutting plane. The forces acting on the soil prism as shown in Figure R 44-2 a) are composed of the actions according to Paragraph 8 and the ground reactions in the lower failure plane according to Paragraph 9. This separation does not influence the results, because characteristic forces are adopted in both cases. The resistance that the system can mobilise after slipping is obtained from the corresponding force polygon as shown in Figure R 44-2 b) in the form of the possible anchor force $R_{A,cal}$. The details are described in the following paragraphs.
4. When analysing stability at the lower failure plane, consideration should be given to whether all the soil between the anchors participates in the formation of a soil prism as discussed in Paragraph 1:
   a) For anchor walls, anchor plates and grouted anchors spaced smaller than half the anchor bond length l_r, sufficient stability is given if the condition for the GEO 2 limit state:

   $P_d \leq R_{A,d}$.

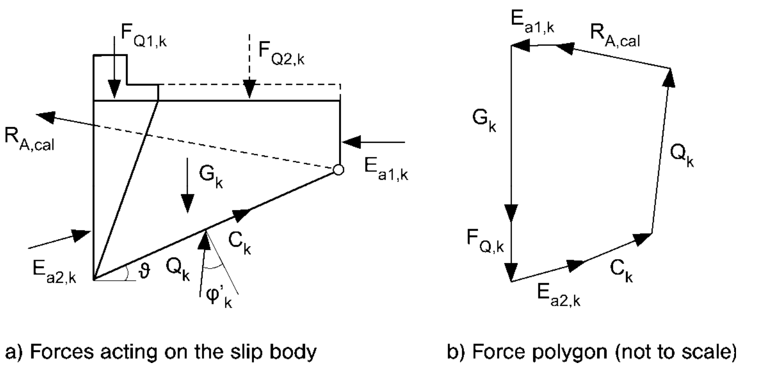


Figure R 44-2. Determination of the resistance $R_{A,cal}$ when verifying stability at the lower failure plane is fulfilled, where:

P_d design value of the anchor load;
$R_{A,d}$ design value of the resistance.

Stability may also be analysed using the horizontal components of the forces involved. The limit equilibrium condition then governs analysis:

$P_{h,d} \leq R_{h,A,d}$.

b) If the spacing *a* of grouted anchors is greater than half of the anchor bond length l_r, then the possible anchor force $R_{A,cal}$ shall be reduced to:

$R_{A,red,cal} = ½ \cdot R_{A,cal} \cdot l_r/a$

in compliance with EAU, Recommendation R 10 [2]. The limit equilibrium condition then governs analysis:

$P_d \leq R_{A,red,d}$.

5. The design value of the force acting on an anchor is determined from:

$P_d = P_{G,k} \cdot \gamma_G + P_{Q,k} \cdot \gamma_Q$;

And the resistance design value from:

$R_{A,d} = R_{A,cal}/\gamma_{R,e}$.

The variables $P_{G,k}$ and $P_{Q,k}$ are obtained from the determination of action effects on the retaining wall. If all variable actions over and above the unbounded distributed load $p_k \leq 10\ kN/m^2$ are increased by the factor f_q according to R 104, Paragraph 5 (Section 4.11), determination of the load design value is simplified.

6. The rearward boundary of the sliding earth mass is defined as listed below:

a) for continuous anchor walls, a plane from the toe of the anchor wall, extending vertically to the ground surface.
b) for individual anchor plates, an equivalent anchor wall shall be assumed at a distance $½ \cdot a_1$ in front of the anchor plates, whereby a_1 represents the clear distance between the anchor plates.
c) the equivalent anchor wall shall be assumed to be at the centre of the planned anchor bond length of grouted anchors.

The lower failure plane of anchor walls and anchor plates is assumed to be at their lower edge, for grouted anchors at the centroid of the anchor bond length. At anchor structures with fixed earth support, being pulled at the head the point of zero shear force shall be assumed.

7. If walls have free earth support the toe of the lower failure plane is assumed to be located at the bottom edge of the wall or soldier pile. Otherwise, the following points apply:

   a) The location of the toe in the contact zone is assumed as follows:

      – in the wall axis for soldier pile walls and sheet pile walls;
      – at the rear wall face for in-situ concrete walls.

   b) If the wall is embedded deeper than necessary to accommodate the horizontal support force, in order to absorb vertical loads (or for any other reason), the bottom edge of the wall shall be the depth which would be sufficient without considering the vertical loads.
   c) If the embedment depth of a retaining wall is not taken into account (either in reality or merely for analysis according to R 15, Paragraph 6 b) (Section 5.5)) and thus support below excavation level is neglected, the toe shall be assumed to be at the depth at which the design earth pressure acting below the excavation level can be accommodated by the design passive earth pressure. Also see Figure R 15-1 c) (Section 5.5).
   d) If wall toe displacement is anticipated for stiff walls loaded by large water pressure despite the wall being lengthened:

      – for safety against failure by uplift;
      – to limit seepage forces or;
      – to seal the excavation;

      the actual wall toe shall be adopted according to [96] as the starting point of the lower failure plane. This does not apply if the walls are braced at the earliest opportunity at the excavation level, e.g. by an underwater concrete slab or by a deep jet grouted base.
   e) For fully or partially fixed earth support and for elastic support of the wall in the ground the point of zero shear force shall be taken as the toe for analysis.

8. The following procedure is used to determine the characteristic actions:

   a) The earth pressure force $E_{a1,k}$ is obtained using the identical characteristic soil parameters which are used to determine the earth pressure force $E_{a2,k}$, the embedment depth and the action effects. Any possible live load at ground level shall always be taken into consideration when determining $E_{a1,k}$. For grouted anchors $\delta_a = \beta$. Analysis may be performed using $\delta_a = {}^2/_3 \cdot \varphi'_k$ for anchor walls and anchor plates.
   b) The characteristic load G_k from soil self-weight is obtained from the geometrical dimensions of the sliding mass and the same unit weight values adopted for determination of the earth pressure force $E_{a2,k}$.
   c) The variable action $F_{Q,k}$ is composed of two elements:

- The variable action $F_{Q1,k}$ is the sum of the live loads adopted for determining the earth pressure $E_{a2,k}$ and the anchor force P_k. This is the proportion of live loads acting on the active failure wedge as shown in Figure R 44-2 a), which is generally limited by a slip surface with the angle $\vartheta_{a,k}$. A slip surface with the angle $\vartheta_{z,k}$ may be governing for:

  – flexible supported walls according to R 6, Paragraph 5 (Section 3.4) and;
  – excavations adjacent to structures according to R 28, Paragraph 12 b) (Section 9.3).

- The variable action $F_{Q2,k}$ as shown in Figure R 44-2 a) is the sum of the live loads acting on the remainder of the ground surface extending from the active failure wedge to the imagined anchor wall. It shall only be adopted if $\vartheta > \varphi'_k$.

The action $F_{Q,k}$ in Figure R 44-2 b) corresponds to the force $F_{Q1,k}$ where $\vartheta \leq \varphi'_k$ or the sum of $F_{Q1,k}$ and $F_{Q2,k}$ where $\vartheta > \varphi'_k$.

9. The following procedure is used to determine the characteristic magnitude of the ground reaction in the lower failure plane:

   a) If applicable, the characteristic cohesive force C_k is obtained from the available cohesion c'_k for the slip surface length L using:
   $C_k = c'_k \cdot L$.

   b) The characteristic reaction force Q_k in the lower failure plane is given by the intersection of the line of acting at an angle φ'_k to the normal to the slip surface and the line of acting of the anchor force $R_{A,cal}$ in the force polygon.

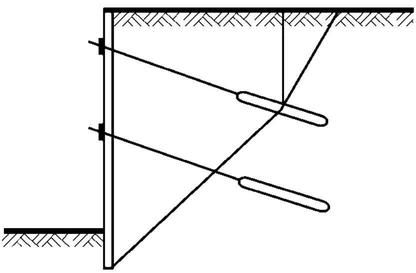
10. The stability analysis may be performed according to [17] for multiple-anchored retaining walls. The regulations in Sections 3 to 9 are supplemented as follows:

    a) Each centre point of the anchor bond length shall be assumed to be the end point of the lower failure plane in every construction stage.
    b) The loads P_k on all anchors with anchor bond lengths within the slipping earth prism or within the active failure wedge resulting in the earth pressure force $E_{ag1,k}$ shall be characteristic forces.
    c) The forces of anchors with anchor bond lengths intercepted by the lower failure plane may be divided into a component in front of the intersection and a component behind it, assuming uniform skin friction distribution along the anchor bond length. The proportion of the anchor force transferred within the sliding mass shall be treated as a load. The

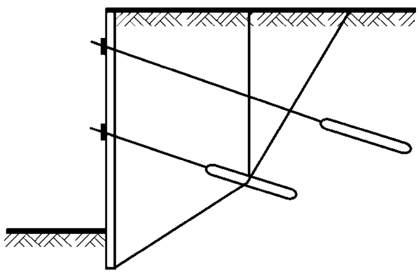
same applies to the anchor forces intercepted by the active slip surface behind the equivalent anchor wall.

d) If, in exceptional cases, the earth pressure from the equivalent load $F_{Q1,k}$ according to Paragraph 7 c) as a result of the continuity effect unloads the anchor, adopted as the end point of the lower failure plane, an additional investigation shall be carried out without this equivalent load.

e) If not all anchors are inclined at the same angle, a mean inclination shall be determined. For a precise analysis the sum of the vertical components and the sum of the horizontal components of the anchor forces, which are treated as loads according to Paragraph b) and Paragraph c), shall be determined. If the mean inclination is estimated, it shall be estimated conservatively, i.e. if so, greater than the precisely determined mean.



a) Lower anchor is outside of the lower failure plane



b) Upper anchor is outside of the active failure plane

Figure R 44-3. Example of anchors whose forces are not taken into consideration as actions

11. For anchored retaining walls designed for increased active earth pressure, or for reduced/full at-rest earth pressure, the stability of the lower failure plane may, in principle, be analysed according to the same rules as for active earth pressure. However, the regulations in Sections 3 to 9 are supplemented as follows:

a) The earth pressure $E_{2,k}$ determined according to R 22 (Section 9.5) and R 23 (Section 9.6) takes the place of the active earth pressure $E_{a2,k}$ in the force polygon as shown in Figure R 44-2 b).

b) The earth pressure $E_{1,k}$ takes the place of the active earth pressure $E_{a1,k}$ in the force polygon as shown in Figure R 44-2 b). It is determined according to the same rules as the earth pressure $E_{2,k}$ as described in R 22 (Section 9.5) and R 23 (Section 9.6).

c) The partial safety factors for permanent and variable loads and for resistances may be linearly interpolated according to R 22, Paragraph 3 (Section 9.5) between:

   – partial safety factors for the transient design situation when adopting active earth pressure and;
   – partial safety factors for the persistent design situation when adopting at-rest earth pressure.

   Because this interpolation only has a minor impact on the one hand, but analysis of stability of the lower failure plane is highly sensitive to inaccuracies on the other hand, it is generally recommended to analyse using the partial safety factors for the persistent design situation, if the retaining wall was designed with increased active earth pressure.

12. See EAU, Recommendation R 10 [2] for taking alternating soil layers and positive water pressures into consideration and for analysing the lower failure plane of anchorages using tension piles.

7.4 Analysis of overall stability (R 45)

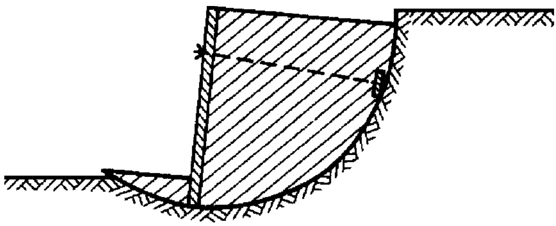
1. In principle, overall stability shall also be analysed for anchored retaining walls. However, empiricism demonstrates that it is sufficient to limit this analysis to exceptional cases. Greater dimensions or greater anchor lengths may occur, for example:

   a) for large ground surcharges in the vicinity of the fixed anchor length;
   b) if a soil layer with a shear strength lower than the overlying layers is present below the toe;
   c) for retaining walls that just reach or barely extend beyond the excavation level;
   d) if the rear of the wall is heavily inclined towards the ground;
   e) if the ground behind the wall rises;
   f) if the ground falls away in front of the wall.

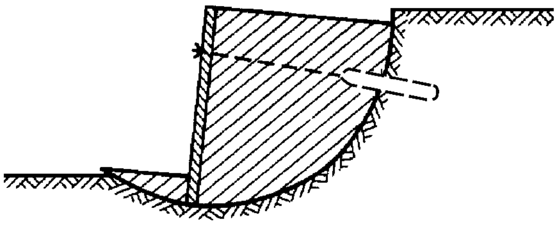
2. When analysing overall stability it is assumed that the anchors fix the retaining wall to the ground behind the wall to form a monolithic structure sliding on a curved slip surface (Figure R 45-1). Here, the wall toe moves further forwards than the top of the wall, in contrast to analysis of the lower failure plane. This is associated with rotation of the monolithic mass around a point, located high above the sliding mass. Analysis of overall stability is a GEO 3 limit state in accordance with R 78, Paragraph 5 (Section 1.4). Sufficient overall stability is given if the limit state condition:

   $E_{M,d} \leq R_{M,d}$

is fulfilled, i.e. if the sum $E_{M,d}$ of the design values of the acting torques (moments) is no greater than the sum $R_{M,d}$ of the resisting torques.



a) Anchorage using an anchor wall (dead man construction)



b) Anchorage using ground anchors

Figure R 45-1. Overall failure for a single-anchored wall

3. In terms of the GEO 3 limit state the design values of the governing torques are obtained as follows:

   a) When determining the acting torques all permanent loads are multiplied by the partial safety factor $\gamma_G = 1.00$ and all unfavourable variable loads by the partial safety factor $\gamma_Q > 1.00$ as shown in Table 6.1 in Appendix A 6.
   b) When determining the resisting torques the shear strength of the soil is reduced by applying the partial safety factors $\gamma'_\varphi/\gamma_{\varphi u}$ and γ'_c/γ_{cu} as shown in Table 6.3 in Appendix A 6.

4. Analysis of overall stability shall generally be performed using circular slip surfaces. Only in well-substantiated cases, e.g. if development of a circular slip surface penetrating deep in the ground is prevented as a result of differing shear strengths or the inclination of the soil layers, may it be necessary to assume slip surfaces formed of planar sections of varying inclination [45, 54, 55]. However, regardless of this, the end section of a circular slip surface tapering out at an angle greater than $\vartheta_p = 45° - ½ \cdot \varphi'_k$ shall always

be replaced by the end tangent at an angle ϑ_p or by the passive earth pressure. See DIN 4084 for additional notes.

5. In principle, the governing failure mechanism is influenced by two factors:

   a) At the top, the end of the anchored structure is governing. For anchor walls and anchor plates the governing slip surface contacts the rear face of the anchored structure, see Figure R 45-1 a). For grouted anchors it is sufficiently precise and generally conservative enough to assume the centroid of the anchor bond length as the effective end point of the anchor as shown in Figure R 45-1 b), based on R 44, Paragraph 5 d) (Section 7.3).
   b) At the bottom, the governing slip surface generally contacts the toe of the retaining wall or the soldier pile. For retaining walls with a shallow embedment depth according to R 44, Paragraph 6 c) (Section 7.3), or for the situation described in Paragraph 1 b), the governing slip surface can also be deeper.

   For a more precise investigation of grouted anchors to DIN 4084, those failure mechanisms completely enclosing the anchor bond length and those intersecting the anchor bond length shall be investigated. In the latter case the activatable action effects may be taken into consideration as resistances. Also see Paragraph 6.

6. For a more precise investigation of single-anchored walls, and always for multiple-anchored walls, it may be necessary to also consider slip surfaces intersecting individual rows of anchors. In these cases the following shall be taken into account:

   a) The torque (moment) resulting from the axial force acting in the intersected anchor, with reference to the centre of rotation of the slip circle, may be taken into consideration if it acts as a support; if it reduces stability, it must be taken into consideration.
   b) If the anchor is intersected in the anchor bond length the effective axial force may be divided accordingly, assuming uniform skin friction distribution in the anchor bond length. Only that proportion of the force transferred to the ground outside of the slip circle is effective.
   c) The additional friction force in the slip plane, generated by the effective component of the anchor force in the slip plane and transferred to the ground outside of the slip circle, may be incorporated in the analysis as a supporting force.
   d) In addition, the shear forces acting against overall failure in the intersected structural components may also be considered. However, these shear forces may only be taken into account:

- as permitted by the yield strength of the steel, taking the prevalent normal, bending and shear stresses into consideration;
- at a magnitude that allows the adopted shear force to be transferred to the ground by the intersected component without causing large deflections.

The second restriction also applies to the bearing members of a soldier pile wall.

e) The information given in Paragraphs a) to d) applies regardless of the anchor type. However, when applying the axial force, differentiation is required in all cases as to whether the anchors are self-tensioning or non-self-tensioning as a result of their angle of intersection with the slip surface. See DIN 4084 for details.

7. For analysis of overall stability, retaining walls designed for active earth pressure only require the partial safety factors given for the transient design situation. The following shall be observed for greater demands on serviceability:

a) The partial safety factors stipulated for the persistent design situation shall be adopted for retaining walls designed for reduced at-rest earth pressure or for the full at-rest earth pressure.
b) If increased active earth pressure is adopted, interpolation may be performed between the partial safety factors for the transient design situation when adopting active earth pressure and the partial safety factors for the persistent design situation when adopting at-rest earth pressure, in accordance with R 22, Paragraph 3 (Section 9.5). However, taking the serviceability state into consideration, it is generally recommended to analyse using the partial safety factors for the persistent design situation, if the retaining wall was designed with increased active earth pressure.

7.5 Measures to counteract deflections in anchored retaining walls (R 46)

1. As can be demonstrated from empirical data, deflections in anchored retaining walls are also anticipated if the walls and their anchoring components are designed and prestressed for increased active earth pressure, or for reduced or full at-rest earth pressure. Governing in this respect are the deflections and deformations of the soil mass which is enclosed by the retaining wall, similar to inside a cofferdam, and a plane connecting the points assumed to transfer the anchor forces to the ground in accordance

with R 44, Paragraph 7 (Section 7.3) (Figure R 46-1). According to R 83, Paragraphs 8 to 11 (Section 4.10), the deflections principally consist of:

a) elastic deformation of the wall;
b) tilting of the cofferdam-like soil mass;
c) shear deformation of the soil mass and the ground below it;
d) horizontal deflection due to compaction of the ground below the excavation level and;
e) an additional relaxing movement due to unloading of the ground when excavating.

These deflections have been observed in excavations in at least medium-dense, cohesionless soil and at least stiff, cohesive soil at depths of more than 10 m to 12 m [39, 51, 74].

Deformation and deflection of the ground surface are associated with the movement of the cofferdam-like soil mass.

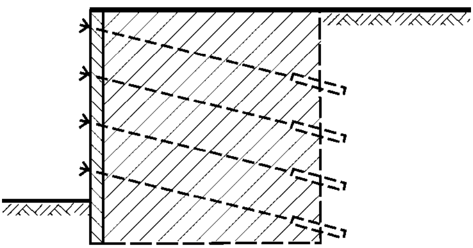


Figure R 46-1. Development of a cofferdam-like soil mass

2. For excavations in water the toe of the wall may move more than usual if the effective vertical stress resulting from seepage pressure below a soft gel base as shown in Figure R 62-1 c) (Section 10.5) is reduced within the soft gel base. The ground reaction force $B_{h,k}$ is then transferred to the soil layer above the soft gel base on the opposite side of the retaining wall and not into the deeper subsurface. The entire soil layer above the soft gel base is thus subject to compaction resulting from the ground reaction force $B_{h,k}$, which may initiate a correspondingly large deflection of the wall toe [96].
3. If the serviceability analysis according to R 83 (Section 4.10) indicates that excessive wall deflections are anticipated for an anchored retaining wall with anchor lengths determined according to R 44 (Section 7.3), appropriate measures shall be taken, e.g.:

   a) lengthening of the anchors;
   b) replacement of at least one row of anchors by bracing;

c) replacement of the anchors by struts in some parts of the excavation to generate fixed datum reference points [29];
d) staged construction of excavation and structure.

The bracing shall be designed for substantially higher loads than would correspond to the proportion of the computed earth pressure, e.g. for double load. If necessary, the observational method shall be adopted compliant with 'Handbuch Eurocode 7, Band 1'.

4. Regardless of the measures implemented according to Paragraph 3 anchors adjacent to existing structures should always be spread and staggered in length in the zone of the anchor bond length. This does not apply if the fixed anchor lengths are located within rigid rock. Spreading can also reduce the mutual influence of the anchors. By staggering the anchor lengths, the danger of a sudden offset in the settlement profile behind the cofferdam-like mass can generally be eliminated and a shallower settlement trough be achieved.

   Where staggering is adopted, every second anchor shall be lengthened, while retaining the original fixed anchor length. If the staggering lies within a region where a sudden offset in the settlement profile needs to be avoided because of adjacent structures, the sum of the additional anchor lengths shall be approximately 20 % of the computed sum of the required anchor lengths in the area of the affected section of the retaining wall. Depending on local conditions and requirements, certain layers of anchors are selected to distribute the spreading.

5. If deflections and deformations need to be limited, it is recommended to monitor at least the horizontal and vertical deflections of the top of the anchored wall from the outset, so that counter-measures can be implemented at an early stage, if necessary. Anchor force measurements and settlement measurements in the close vicinity are also recommended for excavations in soft, cohesive soil and excavations adjacent to structures.

6. The wall deflections mentioned in Paragraph 1 cannot be substantially reduced by very high prestressing. In general, such prestressing merely has the effect of generating internal stress conditions between the retaining wall and the fixed anchor length, which prevents formation of an active earth pressure failure wedge and decompaction of the ground. High prestressing, on the other hand, may lead to heavy lateral compression of the soil mass, damage to cellar masonry in adjacent structures and especially large settlement at the rear of the anchored zone.

8 Excavations with special ground plans

8.1 Excavations with circular plan (R 73)

1. If the depth of a circular excavation is not greater than half of the diameter, the three-dimensional earth pressure distribution from soil self-weight and unbounded distributed loads is only insignificantly different to the earth pressure on an infinitely long retaining wall. If, for flexible excavation structures, the depth is greater than the diameter, the three-dimensional earth pressure distribution is so far below the earth pressure based on classical earth pressure theory that the difference can generally no longer be ignored if economical methods are aimed for.

2. Similar to the earth pressure on an infinitely long retaining wall, the magnitude and distribution of the earth pressure from soil self-weight and unbounded distributed loads depends on the construction methods employed, the stiffness of the wall and the flexibility of the supports. The following limitations are imposed by R 67 (Section 1.5), with respect to the flexibility of the system as a whole:

   a) Generally, diaphragm walls and secant bored pile walls can be regarded as non-yielding systems if they form an unbroken circle and simultaneously serve as a ring beam.
   b) Retaining walls can be regarded as almost non-yielding if they possess a certain inherent flexibility, e.g. sheet pile walls and contiguous bored pile walls, but are supported by stiff ring beams.
   c) Generally, all retaining walls with soil excavation in advance of the installation of the infill walls, and which are supported by rings or other devices, can be regarded as slightly yielding, in particular soldier pile walls with timber infilling.
   d) All retaining walls that rely solely on their restraint in the ground for stability can be regarded as yielding systems, e.g. soldier pile walls or sheet pile walls without supports.

   The installation of segments, liner plates or shotcrete linings can be regarded as producing either slightly yielding or nearly inflexible systems, depending on the excavation depth and ground stability. The same applies to soldier pile walls with shotcrete infilling or in-situ concrete infilling using formwork, which ensures circular load transmission.

3. The following points apply for the determination of earth pressure:

   a) The at-rest earth pressure E_0 may be regarded as the upper limit value for non-yielding systems according to Paragraph 2 a). An earth pressure

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with $E = ½ \cdot (E_0 + E_{aR})$ may be regarded as the lower limit value. E_{aR} is the three-dimensional active earth pressure according to the modified disk-element theory after *Walz* and *Hock* [81], [82].

b) The upper limit value for almost non-yielding systems according to Paragraph 2 b) can be taken as an earth pressure $E = ½ \cdot (E_0 + E_{aR})$ and the lower limit as the three-dimensional earth pressure E_{aR} according to the modified disk-element theory.

c) The upper limit value for slightly yielding systems according to Paragraph 2 c) can be taken as the earth pressure E_{aR} according to the modified disk-element theory; the lower limit value can be based on the simplified approach after *Beresanzew* [83].

d) The earth pressure for yielding systems according to Paragraph 2 d) can be determined using the simplified approach after *Beresanzew*.

e) In cohesionless soils, the approach after *Steinfeld* [84] may be selected in place of the modified disk-element theory after *Walz* and *Hock*, if based on the diagram of possible earth pressure distributions.

f) For an approach utilising the modified disk-element theory, a ring bracing factor of $k_t = 0.5$ shall be adopted when determining the upper limit value of the three-dimensional earth pressure, but $k_t = 1.0$ shall be adopted for determination of the lower limit value. The ring bracing factors $\lambda_s = 0.7$ and $\lambda_s = 1.0$ apply accordingly for the approach after *Steinfeld*.

g) In order to assess the unfavourable stresses at all points of the excavation structure, the action effects shall be determined in conjunction with the adopted live loads for both the upper and lower limit values for the case in question.

h) For retaining systems that cannot transfer vertical loads to the subsoil, e.g. shotcrete shafts, the angle of earth pressure inclination shall be adopted at $\delta_a = 0°$ according to R 89 (Section 2.3).

i) R 4, Paragraphs 3 to 5 (Section 3.2) apply with regard to the minimum earth pressure.

4. It can be assumed that the earth pressure distribution deviates only slightly from a linear increase with depth in non-yielding systems according to Paragraph 3 a). However, if the preconditions for active earth pressure are fulfilled, the total load generated by the three-dimensional active earth pressure shall be distributed across the complete height of the wall, based on the principles of Recommendation R 5 (Section 3.3). If the total earth pressure lies between the at-rest earth pressure E_{0h} and the three-dimensional active earth pressure E_{aR}, the earth pressure distribution shall be interpolated. Due to the lack of measurement data available for circular excavations and because theoretical considerations cannot exclude the possibility that upward redistribution of active earth pressure is less pronounced than for infinitely long retaining walls, it is recommended to analyse using two

limit distributions and to base design of individual components on the greater action effects. The load models given in R 69 (Section 5.2) and R 70 (Section 6.2) can be adopted as the upper limit.

5. Unanticipated deviations from the radial symmetry, e.g. heterogeneity of the ground not recognised in exposures, or accidental geometrical imperfections, should be taken into consideration in the load approach. In approximation, radially acting earth pressure from a bounded distributed load $p_k = 10$ kN/m^2, distributed in keeping with a cosine function, may be adopted as permanently acting as shown in Figure R 73-1, e.g. in keeping with the function $e_h = \max e_h \cdot \cos^2 \alpha$. The maximum value max. e_h is obtained in the at-rest earth pressure limit case from the max. e_h = max. $e_{0ph} = p_k \cdot K_{ogh}$ approach, in the case of active earth pressure from the max. e_h = max. $e_{aph} = p_k \cdot K_{agh}$ approach as for an infinitely long wall. If a value between the at-rest earth pressure and the active earth pressure is adopted for determination of the earth pressure, this also applies to earth pressure from the bounded live load.

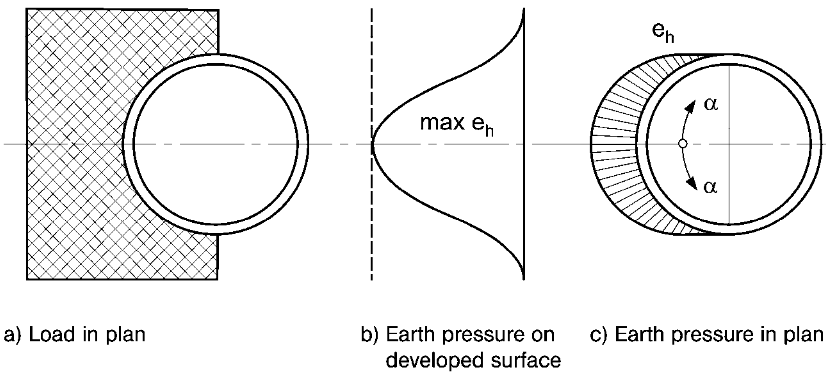


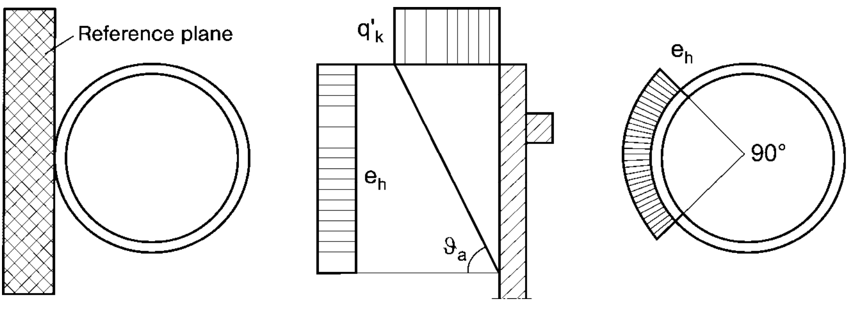
Figure R 73-1. Earth pressure from a bounded distributed load $p_k = 10$ kN/m^2

The recommended approach covers geometrical imperfections in structures without a ring beam and a maximum deviation of the principle A and B axis dimensions of A : B ≤ 1.03. Adherence to this condition shall be examined by on-site measurements. If the centres of secant bored pile walls or the longitudinal axes of individual diaphragm wall slices do not coincide with a pressure line, the imperfection shall be corrected or compensated for by design measures or by structural measures.

6. If traffic or operating loads exceed the unbounded distributed load $p_k = 10$ kN/m^2 according to Section 5, only the actual load positions need be taken into consideration. Two cases may be considered:

a) If the load is represented by a strip load q'_k, according to R 55, Paragraph 3 (Section 2.6), or R 57, Paragraph 4 (Section 2.8), as shown in Figure R 73-2 a), the earth pressure shall be determined according to R 6 (Section 3.4) and R 7 (Section 3.5), as if a plane at a tangent to the circular excavation structure is the governing plane. In approximation, the determined earth pressure can be adopted as a radially acting load e_h for one quarter of the circumference of the excavation as shown in Figure R 73-2 c).
b) If the load is represented by point loads according to R 55 (Section 2.6) or R 57 (Section 2.8), as shown in Figure R 73-3 a), the earth pressure shall be determined according to R 6 (Section 3.4) and R 7 (Section 3.5), as if an imaginary plane at a tangent to the circular excavation structure is the governing plane, taking the associated contact areas and the load distribution in the upper road layers and in the ground according to R 3 (Section 2.5) into consideration. In approximation, the determined earth pressure can be adopted without precise analysis as a radially acting load e_h as shown in Figure R 73-3 c), with the same length l as the circumference of the circle resulting from the load distribution as shown in Figure R 73-3 a), but for a maximum of one quarter of the circumference.

If the earth pressure from soil self-weight is adopted as the at-rest earth pressure, the earth pressure from live loads according to R 23 (Section 9.6) may also be determined according to the theory of elastic half-space; if a value between the at-rest earth pressure and the active earth pressure is adopted for the earth pressure from soil self-weight, this also applies for the earth pressure from live loads.



a) Load in plan b) Load in section (example) c) Earth pressure in plan

Figure R 73-2. Earth pressure from a strip load q'_k

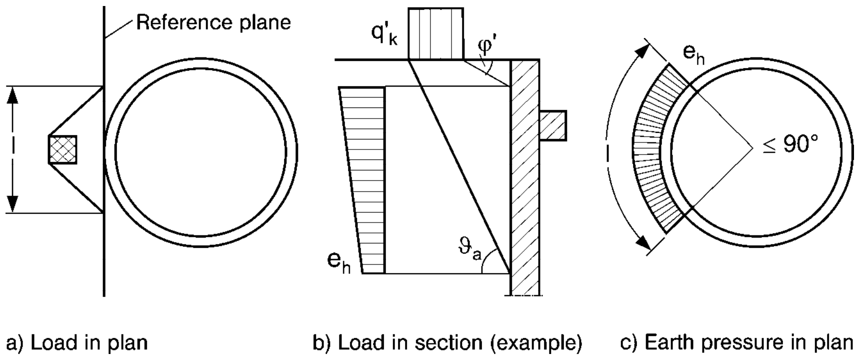


Figure R 73-3. Earth pressure for a point load

7. When determining the earth pressure from foundation loads for excavations adjacent to structures, the information given in Paragraph 6 applies accordingly:

   a) The load distribution and load length at the circumference of the excavation shall be assumed as shown in Figure R 73-3 for footing foundations.
   b) The earth pressure determined from strip footings shall be applied to a quarter of the circumference corresponding to Figure R 73-4 c).

   Otherwise, please observe Section 9.

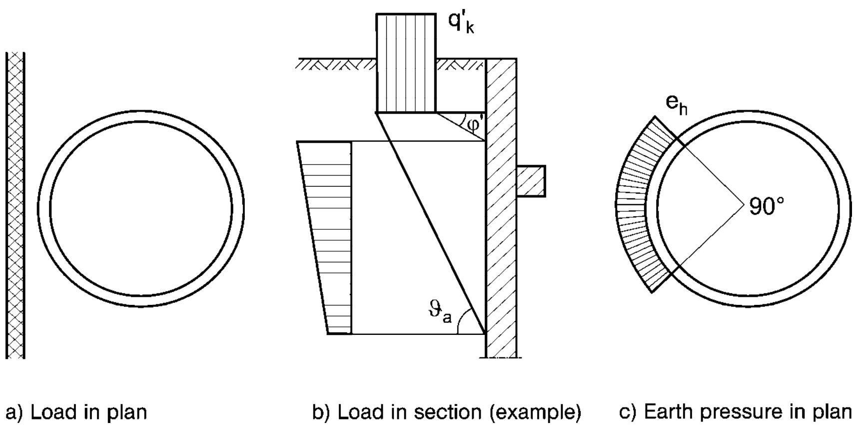


Figure R 73-4. Earth pressure from a strip footing

8. The subgrade reactions resulting from bounded surcharges according to Paragraphs 5 to 7 shall be adopted corresponding to the interaction between the load-deformation behaviour of the excavation structure and the load-deformation behaviour of the ground. In approximation, earth pressure of the same magnitude and distribution as on the load side of the excavation may be adopted as a substitute for the corresponding subgrade reactions on the opposite side. More precise methods shall be applied for higher demands on the precision of the determined action effects and deformations, e.g. for excavations adjacent to structures. If the subgrade reaction modulus method is employed and no precise investigations were carried out, the subgrade reaction modulus may be approximately determined from the horizontal constrained modulus of the ground and the outer radius of the excavation structure using $k_{s,k} = E_{S,k}/r$. The resulting total stress from the load stress $e_{h,k}$ and the subgrade reaction $\sigma_{ph,k}$ activated by the displacement may not be greater than half of the passive earth pressure stress $e_{ph,k}$.

9. Ground reactions resulting from the subgrade may not be adopted at the edge of access openings in the retaining wall. In approximation, it may be assumed that the modulus of subgrade reaction increases linearly from zero at the break-out edge and achieves the value given in Paragraph 8 at a distance of 1.0 m.

10. If the ground below the excavation level serves to support the wall, the passive earth pressure may be adopted as for an infinitely long wall, without the necessity for a more precise investigation of the three-dimensional stress state.

11. Ring- or polygon-shaped, stiff, bracing structures shall be designed for bending under a normal force. A stability investigation may generally be dispensed with if the contact with the retaining wall prevents ring deflection.

8.2 Excavations with oval plan (R 74)

1. If the dimensions of the principle axes A and B of an excavation with a curved, but not circular, plan as shown in Figure R 74-1 deviate by more than 3 % from one another, the deviations in the subgrade reactions compared to those of a circular plan can generally no longer be neglected. These deviations increase rapidly with an increasing A : B ratio and reach a value for A : B ≥ 1.5 for which assumptions and investigations are required which are beyond the scope of this Recommendation. Otherwise, the scope of this Recommendation is restricted to elliptically curved plans for which the radius of the larger curve is no more than 2.5 times that of the smaller

curve. The following approaches apply to elliptically curved plans as shown in Figure R 74-1 with a ratio A : B < 1.5, if no more precise investigations are performed, e.g. with the aid of finite element methods (FEM).

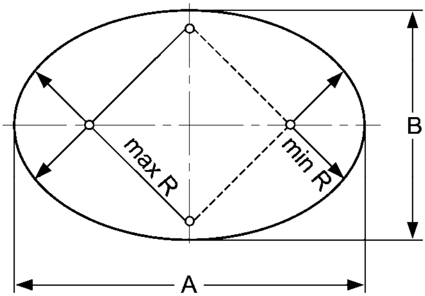


Figure R 74-1. Excavations with elliptically curved plan

2. The magnitude and distribution of earth pressure from soil self-weight and unbounded distributed loads depend on the type of construction method, the stiffness of the wall and the flexibility of the supports. The following limitations apply with respect to the flexibility of the wall in the region of the larger curve radius according to R 67 (Section 1.5) and R 73 (Section 8.1):

   a) Generally, diaphragm walls and secant bored pile walls can be regarded as almost non-yielding systems if they form an unbroken arc and simultaneously serve as a ring beam. A precondition for this is that the ground cannot unload whilst manufacturing the retaining wall.
   b) Retaining walls can be regarded as slightly yielding if they possess a certain inherent flexibility, e.g. sheet pile walls and contiguous bored pile walls, but are supported by stiff ring beams.
   c) Generally, all retaining walls in which the ground face is open before infill walls are installed and which are supported either by ring beams or other measures or not at all can be regarded as yielding systems, in particular soldier pile walls with timber infill walls.

3. The governing design earth pressure is principally dependent on the flexibility of the two elliptical curves with the smaller radius. For a more precise analysis, an initial stress state of the undeformed system shall be assumed, from which a final equilibrium state is developed in the context of the relationships between the earth pressure on the longer sides, the deformations of the excavation structure and the subgrade reactions on the shorter sides, if necessary iteratively. The initial stress state earth pressure

shall be assumed as for circular excavations as a function of the selected construction method. The radius of the section of the elliptical curve represents the respective circle radius. In approximation, the stress reduction associated with the anticipated deformations in those areas with the larger curve radius according to Paragraph 4 also leads to an increase in those areas with the smaller curve radius according to Paragraph 9.

4. The following points apply for the determination of earth pressure in the areas with the larger curve radius:

   a) The upper limit value for almost non-yielding systems according to Paragraph 2 a) can be taken as the earth pressure $E = \frac{1}{2} \cdot (E_0 + E_{aR})$ and the lower limit as the three-dimensional earth pressure E_{aR} according to the modified disk-element theory after *Walz* and *Hock* [81], [82].
   b) The upper limit value for slightly yielding systems according to Paragraph 2 b) can be taken as the earth pressure E_{aR} according to the modified disk-element theory; the lower limit value can be based on the simplified approach after *Beresanzew* [83].
   c) The earth pressure for yielding systems according to Paragraph 2 c) can be determined using the simplified approach after *Beresanzew*.
   d) In cohesionless soils, the approach after *Steinfeld* [84] may be selected in place of the modified disk-element theory after *Walz* and *Hock*, if based on the diagram of possible earth pressure distribution.
   e) For an approach utilising the modified disk-element theory, a ring bracing factor shall be adopted at $k_t = 0.5$ when determining the three-dimensional earth pressure, if the upper limit value is required, but with $k_t = 1.0$ for determination of the lower limit value. The ring bracing factors $\lambda_s = 0.7$ and $\lambda_s = 1.0$ apply accordingly for the approach after *Steinfeld*.
   f) In order to assess the unfavourable stresses at all points of the excavation structure, the action effects shall be determined in conjunction with the adopted live loads for both the upper and lower limit values for the case in question. If large stresses on the long side are unfavourable here, a smaller value than that resulting from Paragraphs a) and c) may be adopted, if separate investigations demonstrate that the earth pressure as a function of the anticipated wall deflection justifies this.
   g) For retaining systems that cannot transfer vertical loads to the subsoil, e.g. for oval shotcrete shafts, the angle of earth pressure inclination shall be adopted at $\delta_a = 0°$ according to R 89 (Section 2.3).
   h) R 4, Paragraphs 3 to 5 (Section 3.2) apply with regard to the minimum earth pressure.

5. If the preconditions for active earth pressure are fulfilled, the total stress developed by the three-dimensional active earth pressure shall be distributed over the wall height, based on the principles of Recommendation R 5

(Section 3.3). If the total earth pressure lies between the at-rest earth pressure E_{0h} and the three-dimensional active earth pressure E_{aR}, the earth pressure distribution shall be interpolated. Due to the lack of measurement data available for elliptical excavations and because theoretical considerations cannot exclude the possibility that upward redistribution of active earth pressure is less pronounced than for infinitely long retaining walls, it is recommended to analyse using two limit distributions and to base the design of individual components on the greater action effects. The load models given in R 69 (Section 5.2) and R 70 (Section 6.2) can be selected as the upper limit.

6. If unfavourable actions are anticipated with regard to the design of individual components of the excavation structure, an unbounded distributed load of at least $p_k = 10$ kN/m^2 on one side similar to Recommendations R 55 to R 57 (Sections 2.6 to 2.8) shall be adopted as a traffic or operating load. The resulting earth pressure shall be adopted for the whole zone of influence as a uniform, radially acting load ordinate as shown in Figure R 74-2, if it acts unfavourably. This load ordinate is obtained in the at-rest earth pressure limit case from the $e_h = e_{0ph} = p_k \cdot K_{0ph}$ approach, in the case of active earth pressure from the $e_h = e_{aph} = p_k \cdot K_{aph}$ approach as for an infinitely long wall. If a value between the at-rest earth pressure and the active earth pressure is adopted for determination of the earth pressure, this also applies to earth pressure from the unbounded distributed load p_k.

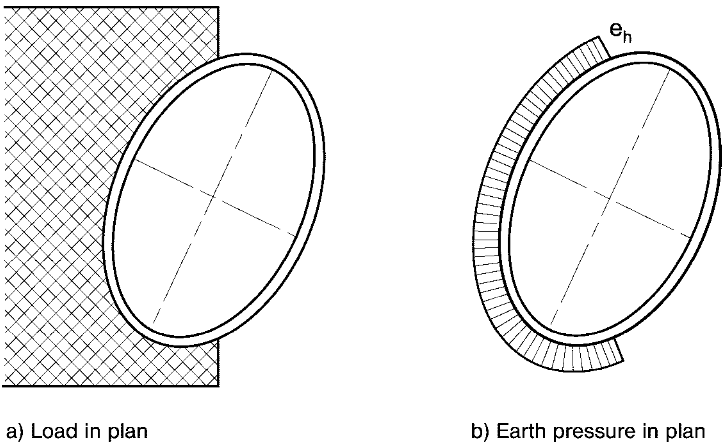


Figure R 74-2. Earth pressure from a distributed load $p_k = 10$ kN/m^2 unbounded on one side

7. If traffic or operating loads exceed the unbounded distributed load $p_k = 10$ kN/m^2 according to Section 6, only the actual load positions need be taken into consideration. Two cases may be considered:

   a) If the load is represented by a strip load q'_k, according to R 55, Paragraph 3 (Section 2.6), or R 57, Paragraph 4 (Section 2.8), as shown in Figure R 74-3 a), the earth pressure shall be determined according to R 6 (Section 3.4) and R 7 (Section 3.5), as if an imaginary plane at a tangent to the circular excavation structure is the governing plane. In approximation, the determined earth pressure can be adopted without precise analysis as a radially acting load e_h as shown in Figure R 74-3 c), but for not more than 1/8 of the circumference to each side of the tangent point and only inasmuch as the earth pressure acts unfavourably.

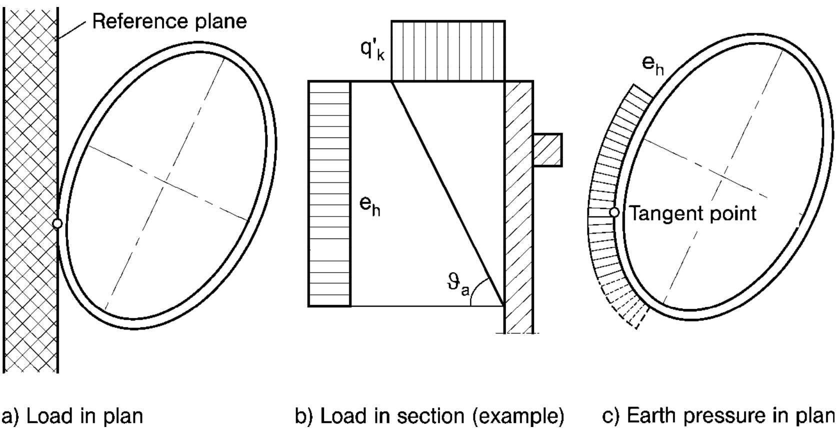


Figure R 74-3. Earth pressure from a strip load q'_k

   b) If the load is represented by point loads according to R 55 (Section 2.6) or R 57 (Section 2.8), as shown in Figure R 74-4 a), the earth pressure shall be determined according to R 6 (Section 3.4) and R 7 (Section 3.5), as if an imaginary plane at a tangent to the excavation structure is the governing plane, taking the associated contact areas and the load distribution in the upper road layers and in the ground according to R 3 (Section 2.5) into consideration. In approximation, the determined earth pressure can be adopted without precise analysis as a radially acting load e_h as shown in Figure R 74-4 c), with the same length l as the total circumference resulting from the load distribution as shown in Figure R 74-4 a), if it acts unfavourably.

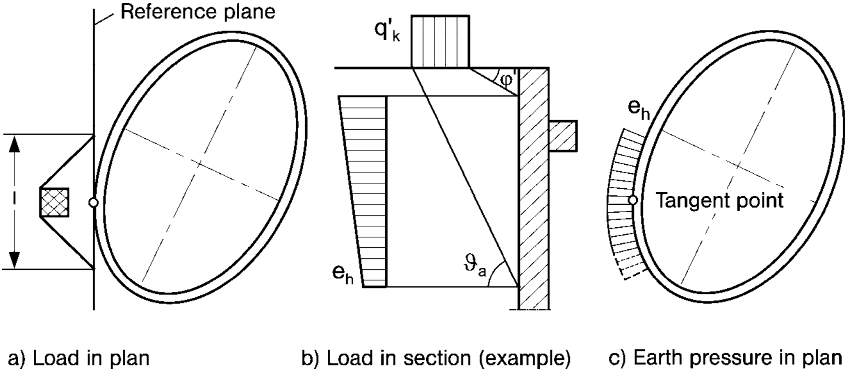


Figure R 74-4. Earth pressure for a point load

If the earth pressure from soil self-weight is adopted as the at-rest earth pressure, the earth pressure from live loads according to R 23 (Section 9.6) may also be determined according to the theory of elastic half-space; if a value between the at-rest earth pressure and the active earth pressure is adopted for the earth pressure from soil self-weight, this also applies for the earth pressure from live loads.

8. When determining the earth pressure from foundation loads, the information in Paragraph 7 applies accordingly:

   a) The load distributions and load length at the circumference of the excavation shall be determined as shown in Figure R 74-4 for footing foundations.
   b) The earth pressure determined from strip footings shall be applied to a quarter of the circumference as shown in Figure R 74-5 c), if it acts unfavourably. Half of the corresponding length shall be adopted in each direction from the point closest to the foundation.

   Otherwise, please observe Section 9.

9. The subgrade reactions in the region of the smaller radius curve may be adopted for determination of the action effects from earth pressure according to Paragraph 2. The same applies if the earth pressure from surcharges according to Paragraphs 6 to 8 acts on one side in the region of the large radius curve. In approximation, earth pressure of equal magnitude and distribution as on the load side of the excavation may be adopted in such cases as a substitute for the corresponding subgrade reactions on the opposite side. If earth pressure in the region of the curve transition ensues from the loads according to Paragraphs 6 to 8, subgrade reactions will also ensue in the region of the large curve radius. The subsequent ground reactions shall

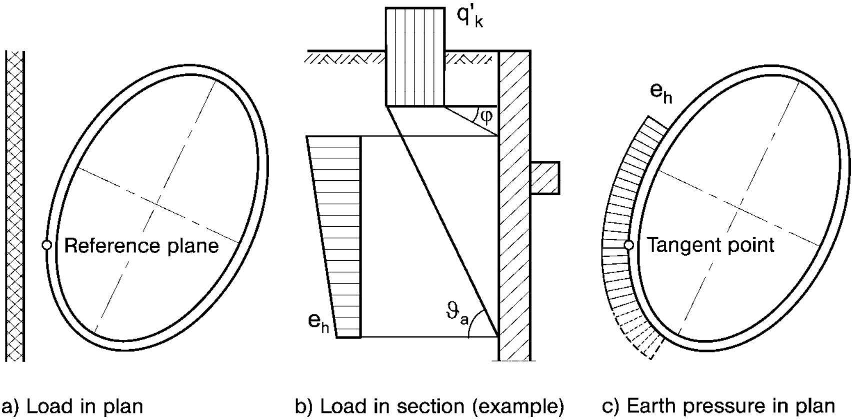


Figure R 74-5. Earth pressure from a strip footing

be adopted corresponding to the interaction between the load-deformation behaviour of the excavation structure and the load-deformation behaviour of the ground. If the subgrade reaction modulus method was employed for this purpose and no precise investigations were carried out, the design value of the subgrade reaction modulus may be approximately determined from the horizontal constrained modulus of the ground and the governing outer radius of the excavation structure using $k_{s,k} = E_{S,k}/r$. The resulting total stress from the load stress $e_{h,k}$ and the subgrade reaction $\sigma_{ph,k}$ activated by the displacement may not be greater than half of the passive earth pressure stress $e_{ph,k}$. Tensional bedding shall be excluded when determining the action effects for the governing load combinations.

10. Ground reactions resulting from the subgrade may not be adopted at the edge of access openings in the retaining wall. In approximation, it may be assumed that the modulus of subgrade reaction increases linearly from zero at the break-out edge and achieves the value according to Paragraph 9 at a distance of 1.0 m.

11. The upper and lower limit values of the characteristic value of the modulus of subgrade reaction shall be taken into consideration for estimating the deformations in the serviceability limit state. If necessary, more precise methods shall be applied. The subgrade reactions on the opposing sides shall be taken into consideration for bounded loads.

12. If the ground below the excavation level is utilised to support the wall, the passive earth pressure may be adopted as for an infinitely long wall according to R 14 (Section 5.3) and R 19 (Section 6.3), without the necessity for more precise investigation of the three-dimensional stress state.

13. Oval- or polygon-shaped, stiff, bracing structures shall be designed for bending under a normal force. A stability investigation may generally be dispensed with if the contact with the retaining wall prevents ring deflection.

8.3 Excavations with rectangular plan (R 75)

1. In principal, the retaining walls and the bracing or anchors of excavations with square or rectangular plans can be designed and constructed similar to those for elongated excavations. However, in the interests of economical design of structural members and a realistic deformation forecast, it is also permissible to take the earth pressure reduction caused by the three-dimensional effect into consideration for cohesionless or at least stiff, cohesive soil. The following procedures may be applied for determination of the reduced earth pressure:

   a) Procedure according to Paragraph 2, which assumes shear forces in the flank faces of a slipping two-dimensional earth wedge.
   b) Procedure according to Paragraph 3, which assumes a slipping three-dimensional body.

   The procedures suggested here assume an excavation structure similar to R 67 (Section 1.5), which is either not supported, yieldingly supported or slightly yieldingly supported, but is sufficiently deformable, in order to facilitate reduction of the at-rest earth pressure to the active earth pressure. Where these displacements are hindered at the excavation corners for diaphragm walls and secant bored pile walls, the at-rest earth pressure may be locally retained; this may, however, generally remain unconsidered.

2. Where procedures are applied that assume shear forces in the flanks of slipping earth wedges, these are based on a conceptual model as shown in Figure R 75-1 a), whereby an earth wedge approaches the excavation from all sides and the corner regions are immovable. Friction forces and, if applicable, cohesion forces are thus mobilised in the boundary surfaces between the slipping earth wedges and the immovable corner masses, thereby preventing slippage of the earth masses towards the excavation walls and reducing the total active earth pressure. Only procedures that do not overestimate the magnitudes of these forces may be adopted. See [85, 86] and DIN 4126. These may be suitable if the corners of the retaining wall are just as flexible as the middle sections of the excavation walls. The reduction in the total earth pressure can be implemented in the design of the individual components as shown in Figure R 75-2 a) in the form of chamfering or as shown in Figure R 75-2 b) in the form of steps in the continuous earth pressure E_h determined without the three-dimensional effect.

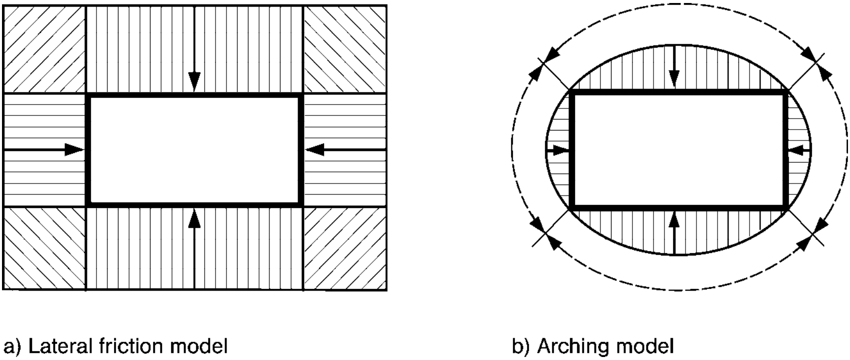


Figure R 75-1. Models for determination of the three-dimensional earth pressure for rectangular excavations

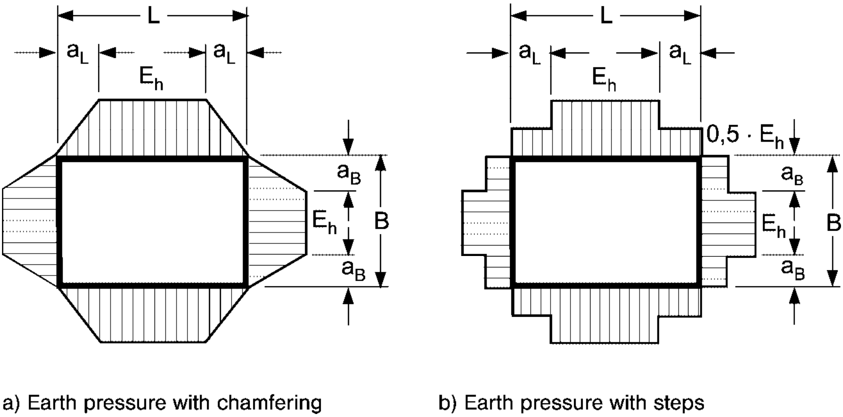


Figure R 75-2. Simplified approach for earth pressure reduction at excavation corners

3. For those procedures that assume a three-dimensional failure mass, the development of an arching effect as shown in Figure R 75-1 b) plays a governing role in the earth pressure reduction. Suitable procedures are those after *Karstedt* [53], and *Piaskovski* and *Kovalevski* [87], or specifications according to DIN 4085. The procedures based on three-dimensional sliding masses are suitable if the corners of the retaining walls are less flexible than the middle sections of the excavation walls. The difference between the total earth pressure for the continuous wall and the total earth pressure for the relevant area of the excavation side walls, corresponding to one of the procedures discussed above, can be implemented in the design of the individual components as shown in Figure R 75-3 a) in the form of cham-

fering, or as shown in Figure R 75-3 b) in the form of steps in the continuous earth pressure E_h determined without the three-dimensional effect.

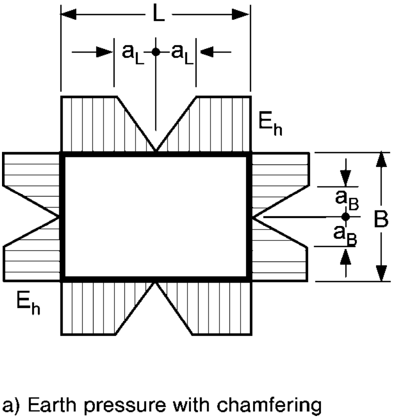


Figure R 75-3. Simplified approach for earth pressure reduction on the excavation sides

4. In Paragraphs 2 and 3, E_h designates the earth pressure on a continuous wall from soil self-weight, unbounded distributed load $p_k \leq 10$ kN/m^2 and, if applicable, cohesion, according to R 4 (Section 3.2), in combination with R 6 (Section 3.4), R 12 (Section 5.1) and R 16 (Section 6.1).

5. The earth pressure E_h on each side of the excavation may be chamfered as shown in Figures R 75-2 a) and R 75-3 a), or reduced to ½ · E_h without further analysis, as shown in Figures R 75-2 b) and R 75-3 b). The wall lengths for which a reduction may be applied follow from *Walz* [88], as a function of the depth H as follows

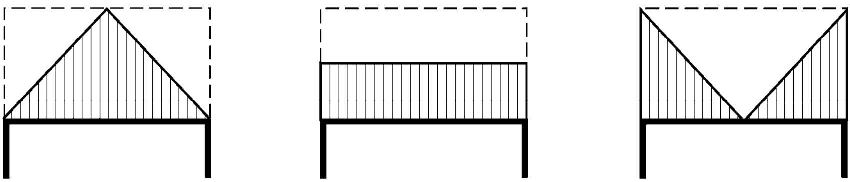
   $a_L = (0.35 - 0.06 \cdot H : L) \cdot H$ on those sides of length L;
   $a_B = (0.35 - 0.06 \cdot H : B) \cdot H$ on those sides of length B.

   Distribution of the earth pressure as shown in Figure R 75-2 is recommended if the preconditions according to Paragraph 2 are fulfilled; earth pressure distribution as shown in Figure R 75-3 is recommended if the preconditions according to Paragraph 3 are fulfilled.

6. If the excavation length for which the total earth pressure E_h may be reduced results in $2 \cdot a_L > L$ or $2 \cdot a_B > B$ for the end walls of narrow excavations (from approx. $H > 2.5 \cdot B$), the total load must then be adopted at a minimum of:

$E^*_{hL} = ½ \cdot E_h \cdot L$ on those sides of length L;
$E^*_{hB} = ½ \cdot E_h \cdot B$ on those sides of length B.

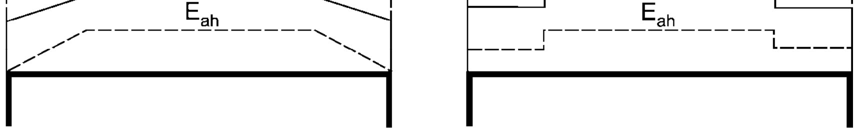
The distribution over the sides of the excavation follows from Paragraphs 2 and 3 as one of the shapes represented in Figure R 75-4. The flexibility of the supports governs selection of one of these shapes. The largest earth pressure should be anticipated where the displacement is smallest. The same applies accordingly for the longer sides of shaft-like excavations.



a) Reduction at the corners   b) Uniformly distributed load   c) Reduction towards centre

Figure R 75-4. Earth pressure on narrow excavation sides and shaft-like excavations

7. If, in exceptional cases, a retaining wall is designed for at-rest earth pressure according to R 23 (Section 9.6), no earth pressure reduction is warranted. When applying increased active earth pressure to retaining walls adjacent to structures, interpolation may be performed between the at-rest earth pressure and the active earth pressure, just as in the area without reduction. See Figures R 75-5 and R 75-6. Here, E_h designates the component of the design earth pressure from soil self-weight according to R 22 (Section 9.5). Paragraphs 2 to 5 govern application of the active earth pressure in the region of structures according to R 28 (Section 9.3) or according to R 29 (Section 9.4).



a) Earth pressure with chamfering   b) Earth pressure with steps

Figure R 75-5. Earth pressure in rectangular excavations with increased active earth pressure and earth pressure reduction at the excavation corners

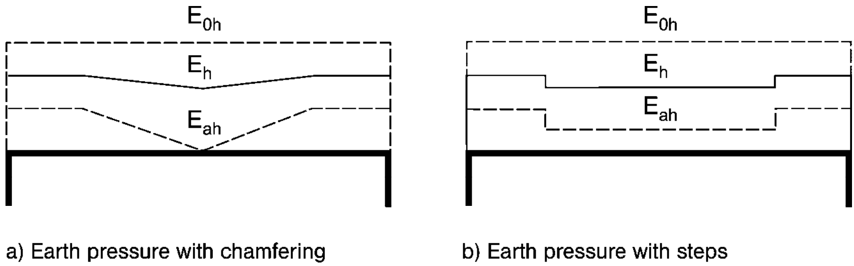


Figure R 75-6. Earth pressure in rectangular excavations with increased active earth pressure and earth pressure reduction at the excavation sides

8. The simplified approaches for earth pressure reduction at the excavation corners (Figure R 75-2) or the excavation sides (Figure R 75-3) represent, greatly simplified, the earth pressure reduced as a result of the three-dimensional effect. Three-dimensional, numerical analyses, capable of taking into account the dependence on the geometrical dimensions of the excavation and on the geotechnical relationships, have proven expedient for modelling the three-dimensional bearing and deformation behaviour of excavations with a rectangular plan [157, 158].
9. The same pressure diagrams may be selected for the distribution of earth pressure across the wall in the region of chamfers or steps as for the earth pressure E_h in regions without reduction.
10. The earth pressure from point loads, line loads or strip loads caused by road and rail traffic according to R 55 (Section 2.6), from site traffic and operations according to R 56 (Section 2.7) and from excavators or lifting equipment according to R 57 (Section 2.8), as well as the earth pressure from building loads according to R 28 (Section 9.3), R 29 (Section 9.4), R 22 (Section 9.5) and R 23 (Section 9.6), may not be reduced.
11. If the ground below the excavation level is utilised to support the wall, the passive earth pressure may be adopted as for an infinitely long wall. A three-dimensional effect at the corners may only be adopted on the basis of separate investigations.

9 Excavations adjacent to structures

9.1 Engineering measures for excavations adjacent to structures (R 20)

1. If structures and facilities are located within the zone of influence of an excavation, the impacts with regard to stability and serviceability of the structure shall be investigated. The required measures depend on the distance, the foundation depth, the structural condition of the building, the sensitivity to settlement, the use of the structure and the ground conditions. Furthermore, the elastic deformations, slippage and local deformations in braced excavations also play a role, in particular for long sets of struts consisting of a large number of individual components. On relatively flexible walls, in particular, struts or anchors are arranged in the foundation's load zone.

   Unsupported retaining walls that are only restrained in the ground are generally not permissible if the free wall height is within the projection zone of foundation loads. The region of the retaining wall lying below the point at which a line projecting from the front edge of the foundation intersects the retaining wall at an angle φ'_k is known as the load projection zone, see Figure R 28-1 a) (Section 9.3).

2. Soldier pile walls may be installed adjacent to structures with extreme caution under following conditions:

   a) if the infill walls comprising timber planks, prefabricated reinforced concrete elements or trench sheet piles are prestressed by wedging and cavities behind the infill walls can be ruled out with certainty;
   b) if the infill wall is manufactured using shotcrete or in-situ concrete;
   c) if the boreholes containing the soldier piles are backfilled with easily compacted soil. If the soil cannot be sufficiently compacted, binder additives are necessary.

3. If it is not possible or expedient to install a soldier pile wall, e.g.:

   – in cohesionless, uniformly graded soils;
   – in soft, cohesive soils;
   – if dewatering is not desirable ;
   – for small retaining wall-structure distances or;
   – for particularly sensitive structures;

   the installation of watertight and especially low-deformation retaining walls may be necessary, e.g. pressed sheet pile walls, diaphragm walls or bored pile walls. In special cases it may be expedient to underpin the structure completely or in part, or to apply soil stabilisation measures.

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4. When selecting the excavation structure it shall be noted that not every system is equally suitable due to influences arising from the construction process. The following may serve as examples:

   a) When driving or vibrating soldier piles and sheet pile walls, loosely compacted, cohesionless soils are compacted and dragged by the piles. This effect may be amplified by the driven objects impacting on obstructions.
   b) For pile walls in loosely compacted soils, soft soils or soils susceptible to liquefy, settlement in the immediate vicinity can ensue due to the unavoidable suction effect withdrawing soil, in particular below water. Also see R 92, Paragraph 3 (Section 12.3).
   c) Where slurry-supported bored piles or diaphragm walls are installed, intersecting voids, e.g. pipelines, can lead to a loss of slurry. Larger cavities shall be ruled out by performing sufficient investigations. As a safeguard against unanticipated cavities, sufficient quantities of slurry shall be held ready as well as implementing countermeasures according to R 92, Paragraph 3 (Section 12.3).
   d) The curing process for single-phase walls shall be observed during the manufacturing sequence.

5. In order to keep the anticipated wall displacement as small as possible it is advisable to:

   – use especially stiff walls;
   – use small spacing between the individual rows of struts or anchors;
   – restrict excavation advance to an unavoidable minimum before installing struts and anchors;
   – prestress the struts and anchors to more than 80 % of the characteristic stress computed for the subsequent construction stage;
   – if necessary, replace anchors with prestressed struts or other bracing elements.

   The degree of prestressing is given by R 8, Paragraph 4 (Section 3.1) for analysis of active earth pressure, R 22, Paragraph 4 (Section 9.5) for analysis of increased active earth pressure and R 23, Paragraph 8 (Section 9.6) for analysis of at-rest earth pressure.

6. For anchored retaining walls, it may be necessary to install all or at least some of the anchors below the structure to be stabilised in order to ensure that any ground displacements associated with a cofferdam effect are sufficiently small. Also see R 46, Paragraph 1 (Section 7.5) and [29, 39] and [72].

7. It may be necessary to carry out stabilising measures on the structure itself regardless of measures for stabilising the excavation. These include, for

example, measures to improve the connection between longitudinal and transverse walls, anchoring-back endangered sections of the structure to sections that are not within the zone of influence of the excavation, or installing brickwork in openings and using binded double walings in order to stiffen walls if the diaphragm action of these is in doubt.

8. The recommendations in R 20 shall be applied accordingly for cases in which sensitive installations may be endangered by constructing the excavation, e.g. pipelines or masts.

9.2 Analysis of retaining walls with active earth pressure for excavations adjacent to structures (R 21)

1. If the struts or anchors of a retaining wall are not prestressed more than stipulated in R 8, Paragraph 4 (Section 3.1), it shall be assumed that a horizontal wall deflection with a magnitude of 1° of the wall height will occur. Ground settlements occurring directly behind the retaining wall that can be up to twice as large as the horizontal wall deflections and only dissipate at large distances from the excavation, may be associated with this wall deflection [157, 158]. As far as the deformations caused by building the structure and the movements of the wall can be accepted, the excavation structure may be designed for active earth pressure.

2. For excavations adjacent to structures in general, the active earth pressure may also be determined on the basis of planar slip surfaces. However, in individual cases, where very large building loads and unfavourable ground layering are prevalent, it may be necessary to determine the earth pressure on the basis of curved or non-circular slip surfaces. Horizontal building loads shall always be taken into consideration. See R 6, Paragraph 6 (Section 3.4) and R 7, Paragraph 5 (Section 3.5) for further information.

3. The principal differentiation here is between:

   – earth pressure $E_{ah,k}$ from soil self-weight, uniformly distributed load $p_k \leq 10$ kN/m^2 and, if applicable, cohesion according to R 4 (Section 3.2), in conjunction with R 6, Paragraph 3 (Section 3.4) and;
   – earth pressure resulting from unbounded distributed loads over and above $p_k = 10$ kN/m^2, as well as additional strip loads q'_k according to R 55 to R 57 (Sections 2.6 to 2.8).

   However, according to R 104, Paragraph 5 (Section 4.11), it is generally permissible to increase these live loads by the factor f_q and to treat them as permanent actions, if they act unfavourably.

4. The magnitude and distribution of the earth pressure on a retaining wall adjacent to a structure depend greatly on the distance and the foundation depth. Two cases are differentiated here:

   a) large distance to structures, see R 28 (Section 9.3);
   b) small distance to structures, see R 29 (Section 9.4).

   The governing differentiation is whether a straight line touches the front edge of the foundation at a smaller angle (Figure R 21-1 a) or a greater one (Figure R 21-l b) than the slip surface at an angle $\vartheta_{a,k}$ for soil self-weight and cohesion alone.

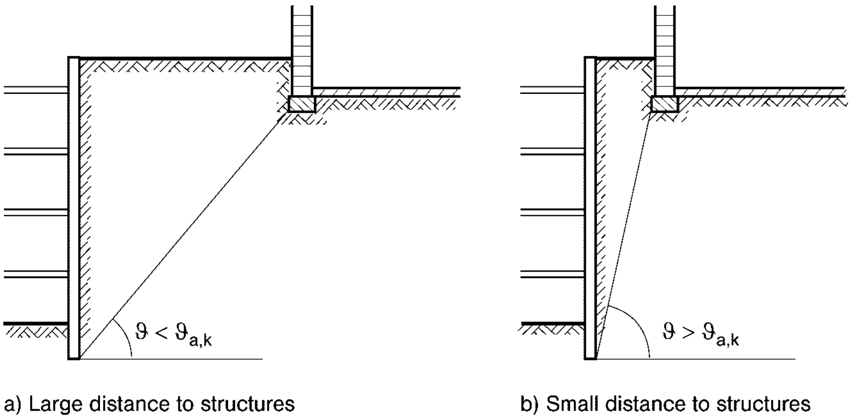

Figure R 21-1. Distance between retaining wall and structure

5. For soldier pile walls, only those portions of the earth pressure occurring above the excavation level are incorporated into the redistribution pressure diagrams according to R 28 (Section 9.3) or R 29 (Section 9.4). When analysing the equilibrium condition $\Sigma H = 0$ according to R 15 (Section 5.5), the earth pressure from building loads occurring below the excavation level shall be taken into consideration (Figures R 28-1 d) and e), Section 9.3).

6. The passive earth pressure is adopted when analysing the embedment depth:

   a) according to R 14 (Section 5.3) or R 19 (Section 6.3) in the case of free earth support;
   b) according to R 25 (Section 5.4) or R 26 (Section 6.4) in the case of earth restraint.

   See R 81 (Section 4.1) and R 82 (Section 4.4) for determination of the action effects.

7. See R 9 (Section 4.7) for analysis of the equilibrium of vertical forces.
8. See R 83 (Section 4.10) for the serviceability analysis.

9.3 Active earth pressure for large distances to structures (R 28)

1. If the conditions for adopting a large distance between the retaining wall and other structures given in R 21, Paragraph 4 (Section 9.2) are met, the earth pressure magnitude shall be determined in two ways:
   a) The earth pressure $E_{ah,k}$ is obtained for a slip surface at an angle $\vartheta_{a,k}$, intersecting the ground surface in front of the structure. Also see Paragraph 2.
   b) The earth pressure $E_{zh,k}$ is obtained for a slip surface at an angle $\vartheta_{z,k}$, originating at the rear edge of the foundation as shown in Figure R 28-1 a). Also see Paragraph 3.

   The greater earth pressure determines further analysis.
2. The general rules given in Chapters 3 to 6 apply with regard to the magnitude and distribution of the earth pressure E_{ah}.
3. The magnitude of the earth pressure $E_{zh,k}$ from the actions discussed in Paragraph 2 and the actions from building loads is obtained according to R 71 (Section 3.6). The earth pressure $E_{zBh,k}$ from the building load is obtained from the earth pressure $E_{zh,k}$ minus the earth pressure $E_{ah,k}$ according to Paragraph 2. For a relatively small angle $\vartheta_{z,k}$, $E_{aBh,k}$ can become very small or even zero. In approximation, the building load's zone of influence can be assumed as shown in Figure R 28-1 a). The upper boundary thus lies between the level of the foundation base and the point at which a straight line originating at the front edge of the foundation, and projected at an angle $\leq \varphi'_k$ to the horizontal, intersects the rear face of the wall. The lower boundary is at the level of the wall toe. The horizontal component shall also be taken into consideration for inclined foundation loads. See R 6, Paragraph 6 (Section 3.4) and R 7, Paragraph 5 (Section 3.5) for further information.
4. Generally, that portion of the earth pressure $E_{ah,k}$ from soil self-weight, uniformly distributed load $p_k \leq 10$ kN/m^2 and, if applicable, cohesion according to R 4 (Section 3.2), in conjunction with R 6, Paragraph 3 (Section 3.4), may be converted to a realistic pressure diagram extending from ground level to the excavation level. The lower boundary of the earth pressure redistribution may also be assumed at a deeper point, if:
   a) for soldier pile walls according to R 5, Paragraph 3 b) (Section 3.3) a greater upward earth pressure redistribution is necessary in order to analyse $\Sigma H = 0$ according to R 15, Paragraph 6 c) or Paragraph 7 c) (Section 5.5).

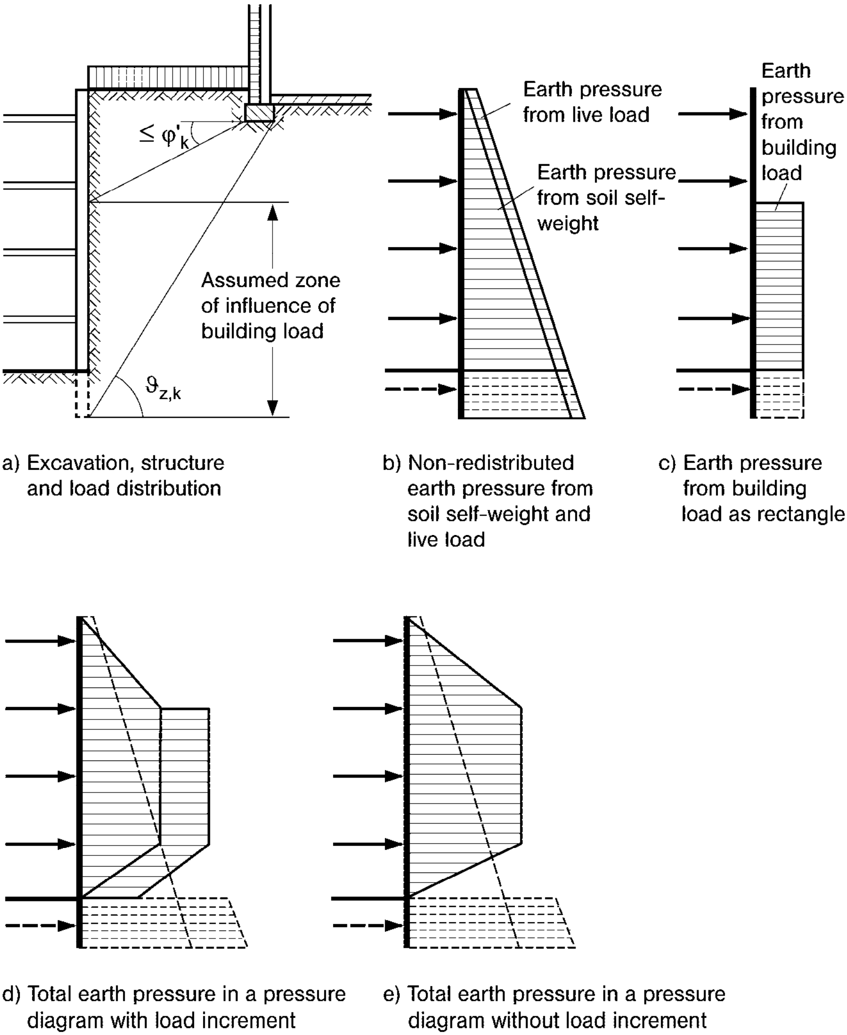

Figure R 28-1. Distribution of active earth pressure taking into consideration the influence of a building load with large distance between retaining wall and structure (example for a soldier pile wall with free-earth support)

b) for sheet pile walls or in-situ concrete walls according to R 5, Paragraph 3 c) (Section 3.3) a greater earth pressure redistribution is aimed for and supported by appropriate prestressing of the upper rows of struts or anchors.

The earth pressure from building loads may be incorporated into this pressure diagram taking into consideration the zone of influence according to Paragraph 3, so that any sudden alteration in the earth pressure ordinate lies within the area of a support point (Figure R 28-1 d), or so that no sudden alteration of the earth pressure ordinate occurs (Figure R 28-1 e).

5. According to R 104, Paragraph 5 (Section 4.11), it is generally permissible to increase the building live load by the factor f_q and to then treat it as a permanent action, together with the building dead weight.

9.4 Active earth pressure for small distances to structures (R 29)

1. If the conditions for adopting a short distance between the retaining wall and other structures given in R 21, Paragraph 3 (Section 9.2) are met, it is convenient to determine the earth pressure $E_{ah,k}$ from soil self-weight, uniformly distributed load $p_k \leq 10$ kN/m^2 and, if applicable, cohesion, or, alternatively, the minimum earth pressure according to R 4, Paragraph 5 (Section 3.2), in conjunction with R 6, Paragraph 3 (Section 3.4), separately for the following load components:
   a) for the self-weight of the soil above the foundation base between the retaining wall and the structure and for the effective live load between the retaining wall and the structure.
   b) for the self-weight of the soil below the foundation base, for the self-weight of the soil above the foundation base and within the structure and the basement floor, and for a live load acting on the basement floor.

2. The earth pressure from the live load and the self-weight of the soil above the foundation base between the retaining wall and the structure is adopted from a slip surface as shown in Figure R 29-1 a) at an angle $\vartheta_{a,k}$ projected from the front edge of the foundation (Figure R 29-1 b). The earth pressure determined in this way is redistributed to the region between ground level and the intersection of the assumed slip surface with the retaining wall, according to R 12, Paragraph 3 (Section 5.1), or R 16, Paragraph 3 (Section 6.1) (Figure R 29-1 d), taking cohesion into consideration if applicable.

3. The earth pressure determined from the self-weight of the soil below the foundation base is redistributed to the region between the foundation base and the excavation level for soldier pile walls, sheet pile walls and in-situ concrete walls, unless it is a special case according to R 15, Paragraph 5 c) or Paragraph 6 c) (Section 5.5) taking cohesion into consideration, if applicable. The soil self-weight above the foundation base in the region of the structure can be converted to a surcharge and adopted as a uniformly distributed load together with the dead weight of the basement floor and any live load $p_k \leq 10$ kN/m^2 in the basement.

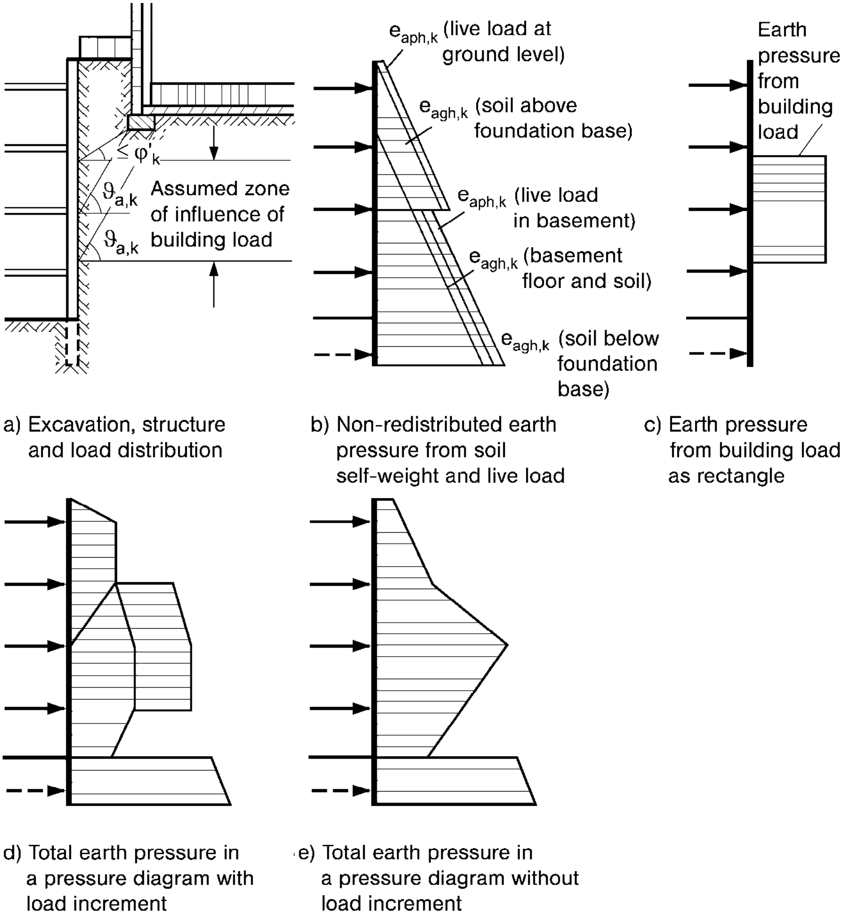

Figure R 29-1. Distribution of active earth pressure taking into consideration the influence of a building load with small distance between retaining wall and structure (example for a sheet pile wall or in-situ concrete wall with free-earth support)

4. The earth pressure from the building load $E_{aBh,k}$ is obtained according to the information in R 6, Paragraph 3 (Section 3.4), assuming a slip surface angle $\vartheta_{a,k}$. In approximation, the zone of influence of the building load may be assumed as shown in Figure R 29-1 a) and the distribution of earth pressure from a building load as a uniformly distributed load as shown in Figure R 29-1 c). If two or more foundations influence the magnitude of the earth pressure, the individual foundation earth pressure forces are first determined separately and then superimposed. The horizontal component

shall also be taken into consideration for inclined foundation loads. See R 6, Paragraph 6 (Section 3.4) and R 7, Paragraph 5 (Section 3.5) for further information.

5. In principle, the earth pressure $E_{aBh,k}$ from building loads shall be divided into a permanent component $E_{aBgh,k}$ from building dead weight and a variable component $E_{aBgh,k}$ from building live loads. According to R 104, Paragraph 5 (Section 4.11), however, it is generally permissible to increase the building live load by the factor f_q and to then treat it as a permanent load together with the building dead weight. Estimates may be used if as-built documentation is not available.

6. The pressure diagrams determined according to Paragraphs 2 to 4 may be superimposed. The resulting total pressure diagram may be selected as shown in Figure R 29-1 d) or Figure R 29-1 e). The earth pressure from the building load may be incorporated into the pressure diagram for the lower earth pressure component taking the zone of influence into consideration according to Paragraph 2.

9.5 Analysis of retaining walls with increased active earth pressure (R 22)

1. If the horizontal deflection of a retaining wall, and thus the settlement behind the wall, needs to be more heavily restricted than stipulated in R 21, Paragraph 1 (Section 9.2), according to R 8, Paragraph 3 (Section 3.1), using the measures stipulated in R 20, Paragraph 4 (Section 9.1), the excavation structure shall be designed for increased active earth pressure.

2. For large distances to structures according to R 28 (Section 9.3) the mean value:

   $$E_{h,k} = 0.50 \cdot (E_{0h,k} + E_{0Bh,k}) + 0.50 \cdot (E_{ah,k} + E_{aBh,k})$$

   between the horizontal component of the earth pressure $E_{0,k}$ and the horizontal component of the active earth pressure $E_{a,k}$ is generally sufficient. Where structures and installations are not sensitive the following earth pressure is sufficient:

   $$E_{h,k} = 0.25 \cdot (E_{0h,k} + E_{0Bh,k}) + 0.75 \cdot (E_{ah,k} + E_{aBh,k}).$$

   Where structures and installations are sensitive it is necessary to adopt the following earth pressure:

   $$E_{h,k} = 0.75 \cdot (E_{0h,k} + E_{0Bh,k}) + 0.25 \cdot (E_{ah,k} + E_{aBh,k}).$$

   The magnitude of the characteristic at-rest earth pressures and the characteristic active earth pressures shall be determined according to Paragraph 4.

3. The following approaches apply for small distances to structures according to R 29 (Section 9.4):

   a) $E_{h,k} = 0.25 \cdot E_{0h,k} + 0.75 \cdot E_{ah,k} + E_{aBh,k}$ for non-sensitive structures and installations;
   b) $E_{h,k} = 0.50 \cdot E_{0h,k} + 0.50 \cdot E_{ah,k} + E_{aBh,k}$ in normal cases;
   c) $E_{h,k} = 0.75 \cdot E_{0h,k} + 0.25 \cdot E_{ah,k} + E_{aBh,k}$ for sensitive structures and installations.

   The magnitude of the characteristic at-rest earth pressure $E_{0h,k}$ and the characteristic active earth pressures shall be determined according to Paragraph 4. Adopting $E_{aBh,k}$ as stipulated takes into consideration that the active earth pressure from building loads is numerically greater than the at-rest earth pressure from building loads.

4. The variables discussed in Paragraphs 2 and 3 are obtained as follows:

   a) The magnitude of the characteristic at-rest earth pressure $E_{0h,k}$ from soil self-weight, unbounded distributed load and, if applicable, cohesion, as well as the magnitude of the characteristic at-rest earth pressure $E_{0Bh,k}$ from building loads, shall be determined according to R 18 (Section 3.7).
   b) The magnitude of the characteristic active earth pressure $E_{ah,k}$ from soil self-weight, unbounded distributed load and, if applicable, cohesion, or alternatively the minimum earth pressure, as well as the magnitude of the characteristic active earth pressure $E_{aBh,k}$ or $E_{zBh,k}$ resulting from building loads shall be determined according to R 28 (Section 9.3) for large distances to structures and according to R 29 (Section 9.4) for small distances to structures.

5. In the case of an earth pressure lying between the active earth pressure and the at-rest earth pressure, it can be assumed that earth pressure redistribution occurs in a similar manner to active earth pressure but with a tendency to decrease the greater the ratio of at-rest earth pressure to earth pressure This earth pressure may therefore also be converted to a simple pressure diagram with the bending points or sudden load alterations in the region of the support points (Figure R 22-1 d). The difference between structures with large distances and those with small distances to the retaining wall according to R 28 (Section 9.3) and R 29 (Section 9.4) also applies accordingly for increased active earth pressure. If only the struts or anchors in the zone of influence of the building load are especially highly prestressed, the earth pressure in this area is assumed to be more concentrated (Figure R 22-1 e). Adjacent structures and installations may not be subjected to additional loads from increased prestressing of anchors or struts without analysis.

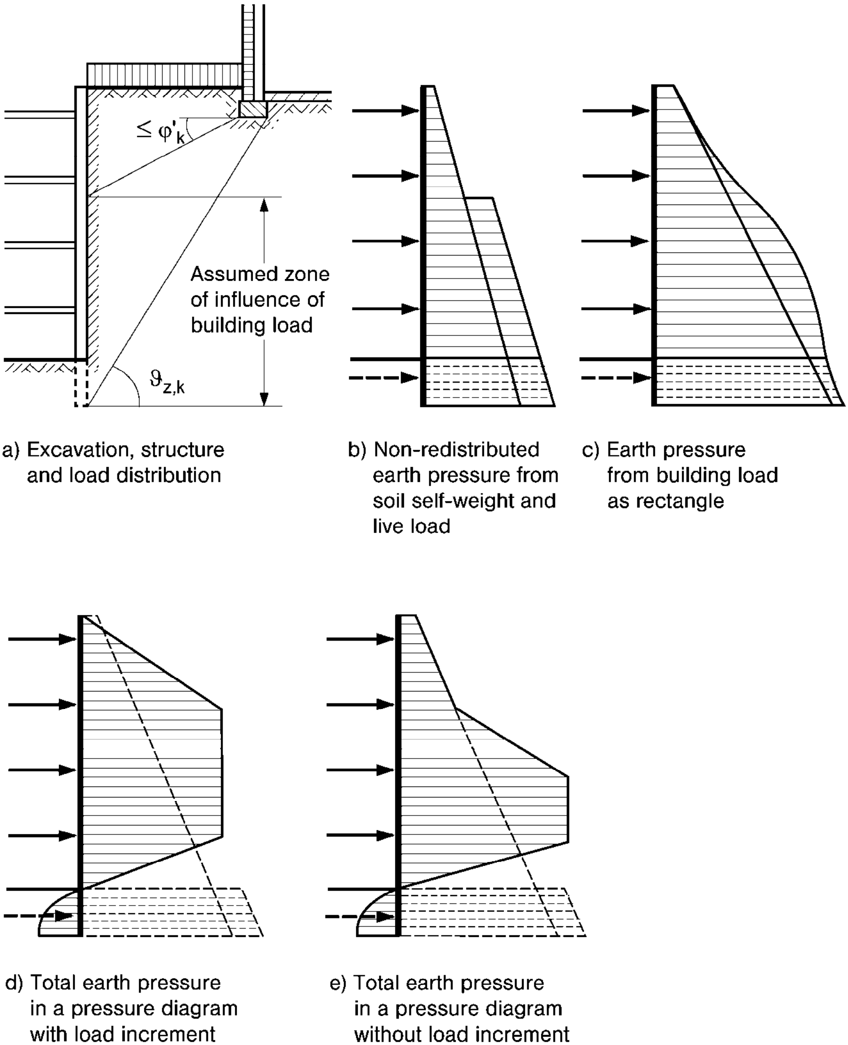

Figure R 22-1. Distribution of increased active earth pressure taking into consideration a building load with large distance between retaining wall and structure (example for a soldier pile wall with free-earth support)

6. The passive earth pressure is determined:
   a) according to R 14 (Section 5.3) or R 19 (Section 6.3) in the case of a free earth support;
   b) according to R 25 (Section 5.4) or R 26 (Section 6.4) in the case of an earth restraint;

but with the stipulation that in order to reduce base deflections in at least medium-dense, cohesionless, or at least plastic, cohesive soils the design passive earth pressure must be reduced by the factor:

– $\eta_{Ep} \leq 0.6$ for walls contiguous in the base area;
– $\eta_{Ep} \leq 0.8$ for walls continuous in the base area.

If the ground in the wall base area consists of soft, cohesive soil a design shall be selected that does not require an earth support.

7. The information in chapter 4 applies for analysis of the embedment depth and for determination of the design action effects. The governing partial safety factor for permanent actions shall consist of the same ratios of the partial safety factors γ_{E0g} and γ_G as the relevant characteristic earth pressure $E_{h,k}$ according to Paragraph 2, for large distances to buildings, or according to Paragraph 3, for small distances to buildings. The main safety factors are the partial factors for the design situations DS-T or DS-T/A given in table 6.1 in appendix A 6. In the case of Paragraph 3, the partial safety factors for active earth pressure govern the $E_{aBh,k}$ component.

8. The vertical earth pressure components consist of the vertical components of the at-rest earth pressure and the active earth pressure, similar to the horizontal components. It shall be demonstrated that the vertical component of the design earth pressure can be transferred to the ground by the retaining wall according to R 9 (Section 4.7), and that subsequent settlements have no detrimental impact on the adjacent structure. It may be necessary to forgo the adoption of an earth pressure angle when determining the active earth pressure. This results in additional vertical loads adjacent to the retaining wall due to the circumvented distribution of the building load in the subsurface, which can lead to intolerable settlement. Also see Figure R 22-2.

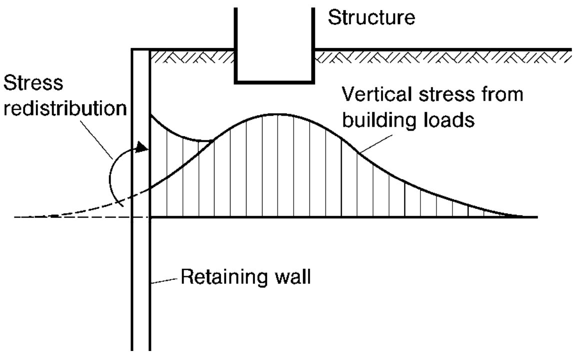

Figure R 22-2. Stress redistribution for circumvented load distribution

9. Even if the earth pressure is based on the increased active earth pressure, it shall be demonstrated that $\Sigma H = 0$ for soldier pile walls in accordance with R 15 (Section 5.5). The earth pressure acting below the excavation level shall be adopted in the same ratio as the active earth pressure and at-rest earth pressure acting above the excavation level. If a building load also acts below the excavation level, this shall be taken into consideration. Paragraph 6 applies for the design passive earth pressure calibration factor.
10. Generally, it is not necessary to prestress the struts and anchors for the new, computed characteristic load at each new construction stage. It is generally sufficient to prestress the struts and anchors for the characteristic support forces projected for the fully excavated stage from the outset, including in the advancing states. However, it is possible that the row above the last installed row unloads somewhat when the current row is prestressed. Post-stressing for possibly greater support forces occurring during the retreating states can generally also be dispensed with . However, monitoring the movements of the structure and the retaining wall by taking measurements where sensitive structures are involved is recommended, as well as monitoring the stresses on the struts or anchors, and providing for post-stressing measures where necessary.
11. See R 83 (Section 4.10) for details of the serviceability analysis. The notes on the possible prevention of load distribution and associated settlements in Paragraph 8 shall be observed. Use of the finite-element method according to R 103 (Section 4.6) is recommended for more precise investigations.

9.6 Analysis of retaining walls with at-rest earth pressure (R 23)

1. It is generally recommended to adopt the earth pressure at either $E_{h,k} = 0.75 \cdot (E_{0h,k} + E_{0Bh,k}) + 0.25 \cdot (E_{ah,k} + E_{aBh,k})$ (for large distances to structures) or $E_{h,k} = 0.75 \cdot E_{0h,k} + 0.25 \cdot E_{ah,k} + E_{aBh,k}$ (for small distances to structures) in accordance with R 22, Paragraphs 2 and 3 (Section 9.5) for sensitive structures and installations adjacent to the retaining wall. Ground unloading can be avoided by using continuous walls and, in addition, an inflexible support according to R 67, Paragraph 5 (Section 1.5) and it is necessary to adopt the at-rest earth pressure on the retaining wall, even if this provides no guarantee that settlement of adjacent structures will not occur.
2. The magnitude and distribution of the at-rest earth pressure are obtained according to R 18 (Section 3.7). The following points apply when defining the pressure diagram:
   a) The at-rest earth pressure from soil self-weight is assumed to increase linearly with depth if the base is stiffened at an early stage before the excavation level is reached (Figure R 23-1 a). If the ground beneath the

excavation level is utilised to a large degree for wall support, the full at-rest earth pressure can no longer act in this region due to the unavoidable displacement of the wall toe. In such cases, therefore, the earth pressure ordinate may be assumed as being constant from the lowest row of supports downwards for retaining walls with at least two rows of struts or anchors (Figure R 23-l b). It cannot be assumed that the full at-rest earth pressure is maintained for single-propped walls without timely bracing of the base. One exception to this is the construction stage before the second set of struts is installed as shown in Figure R 23-1 b), due to the remaining large wall embedment depth.

b) The pressure diagrams described in Paragraph a) for the at-rest earth pressure from an unbounded distributed load $p_k \leq 10$ kN/m^2 are superimposed with an ordinate remaining uniform for the entire height of the wall.

c) The at-rest earth pressure from vertical or horizontal building loads may be converted to a simple pressure diagram. It shall begin approximately at the level of the base of the building and the resultant shall be approximately at the point of intersection of a line at 45° from the horizontal with the rear of the retaining wall, originating at the load axis of the base of the structure. See Figure R 23-3 for examples.

d) To determine the characteristic action effects, the pressure diagram resulting from superimposing the individual at-rest earth pressure components may be simplified in a way that, for an unchanged total load magnitude, a pressure diagram ensues which displays no sudden hanges (Figure R 23-2 d) and (Figure R 23-2 e), or for which a sudden change

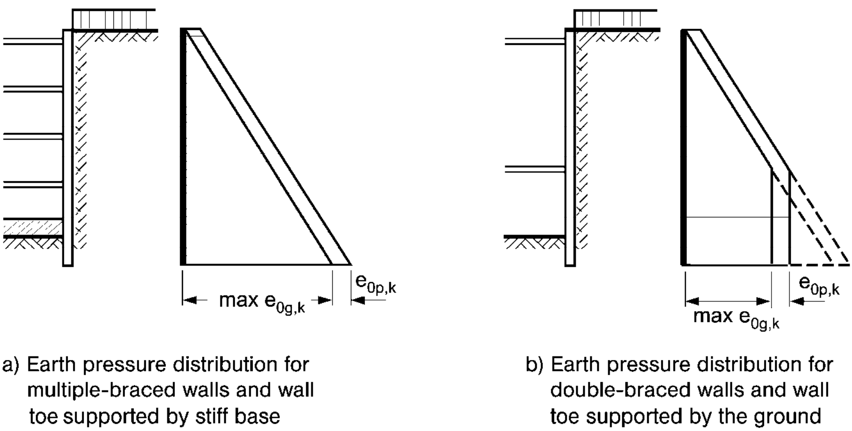

Figure R 23-1. Load model determination for in-situ concrete walls adopting at-rest earth pressure

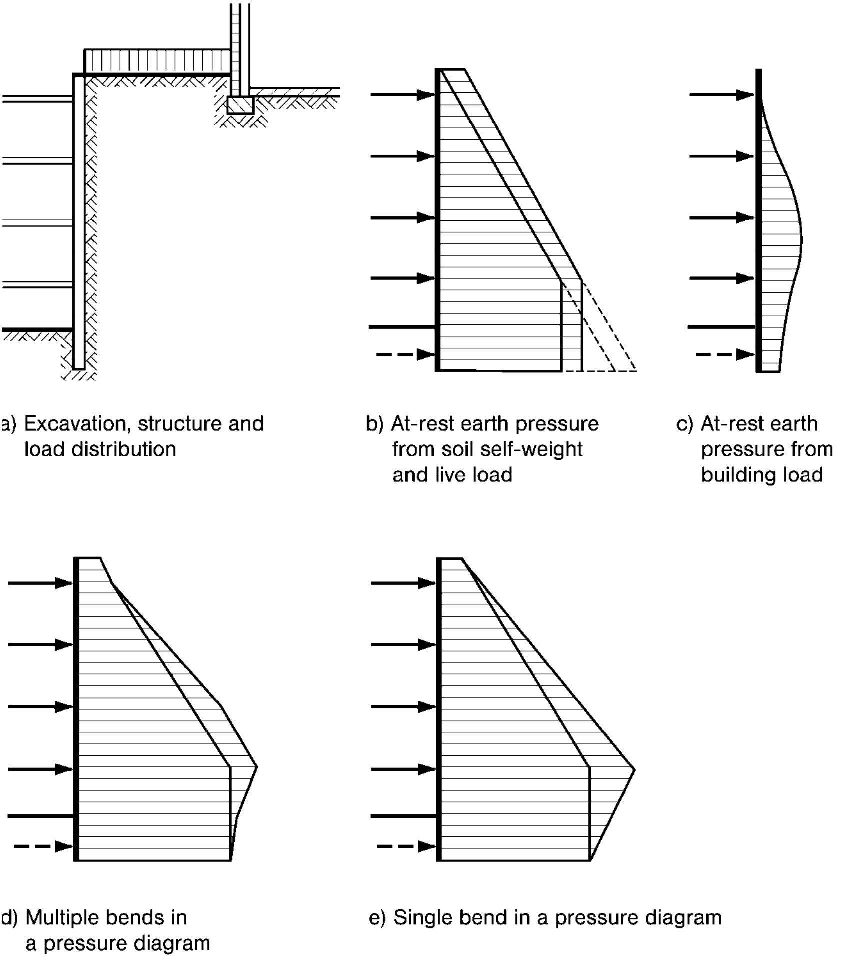

Figure R 23-2. At-rest earth pressure distribution for a sheet pile wall or in-situ concrete wall with free earth support and consideration of the influence of a building load (example of an in-situ concrete wall with free earth support)

lies at a support point. This also applies to the variable component $E_{0Bh,k}$ of the earth pressure from building loads, if the simplification according to R 28, Paragraph 5 (Section 9.3) or R 29, Paragraph 5 (Section 9.4) is adopted.

3. The passive earth pressure is adopted according to R 19 (Section 6.3), because a geotechnical restraint is not generated in the ground for a stiff retaining wall. However, in order to reduce the toe deflections in medium-

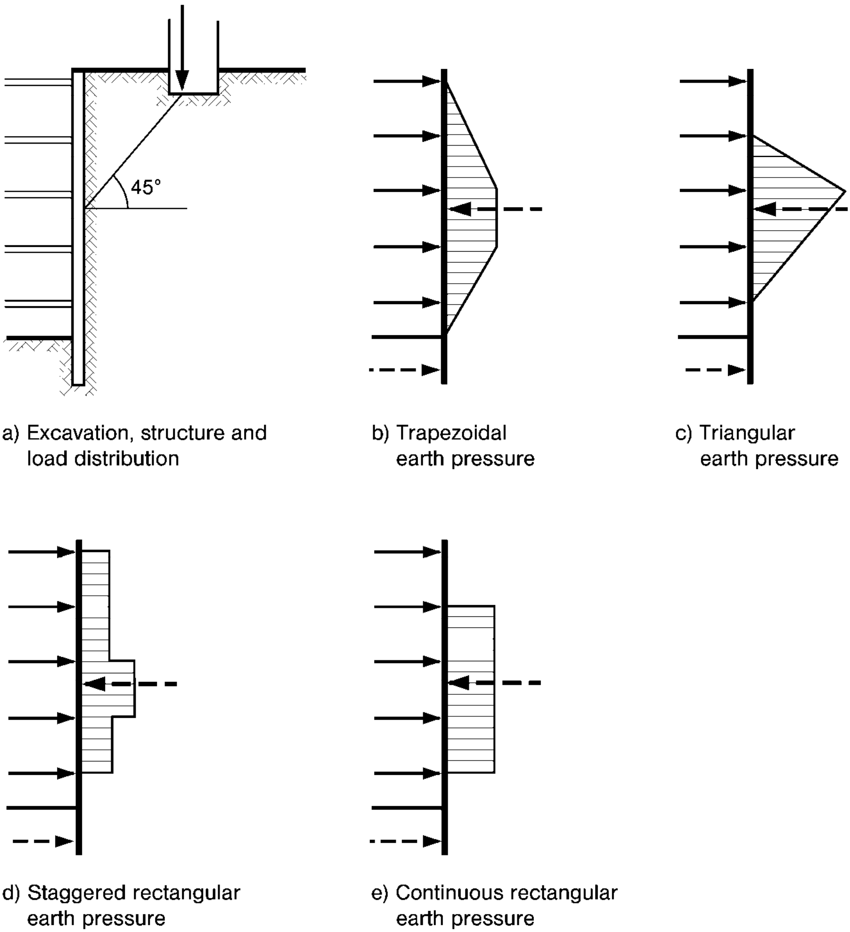

Figure R 23-3. At-rest earth pressure approximations for building loads and non-yielding retaining walls

dense or dense soils or at least stiff, cohesive soils the design passive earth pressure is reduced by the calibration factor $\eta_{Ep} \leq 0.5$. If loosely compacted, cohesionless soil occurs below the excavation level the calibration factor shall be reduced further or a design be selected that does not require an earth support.

4. If the wall is deflected sufficiently below a stiffened base a ground reaction may be adopted on the ground side resulting in a support moment at the level of the stiffened base. Also see Figure R 63-3 b) (Section 10.6).

5. The information given in section 4 applies to the determination of the embedment depth and the design action effects. The governing factor is the partial safety factor γ_{E0g} for permanent actions as a function of the load case as shown in table 6.1 in Appendix A 6.
6. Deformations of the retaining wall may lead to the actual characteristic earth pressure distribution deviating from the assumed at-rest earth pressure distribution as shown in Figure R 23-1 (Section 9.6). Instead of a precise analysis using a redistributed active earth pressure in accordance with R 28 (Section 9.3) or R 29 (Section 9.4) it is permissible to design the struts and anchors in the upper third of the wall for characteristic support forces that are 30 % greater than the support forces determined using the at-rest earth pressure only.
7. It shall be demonstrated that the vertical component of the characteristic at-rest earth pressure from soil self-weight (for an inclined ground surface) and the at-rest earth pressure from the building load can be transferred by wall friction to the retaining wall at every point of the wall, adopting the characteristic earth pressure angle tan $\delta_{a,k}$, and can be transferred by the wall to the subsurface without appreciable settlement according to R 9 (Section 4.7). If this cannot be demonstrated, preservation of the original stress state is not guaranteed and adoption of the at-rest earth pressure is not justified.
8. If the at-rest earth pressure is adopted the wall deformations shall be monitored using measurements. Struts and anchors shall be prestressed to the characteristic load upon installation and post-stressed if necessary.
9. See R 83 (Section 4.10) for details of the serviceability analysis.

9.7 Mutual influence of opposing retaining walls for excavations adjacent to structures (R 30)

1. If a horizontally braced excavation is only subject to earth pressures from structures on one side of the excavation, both walls can generally be designed according to the analysis for the retaining wall adjacent to the structure if no more precise analysis is performed. For example, if, for structures close to the retaining wall (Figure R 30-1), lower strut forces occur in the upper section of the side of the excavation subject to the building load, the same deliberations shall be made as if there are structures on both sides. See Paragraphs 3 and 4.
2. If the retaining walls in a horizontally braced excavation subject to earth pressure from structures on one side only are differently designed, the retaining wall opposite the structure can, in approximation, be designed for

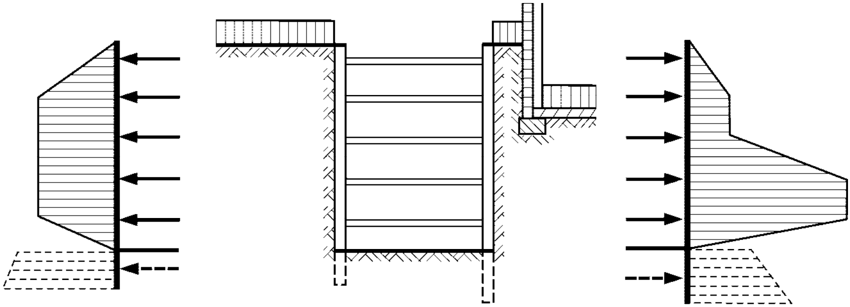

Figure R 30-1. Excavation with horizontal bracing and one-sided loading from a structure

the same action effects as the wall adjacent to the structure, if this wall does not substantially differ from the retaining wall adjacent to the structure with regard to stiffness and embedment depth. If they do differ substantially it may be necessary to separately investigate the wall opposite the structure. The characteristic support forces of the retaining wall subject to building loads shall be applied as loads to the retaining wall opposite the structure. The pressure diagram for this structure shall then be selected as appropriate for the prevalent loads, stiffness conditions and earth pressure theory.

3. For horizontally braced retaining walls subject to building loads on both sides of the excavation (Figure R 30-2), each retaining wall shall be inves-

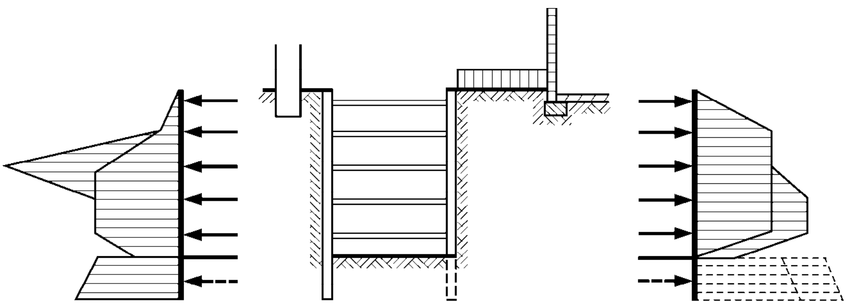

Figure R 30-2. Excavation with horizontal bracing and bilateral loading from a structure tigated separately. If this procedure results in different pressure diagrams on each side of the excavation, the respectively larger load ordinates from each wall shall be adopted for the opposing wall, in the case of similar stiffness conditions, and both walls be designed for the same resultant pressure diagram, with the exception of the zone below the excavation level (Figures R 30-3 and R 30-4). If the stiffness conditions for the two retaining walls are grossly dissimilar the respective pressure diagrams shall be developed so that roughly similar support forces result.

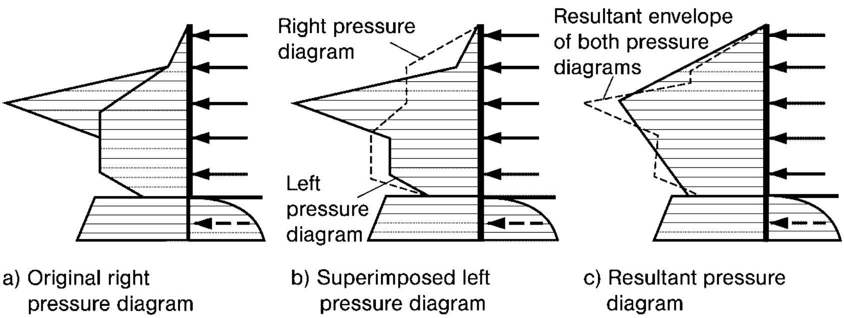

Figure R 30-3. Superimposing pressure diagrams on the left of the excavation

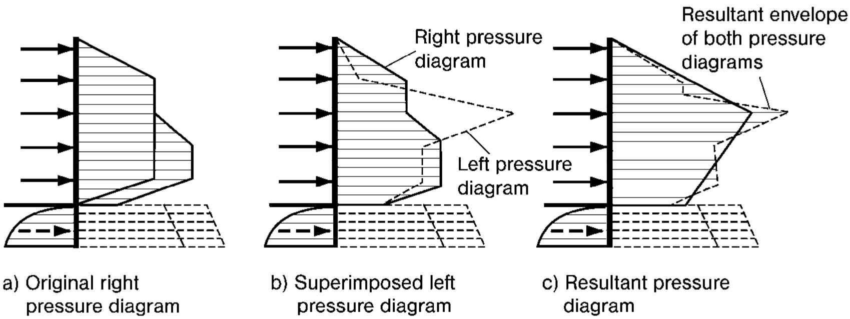

Figure R 30-4. Superimposing pressure diagrams on the right of the excavation

4. When transferring large strut forces from an opposite retaining wall to the retaining wall with the lesser earth pressure load, it shall be examined whether the higher earth pressure can be accepted and transferred.

10 Excavations in water

10.1 General remarks on excavations in water (R 58)

1. In principle, the following cases may be differentiated with regard to the varying manifestations of water associated with excavations:

   – open water, e.g. lakes, rivers;
   – free (phreatic) groundwater;
   – confined groundwater.

2. Where excavations employ water management measures the following cases are differentiated:

   a) If drawdown is performed as shown in Figure R 58-l a), both horizontal and downward directed flow forces occur in the soil mass pertinent to the excavation structure. In this context, R 59 (Section 10.2) shall be observed when determining the flow forces and R 60 (Section 10.3) when analysing the stability of the excavation structure.
   b) Where percolation around the wall toe occurs as shown in Figure R 58-1 b), upward directed flow forces also occur. In this context, R 59 (Section 10.2) shall be observed when determining the flow forces, R 61 (Section 10.4) when analysing the hydraulic heave safety of the excavation level and R 63 (Section 10.6) when analysing the stability of the excavation structure.
   c) If a practically impermeable soil layer is present below the excavation level, e.g. where a deep sealing base is employed as shown in Figure R 58-1 c), the flow of water into the excavation is prevented and a hydrostatic pressure develops. In this context, R 62 (Section 10.5) shall be observed when analysing the buoyancy safety of the excavation level and R 63 (Section 10.6) when analysing the stability of the excavation structure.

   In special cases it may be expedient to keep the water level higher on the inside than on the outside of the excavation, at least for a certain period of time, e.g. when excavating for an underwater concrete base.

3. Where retaining walls in cohesionless soils and soft to stiff, cohesive soils are involved, it may be assumed that the intimate contact between the retaining wall and the ground, and thus the flow net, are also retained if small displacements or deformations occur as a result of earth and water pressure. However, if the ground behind the retaining wall does not possess sufficient lateral deformability, e.g. rock-like ground or a hard or nearly hard, cohesive soil, which is at least temporarily stable without support due to its

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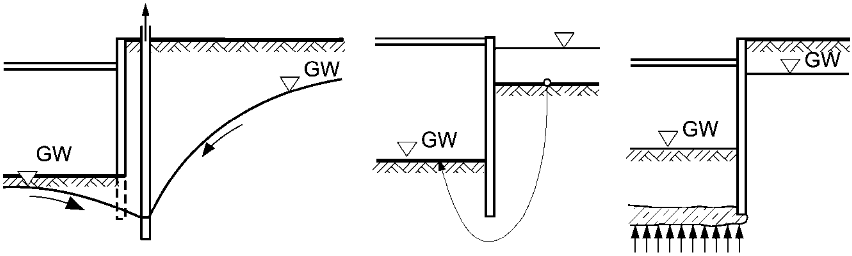


Figure R 58-1. Impacts of water on excavation structures

shear strength, formation of a gap between the retaining wall and the ground is possible, in which hydrostatic pressure corresponding to the external water level can develop.

4. In loose sand and silt in particular there is a danger of piping failure, which begins with increased local flow at the excavation level, progresses by flushing out soil particles in a tube-like formation (piping) and subsequently leads to a sudden inrush of water if a heavily water-bearing layer or open water is met. Piping failure is difficult to assess numerically and can only be avoided by constructive measures. See Recommendation R 116 [2] of the EAU, and R 64, Paragraph 14 (Section 10.7) of this publication.

5. Shortening of the flow path, presenting a hazard to the retaining wall, can occur if leakage zones arise between the individual elements when constructing the retaining wall and are not noticed in due time. A similar phenomenon can develop if water-bearing voids reaching deep into the ground in slightly permeable, slightly cohesive soil occur, e.g. poorly backfilled boreholes or other voids caused by pulling out piles. In this case the water finds its way under high pressure, again in a tube-like formation similar to piping failure, to the excavation level. See Recommendation R 116 [2] of the EAU, and R 64, Paragraph 14 (Section 10.7) of this publication for possible structural countermeasures.

6. The highest water level at which the excavation structure shall remain stable shall be stipulated according to R 24, Paragraph 1 e) (Section 2.1). Appropriate safety measures against higher water levels shall be provided for, e.g. controlled flooding according to R 64, Paragraph 14 (Section 10.7).

7. According to R 65, Paragraph 4 (Section 10.8) the groundwater within the excavation shall be lowered to approximately 0.50 m below the excavation level. In the following simplified figures, the computed water table is shown at the excavation level.

8. When designing watertight bases using soil stabilisation, DIN 4093:2012 shall be observed.

10.2 Flow forces (R 59)

1. Flow forces develop if a potential difference, which induces groundwater flow, is present as shown in Figure R 58-1 a) or Figure R 58-l b) (Section 10.1). The flow force is a mass force, which is transferred from the water to the soil skeleton due to the flow resistance in the direction of water flow. In the special case of vertical flow this has the effect of altering the unit weight of the percolated soil. If the flow is directed from top to bottom, this mathematical unit weight increases, if it flows from bottom to top, the mathematical unit weight is reduced.
2. The flow forces are calculated with the aid of the groundwater potential. In principle, this can be determined in one of two ways:
   a) If the groundwater potential at any point in the subsurface is required a flow net is used. It is determined based on EAU, Recommendation R 113 [2], e.g. using numerical analyses based on potential theory.
   b) If the groundwater potential at individual, specific points only is required, e.g. at the toe of a retaining wall, graphs and tables or simple mathematical approaches may be utilised for uniformly permeable ground [26, 56, 57, 58]. The flow forces can also be computed from the information given in the EAU, Recommendation R 114 [2], and in DIN 4085, for determining the change in unit weight of the soil resulting from flow force. However, this approach can only be applied to excavations if they are at least twice as wide as the difference in pressure head between the outer and inner water levels.
3. The flow forces in homogeneous soils are determined independent of the value of the coefficient of permeability. The governing factor is not the quantity of flowing water, but the difference in the potentials between external and internal water level.
4. The following points apply with regard to the permeability of the ground:
   a) Because the potential dissipation is always concentrated in the less permeable layers, alternating vertical permeability due to ground stratification shall always be taken into consideration when determining the flow forces. Also see R 61, Paragraph 6 (Section 10.4). In particular, the possibility of horizontal water ingress through the more permeable layers shall be examined.
   b) If the ratio of the horizontal to the vertical permeability $k_h/k_v \leq 3$ as a result of the natural anisotropy of the ground, this difference is gener-

ally not adopted. If in doubt, e.g. where horizontal flow lines are long, the anisotropy shall be taken into consideration during the investigation [159, 160].

c) The groundwater flow boundary conditions, in particular with regard to the inflow conditions, shall be realistically modelled for the numerical analysis of the flow net.

5. See R 63 (Section 10.6) for details of mathematical determination of the impact of flowing groundwater on positive water pressure, earth pressure and passive earth pressure.

10.3 Dewatered excavations (R 60)

1. If the groundwater is lowered as shown in Figure R 58-1 a) (Section 10.1) in order to dewater an excavation, investigations to determine the impact of flow forces on the stability of the excavation structure shall be performed. If necessary, the flow forces shall be taken into consideration for stability analysis.

2. In highly permeable soils the profile of the water surface is generally so flat that the groundwater has no impact on earth pressure. In silts and fine-sands, however, the drawdown curve may be so steep that it intersects the failure slip surface and influences the magnitude of the active earth pressure (Figure R 60-l a). This condition may occur for a short time only during the groundwater drawdown phase. It is then classified as design situation DS-T/A.

3. Complete groundwater drawdown in slightly permeable soil layers or where there are several aquifer levels is often only possible by applying

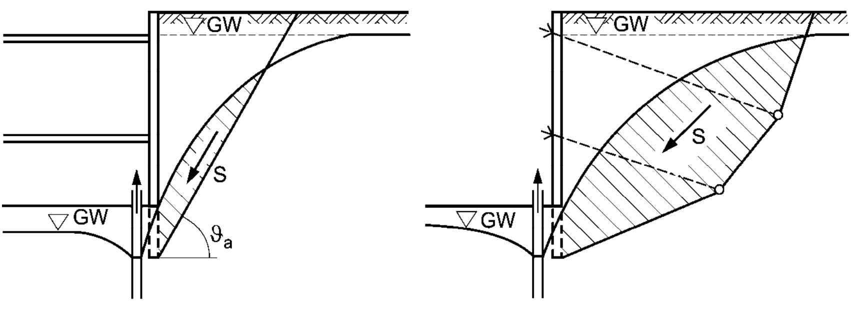


Figure R 60-1. Flow forces resulting from groundwater drawdown additional measures. The following effects result from the remaining, upper groundwater, as shown in Figure R 60-2 a):

a) The water pressures is maintained in the upper aquifer level.
b) A gradient $i_a = (h + d)/d$ develops in the slightly impermeable layer. This increases the mathematical unit weight of the soil from $\gamma_a = \gamma' + \gamma_w$ to $\gamma_a = \gamma' + i_a \cdot \gamma_w$.

The remaining water pressure is shown in Figure R 60-2 c).

4. When determining the passive earth pressure it shall generally be assumed that the water level inside the excavation can be at the excavation level and that the soil is therefore fully buoyant. The effects of groundwater drawdown and thus the adopted unit weight of the naturally moist soil may only be taken into consideration if measures are taken against possible pump failure as specified in R 66, Paragraph 1 (Section 10.9), and then only if the anticipated drawdown curve justifies this. If a cohesive layer below the excavation level is subjected to pressure from below from confined groundwater despite water management measures, the unit weight reduction due to flow force shall be taken into consideration and the safety against base heave ensured according to R 61 (Section 10.4) or R 62 (Section 10.5).
5. If the drawdown curve intersects the soil region governing stability, as shown in Figure R 60-1 b), the effect of the flow force shall be taken into consideration for both the deep-seated stability analysis according to R 44 (Section 7.3) and for the general stability analysis according to R 45 (Section 7.4).
6. The effective unit weight of a saturated, cohesive soil is increased from γ' to γ_r by lowering the groundwater table or by groundwater relief. This has a

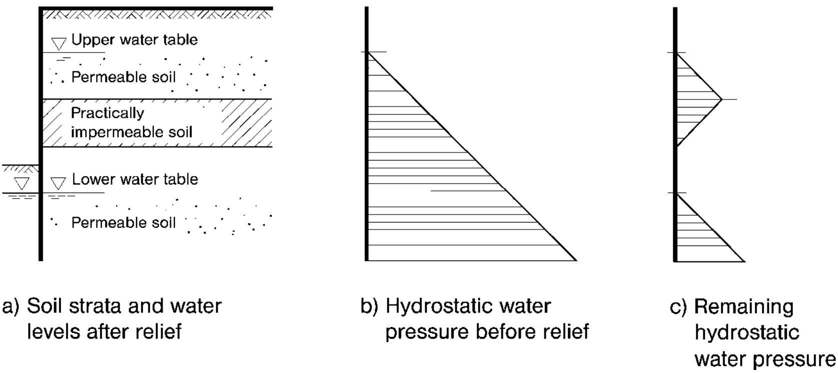


Figure R 60-2. Water pressures before and after relief below a practically impermeable soil stratum similar effect as applying a load at ground level and may cause considerable settlement of soft, cohesive soils, which may also be detrimental to buildings in the region of the drawdown. If necessary, dewatering measures shall be dispensed with and different construction methods adopted.

10.4 Analysis of hydraulic heave safety (R 61)

1. In permeable soils, the base of the excavation may fail by hydraulic heave if only sump pumping is utilised inside the excavation and no further measures are taken (see Paragraph 10). Hydraulic heave failure occurs when upward directed flow forces are equal to the sum of the dead-weight of the buoyant soil and any additional restraining forces. Also see Paragraph 5.

2. The flow forces acting in the region of the investigated failure mass shall be determined according to R 59, Paragraph 2 (Section 10.2). An additional increase in the upward directed flow force occurs in the corners of excavations and in narrow excavations [56, 57, 59]. If it is necessary for the hydraulic heave safety to be equal at all points of a rectangular excavation, the retaining walls shall be embedded deeper at the corners, or other measures provided for. See also [117]. The necessary embedment depth increases with decreasing excavation width. The occurrence of a three-dimensional flow force effect, leading to an increase in the necessary embedment depth of the wall, primarily depends on the excavation geometry and the thickness of the water-bearing stratum. In case of doubt, separate investigations shall be performed.

3. If necessary, the possibility of seepage path shortening, e.g. by fissure formation in accordance with R 58, Paragraph 3 (Section 10.1), shall be taken

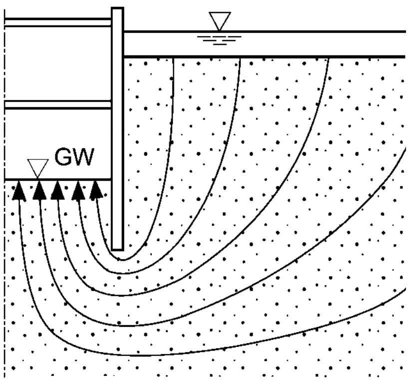


Figure R 61-1. Restriction of flow cross-section in the region of an upward directed flow in narrow excavations into consideration. The governing depth for analysis of hydraulic heave safety for staggered wall toes is always the lesser embedment depth.

4. No ground resistances are taken into consideration in a hydraulic heave analysis, only actions: The flow force as an unfavourable permanent action and the dead-weight of the soil as a favourable permanent action. Hydraulic heave failure is therefore classified as a failure resulting from the loss of equilibrium and thus assigned to the HYD limit state. In order to achieve sufficient safety against hydraulic heave failure it shall be demonstrated that the condition:

   $$S_k \cdot \gamma_H \leq G'_{stb,k} \cdot \gamma_{G,stb}$$

   is met for homogeneous ground (Eurocode 7 Handbook, Volume 1, Section 10.3).

   Where:

   S_k the characteristic flow force within the percolated soil mass;
   γ_H the partial safety factor for the flow force in favourable or unfavourable ground in the HYD limit state taken from Table 6.1, Appendix A 6;
   $G'_{stb,k}$ the characteristic dead-weight of the buoyant, percolated soil mass;
   $\gamma_{G,stb}$ the partial safety factor for favourable permanent actions in the HYD limit state taken from Table 6.1, Appendix A 6.

   A rectangular soil mass as shown in Figure R 61-2, with a width equal to half of the embedment depth [60], is generally adopted as the percolated soil mass. This approach applies to a homogeneous soil. More precise investigations are required for stratified ground. The simpler and more conservative stability analysis is performed for a flow line along the wall [61]. Friction forces between the failure body and the retaining wall may only be taken into consideration following special investigations. Analysis of hydraulic heave safety requires expertise and experience in the geotechnical field.

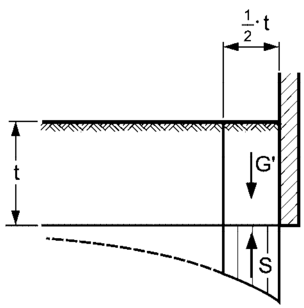


Figure R 61-2. Analysis of hydraulic heave safety after *Terzaghi* and *Peck*

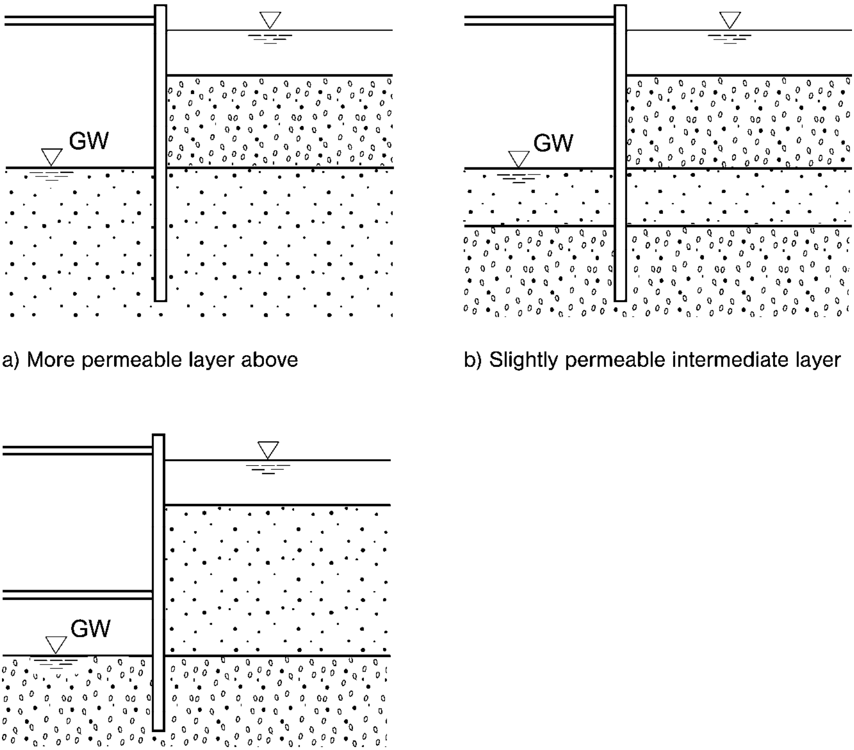
5. Past experience has shown that a shallower embedment depth than for cohesionless soils is sufficient to avoid hydraulic heave failure as a result of percolation around the wall toe in cohesive soils. Mathematically, this can only be demonstrated if the cohesion on the free sides and the tensile strength of the ground on the underside of the assumed failure body are adopted. Competence and experience in the geotechnical field are required for this. The justified objection that tensile strength may be lost locally due to cohesive or cohesionless layers may, if applicable, be countered by analysis of buoyancy safety according to R 62 (Section 10.5). A water-bearing layer is assumed at the level of the base of the retaining wall. If buoyancy safety cannot be demonstrated, relief wells or spill wells according to Paragraph 10 a) shall be installed in order to decrease the flow forces on the inside of the excavation.

6. If the ground is subjected to variable permeability, the potential dissipation is concentrated in the less permeable layers. In principle, two cases shall be differentiated here:

   a) With regard to the safety against hydraulic heave failure, a less permeable layer below the excavation base as shown in Figure R 61-3 a) acts unfavourably. In this case, only the seepage path through the less permeable layer may be adopted in the analysis.
   b) It is particularly unfavourable if this less permeable, possibly cohesive, layer as shown in Figure R 61-3 b) is underlain by a permeable layer, which in turn is connected hydraulically to the upper, permeable layer.
   c) If the less permeable layer is located above the permeable layer as shown in Figure R 61-3 c), the associated favourable effect may only be considered under certain conditions, because the lateral inflow may critically and unfavourably influence safety against hydraulic heave failure. The filter stability of the permeable layer shall also be analysed [79]. Otherwise, it is recommended to monitor the changes in porewater pressures according to the observational method described in the Eurocode 7 Handbook, Volume 1.

7. Excavations in groundwater exhibit less vulnerability to hydraulic heave than excavations in open water, if a drawdown curve develops and the positive water pressure therefore decreases in the region of the excavation. However, the short-term drawdown curve produced during the respective excavation phase governs the hydraulic heave safety analysis. Generally, for slightly permeable soils, in particular for silt and fine sand, the potential of the non-lowered groundwater table is adopted as the basis for analysis.

8. The partial safety factors $\gamma_{G,dst}$ and $\gamma_{G,stb}$ required for analysis of hydraulic heave safety in the HYD limit state can be taken from Table 6.1 of Appen-



a) More permeable layer above

b) Slightly permeable intermediate layer

c) More permeable layer below

Figure R 61-3. Influence of ground stratification

dix A 6. The following points apply with regard to the partial safety factor γ_H for the flow force in a favourable or unfavourable subsurface:

a) Gravel, gravel-sand and at least medium-dense sand with grain sizes greater than 0.2 mm are deemed as favourable soils, as well as at least stiff, clayey, cohesive soil.
b) Loose sand, fine-sand, silt and soft, cohesive soil are deemed unfavourable.
c) In unfavourable ground the partial safety factors given for favourable ground may be adopted if an at least 0.3 m thick mechanically filter-stable and hydraulically effective ground layer is present. The filter stability [161] of the soil stratum shall be demonstrated.

In unfavourable ground the piping failure hazard shall be investigated (see Eurocode 7 Handbook, Volume 1, Paras. 10.4 and 10.5). Countermeasures shall be provided for where necessary. See the EAU, Recommendation R 116 [2].

9. Because of the danger of errors of judgement and the associated large hazard potential, a mathematical analysis of the safety against hydraulic heave shall always be performed. Simplified approaches, e.g. those given in R 61 in earlier editions of EAB, may not be adopted.

10. If investigations do not demonstrate sufficient hydraulic heave safety, the following measures may be taken in addition to enlarging the embedment depth:

    a) installation of spill wells (relief wells) within the excavation, see Section 10.8, Paragraph 6;
    b) installation of gravity or vacuum wells within the excavation;
    c) partial or complete dewatering or groundwater relief;
    d) installation of a surcharge filter;

    or a different excavation system shall be used, e.g. watertight excavation or compressed air methods.

10.5 Analysis of buoyancy safety (R 62)

1. If the retaining walls form a closed body with a highly impermeable layer at the excavation level or lower, sufficient buoyancy safety shall be demonstrated. This is principally the case in the following circumstances:

    a) The retaining walls are so deep that they embed in a practically impermeable soil layer at the excavation level (Figure R 62-1 a), underlain by a permeable layer. In this case relief wells according to R 65, Paragraphs 6 and 7 (Section 10.8) are required within the excavation, if heterogeneities in the practically impermeable layer cannot be ruled out.
    b) A sufficiently thick, practically impermeable layer is present at great depth below the excavation level (Figure R 62-1 b), underlain by a permeable soil layer.
    c) A practically impermeable, sufficiently thick sealing layer is installed sufficiently deep below the excavation level, e.g. by grouting, by jet grouting or by freezing (Figure R 62-1 c).
    d) The excavation is sealed by an anchored underwater concrete base (Figure R 62-1 d).
    e) An moderately deep, anchored, grouted or jet-grouted base, covered with soil, is provided below the excavation level (Figure R 62-1 e).

    A soil layer is regarded as practically impermeable if it has a permeability at least two orders of magnitude less than the permeability of the surrounding ground.

2. Sufficient buoyancy safety shall be given at all times. If the sealing base is not anchored using tension piles (Figure R 62-1 a), b) and c), it shall be demonstrated that the

   $$V_{dst,k} \cdot \gamma_{G,dst} \leq (G_{B,k} + G_{W,k} + T_k + P_{v,k}) \cdot \gamma_{G,stb}$$

   condition is met in the UPL limit state.

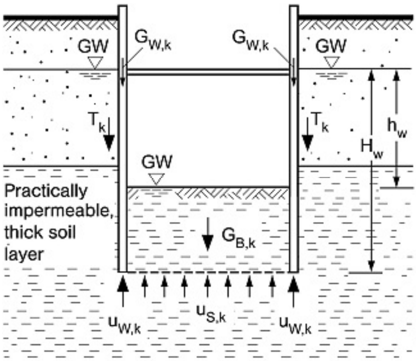
   Where:

   $V_{dst,k}$ the vertical component of the characteristic hydrostatic water pressure on the base ($u_{S,k}$) and the wall ($u_{W,k}$) acting on the underside of the practically impermeable soil layer or sealing layer;
   $\gamma_{G,dst}$ the partial safety factor for unfavourable permanent actions in the UPL limit state taken from Table 6.1, Appendix A 6;
   $G_{B,k}$ the lower characteristic value of the self-weight of the overlying soil, including the sealing layer as shown in Figure R 62-1;
   $G_{W,k}$ the characteristic value of the self-weight of the retaining wall, including bracing;
   $\gamma_{G,stb}$ the partial safety factor for favourable permanent actions in the UPL limit state taken from Table 6.1, Appendix A 6;
   T_k the characteristic value of the vertical component of the earth pressure acting on the retaining walls as a permanent, downward directed action as shown in Figure R 62-1.
   $P_{v,k}$ the characteristic value of the vertical component of the anchor force supporting the retaining walls, as a permanent, downward directed action.

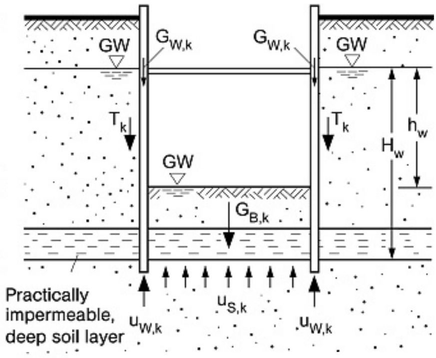
   The forces T_k and $P_{v,k}$ are treated as downward directed actions and not as resistances, because they do not occur as a result of the upward directed water pressure. See Paragraphs 7 to 10 for restrictions when adopting downward directed actions.

3. When adopting the actions from the self-weight of the retaining wall $G_{W,k}$, the vertical earth pressure T_k and any vertical anchor force component $P_{v,k}$, transfer of the forces in the wall/sealing base interface shall be demonstrated.

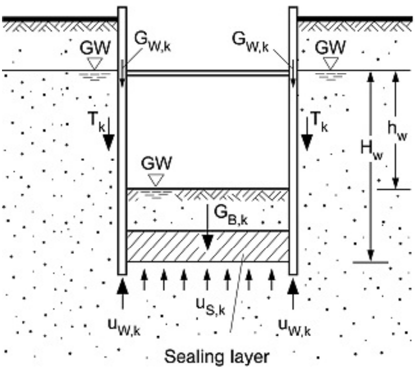
4. In order to demonstrate that the conditions in Section 2 are met, the forces $G_{W,k}$, T_k and $P_{v,k}$ acting on the overall watertight excavation system may only be adopted if it can be demonstrated that the buoyant force $V_{dst,k}$ resulting from $u_{S,k}$ can be transferred to the retaining walls. In general, the forces may only be taken into consideration in narrow excavations or in the boundary zone as far as the first row of tension piles as shown in Figures R 62-1 c) and d).



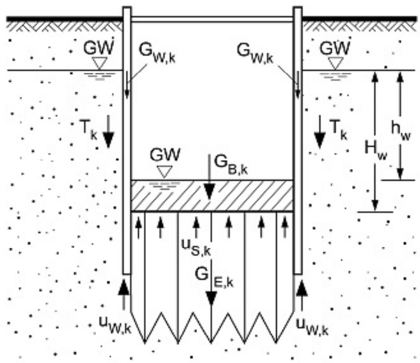
a) Sealing layer with practically impermeable, thick soil layer



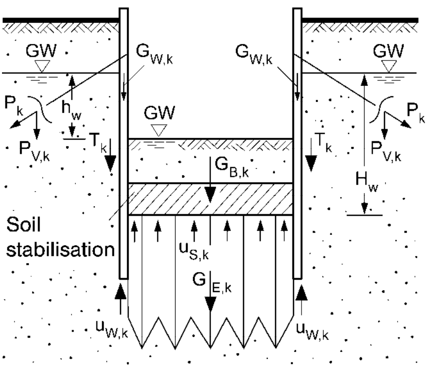
b) Sealing layer with practically impermeable, deep soil layer



c) Artifical, deep sealing layer



d) Sealing layer with an anchored underwater concrete base



e) Sealing layer with anchored, moderately deep soil stabilisation

Figure R 62-1. Forces adopted for analysis of buoyancy safety

5. If a resistance from a base anchored by tension piles acts when analysing buoyancy safety, two limit cases shall always be investigated: the bearing capacity of the individual tension piles according to Paragraph a) on the one hand, and the bearing capacity of the tension piles taking pile group effects according to Paragraph b) into consideration on the other.

   a) Assuming that the bearing capacity of the individual tension elements is decisive, sufficient safety against pull-out shall be demonstrated for the GEO 2 limit state. The tensile stress design value $F_{t,d}$ required for this analysis is determined from:

      $$F_{t,d} = V_{dst,k} \cdot \gamma_G - (G_{B,k} + G_{W,k} + T_k + P_{v,k}) \cdot \gamma_{G,inf}$$

      for the system as a whole, and from

      $$F_{t,i,d} = V_{dst,i,k} \cdot \gamma_G - G_{B,i,k} \cdot \gamma_{G,inf}$$

      for the individual pile in a pile group.

      Where:

      $F_{t,d}$ the design value of the load on the tension pile group as shown in Figures R 62-1 d) and e);
      $F_{t,i,d}$ the design value of the stress on a tension pile;
      $V_{dst,i,k}$ the characteristic hydrostatic water pressure $u_{S,k}$ on the base for the grid area $l_a \cdot l_b$ (see Figure R 62-2);
      $G_{B,i,k}$ the characteristic value of the self-weight of the overlying soil, including the sealing layer for the grid area $l_a \cdot l_b$;
      γ_G the partial safety factor for permanent loads as given in Table 6.1 of Appendix A6,
      $\gamma_{G,inf}$ the partial safety factor $\gamma_{G,inf} = 1.0$ for favourable permanent surcharges.

      Sufficient safety against failure is given if, for pile groups, the condition:

      $$F_{t,d} \leq R_{t,d}$$

      is met, and for an individual tension pile

      $$F_{t,i,d} \leq R_{t,i,d}.$$

      Where:

      $R_{t,d}$ the design value of the tension pile resistance of a pile group according to R 86 (Section 13.11);
      $R_{t,i,d}$ the design value of the individual tension pile resistance according to R 86 (Section 13.11).

      See Paragraphs 2 to 10 for restrictions when adopting the downward directed action.

b) Sufficient buoyancy safety in the UPL limit state shall be demonstrated for the tension pile. This is demonstrated if the following condition is met for the pile group:

$$V_{dst,k} \cdot \gamma_{G,dst} \leq (G_{B,k} + G_{W,k} + T_k + P_{v,k} + G_{E,k})\, \gamma_{G,stb}$$

and

$$V_{dst,i,k} \cdot \gamma_{G,dst} \leq (G_{B,i,k} + G_{E,i,k}) \cdot \gamma_{G,stb}$$

for the individual tension pile.

Where, in addition to the variables described above:

$G_{E,k}$ the characteristic weight of the buoyant ground attached to a tension pile group;

$G_{E,i,k}$ the characteristic weight of the buoyant ground attached to a tension pile.

See Paragraphs 2 to 10 for restrictions when adopting the downward directed action.

6. For determination of the characteristic buoyant forces $V_{dst,k}$ or $V_{dst,i,k}$, the full hydrostatic pressure $u_{S,k} = \gamma_w \cdot h_w$ obtained from the design water level shall be adopted on the base. The governing base surface area is the underside of the practically impermeable soil layer. For the purpose of calculations, the underside shall be adopted high enough that all possible irregularities are taken into consideration conservatively. If the weight $G_{W,k}$ is adopted, the water pressure $u_{W,k}$ acting on the underside of the retaining walls shall also be taken into consideration when determining the characteristic value of the buoyant force $V_{dst,k}$. Any difference in the wall depths shall be taken into consideration.

7. The following points apply for determination of the characteristic value of the dead-weight $G_{B,k}$ and $G_{W,k}$ as shown in Figure R 62-1:

   a) The unit weight of concrete may be adopted at a maximum of 23 kN/m^3 and that of reinforced concrete at a maximum of 24 kN/m^3.
   b) The characteristic value of the dead-weight of the soil $G_{B,k}$ within the excavation shall be determined using the wet unit weight of the soil above the water table and the saturated unit weight below it. The water level within the excavation shall be adopted conservatively.
   c) The characteristic value of the dead-weight of the retaining wall $G_{W,k}$ is determined as follows:

   – from the weight of the steel in the wall for a sheet pile wall;
   – from the footprint area and wall height for a diaphragm wall or bored pile wall;

– from the weight of the struts and the waling for bracing, if they act in the respective construction stage.

d) The characteristic value of the unit weight of grouted and jet-grouted bodies is adopted at the same value as the unit weight of the ground, if the unit weight is not determined separately.

8. The force T_k is obtained from:

$$T_k = \eta_z \cdot E_{ah,k} \cdot \tan \delta_{a,k}$$

using the calibration factors:

$\eta_z = 0.8$ for DS-T;

$\eta_z = 0.9$ for DS-A.

The active earth pressure $E_{ah,k}$ on the retaining wall may only be adopted as the lower characteristic value (see the Eurocode 7 Handbook, Volume 1, Section 10.2).

9. Only the lock-off force P_f may be adopted when determining the vertical component $P_{v,k}$ of the tensile force of prestressed anchors supporting the retaining wall.

10. The characteristic self-weight $G_{E,i,k}$ of the soil held by base anchors may be determined using the geometric relationships as shown in Figure R 62-2 using:

$$G_{E,i,k} = \eta_z \cdot \gamma' \cdot l_a \cdot l_b. \cdot \left(L - 1/3 \cdot \cot\varphi \cdot \sqrt{\left(l^2_a + l^2_b\right)} \right).$$

Where:

$G_{E,i,k}$ the characteristic weight of the ground attached to a tension element;
L the length of the tension element below the lower surface of the base;
l_a the larger grid dimension of the tension elements;
l_b the smaller grid dimension of the tension elements;
γ' the lower characteristic value of the unit weight of the buoyant soil;
η_z in accordance with Section 8.

Additional approaches addressing base anchoring and the attached ground can be found in [162].

If a tension pile group is analysed, the number of tension piles shall be taken into consideration.

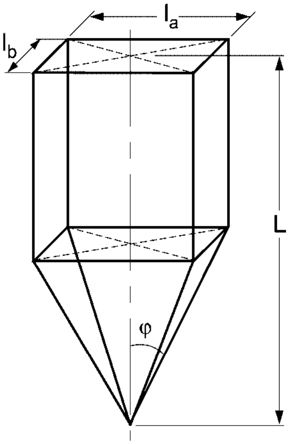


Figure R 62-2. Geometry of the ground attached to a single tension pile

11. With regard to base anchoring using tension piles, empirical data is available for micropiles and vibro-injection piles. The maximum pile spacing shall not exceed 3.5 m.

12. When designing the anchored, sealing base, failure of a tension pile in the STR limit state and design situation DS-A, adopting the partial safety factors given in Table 6 for the adjacent piles shall be analysed.

13. In addition to the buoyancy safety analysis, analysis of hydraulic heave safety according to R 61 (Section 10.4) shall also be performed, if:

    a) the retaining walls are only shallowly embedded in the practically impermeable layer (Figure R 62-1 a);
    b) the retaining walls embed in a layer with a permeability less than two orders of magnitude smaller than the overlying soil.

    Further analysis of the safety against hydraulic heave resulting from vertical percolation through the practically impermeable layer, as discussed in [147], is not necessary.

14. If the practically impermeable base consists of fine-grained soil and the layer above of coarse-grained soil, the filter stability shall be analysed [79].

15. Heave clearly exceeding that already anticipated in dry excavations according to R 83, Paragraph 13 (Section 4.11) may be associated with the installation of a base anchored by tension piles. See [137], [141] and R 83, Paragraph 11 (Section 4.11).

    In relatively stiff bases this heave leads to imposed loads; it may be necessary to take these loads into consideration in design [163].

10.6 Stability analysis of retaining walls in water (R 63)

1. If percolation around the wall toe is prevented, the full hydrostatic water pressure from the open water surface or the groundwater level to the wall toe on the outside, or the hydrostatic water pressure from the lowered groundwater level to the wall toe on the inside, shall be adopted (Figure R 63-l b) as the characteristic load on the retaining walls.

   The differential water pressure between the water pressure on the outside and that on the inside of the retaining wall is treated as the only characteristic action according to the Eurocode 7 Handbook, Volume 1, Paragraph 9.6 A(8).

2. If water percolates around the wall toe, this shall always be taken into consideration. The following approaches may be adopted:

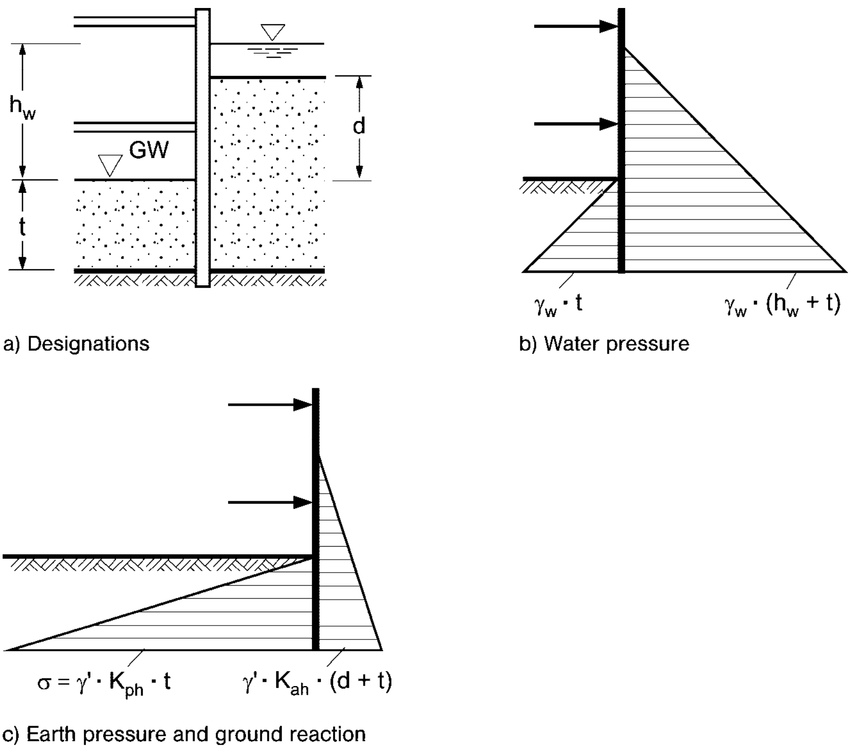


Figure R 63-1. Earth pressure, water pressure and ground reaction for a non-percolated retaining wall in water (simplified representation)

a) The water pressure on the outside of the retaining wall decreases:

$\Delta w = i_a \cdot z_a$.

The water pressure on the inside increases (Figure R 63-2 b):

$\Delta w = i_p \cdot z_p$.

b) The earth pressure on the outside of the retaining wall increases as a result of the increase in unit weight due to the flow force (Figure R 63-2 c):

$\Delta\gamma'_a = i_a \cdot \gamma_w$.

c) The passive earth pressure on the inside decreases considerably due to the decrease in unit weight:

$\Delta\gamma'_p = -i_p \cdot \gamma_w$.

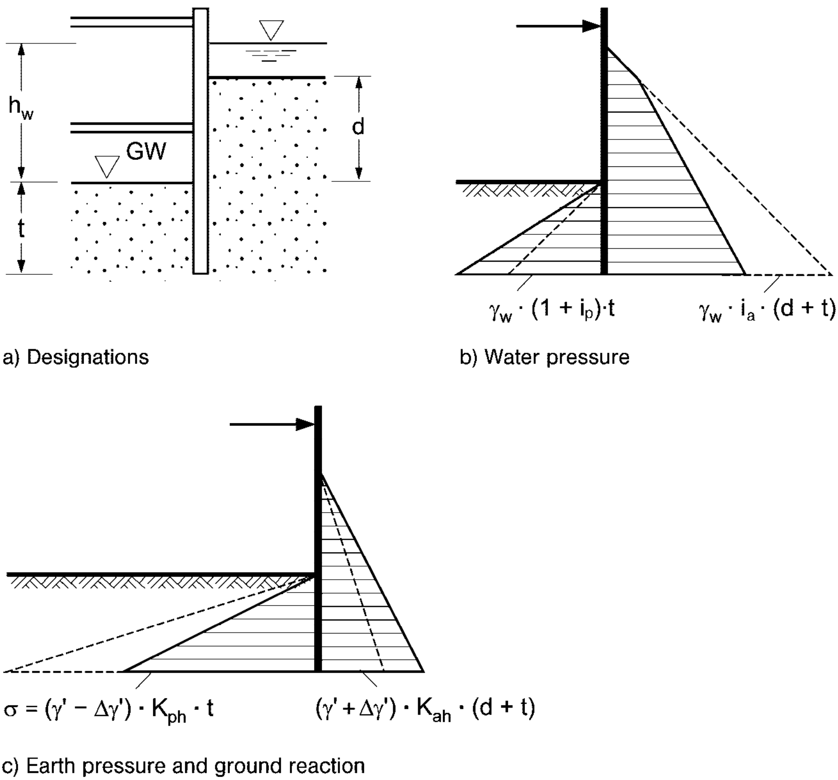


Figure R 63-2. Earth pressure, water pressure and ground reaction for a percolated retaining wall in water (simplified representation)

See R 59 (Section 10.2) for determination of the flow forces. Figure R 63-2 shows a simplified representation of the linear dissipation in potential difference with i_a on the active earth pressure side and i_p on the passive earth pressure side. The hydraulic gradients i_a and i_p may be calculated linearly from the determined potential differential on each side of the wall starting at the toe.

3. Earth pressure redistribution as stipulated in R 5 (Section 3.3) shall also be anticipated if the soil is completely or partially buoyant. However, this does not include the increase in earth pressure resulting from flow. It may be included in the redistribution diagram as a sufficient approximation.
4. For excavations in open water, surcharge loads according to R 24, Paragraph 4 (Section 2.1), or abnormal loads according to R 24, Paragraph 5, shall also be adopted, in addition to water pressure and earth pressure. In particular, these include:

   a) wave action, see the EAU, Recommendation R 135 [2];
   b) berthing forces of ships, see EAU, Recommendations R 38 and R 12 [2];
   c) ice floe impact forces, see printed matter Ril 804 published by the *Deutsche Bahn AG* and [62];
   d) sheet ice pressure, see [62] and Ril 804.

   Further information on adopting ice loads is given in the EAU, Recommendation R 177 [2].
5. In principle, the same rules apply for adopting the ground reaction, for determining the action effects and for designing the individual components as for dry excavations. If the ground reactions down to a deeper, practically impermeable layer are utilised to support the retaining wall at the wall toe, the passive earth pressure angle required for stability analysis may be adopted at a maximum of $\delta_{p,k} = -20°$. The downward directed failure plane associated with the usually adopted wall friction angle $\delta_p = -\varphi$ only affects a soil layer in, or below, the practically impermeable layer with a minor vertical stress. Also see [96].

   A base constructed using jet grouting methods forms a wall support, so that the ground reactions above the sealing layer are only utilised corresponding to the wall deformations that actually occur.
6. The following procedures may be used to determine the action effects:

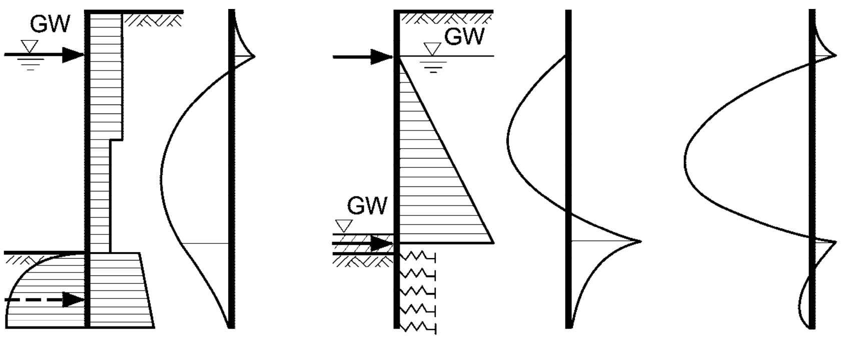
   a) According to R 24 (Section 2.1), in conjunction with R 79 (Section 2.4), the agreed design water level is assigned to design situation DS-T and the water level that will flood the excavation if adopted or at which the excavation shall be flooded, design situation DS-T/A.
   b) If only the stability analysis for the STR and GEO 2 limit states is pertinent according to R 11, Paragraph 2 (Section 4.2), analysis may be

performed using the embedment depth for the advancing states according to R 80, Paragraph 9 (Section 4.3), as long as the equilibrium conditions are met.

c) Because water pressure generally produces unfavourable actions and may be dealt with as a permanent action, it may be incorporated in a combined pressure diagram with the buoyancy-reduced earth pressure according to R 104, Paragraph 4 (Section 4.11). However, when determining the vertical forces it shall be noted that only the earth pressure component with wall friction occurs. The combined pressure diagram is not expedient if the action effects are determined using classical earth pressure distribution and if earth pressure redistribution is replaced by surcharges to the determined support forces.

Replacement of the anticipated ground reactions by a fixed support when defining the structural system according to R 11, Paragraph 3 (Section 4.2) is generally not permissible.

7. If, at the same time, major changes in actions and in the structural system occur from one construction stage to the next, the action effects of the new construction stage shall be determined by superimposing the action effects of the previous construction stage on the changes in action effects produced by these major changes. This contrasts with R 11, Paragraph 2 b) (Section 4.2), where each construction stage may be analysed separately. This occurs if the excavation is drained after installation of an underwater concrete base [97]. Also see Figure R 63-3.



a) Bending moment form earth pressure

b) Bending moment form water pressure

c) Superimposed bending moments

Figure R 63-3. Determination of bending moments for a simultaneous change in load and structural system (example)

8. Water pressure generally dominates over earth pressure for excavations in open water or in groundwater. Because water pressure does not provide a vertical component action, in contrast to earth pressure, a principal downward action is lacking for analysis of the vertical component of the mobilised passive earth pressure according to R 9 (Section 4.7), at least for unsupported and for braced retaining walls. This leads to considerably greater embedment depths than for dry excavations or excavations in lowered groundwater. In addition, the usual embedment depth surcharge of $\Delta t_1 = 0.20 \cdot t_1$ according to R 26, Paragraph 5 (Section 6.4) is not sufficient for walls with a fixed-earth support. It shall be increased to $\Delta t_1 = 0.40 \cdot t_1$ or determined according to R 26, Paragraph 8.

9. In order to prevent wall displacements if the external water level subsequently increases, anchors may generally be prestressed to a minimum of 80 % of the service load and to a maximum of 100 % of the service load. The movement at the top of the wall caused by prestressing the anchors initially generates an increased active earth pressure at the rear wall face, which is reduced completely or in part to the smaller active earth pressure, where applicable redistributed upwards, by the subsequently increasing water pressure.

10. See Recommendations R 100 and R 101 [2] of the Recommendations of the Committee on Waterfront Structures (*EAU*) for analysis of the stability of cellular and box cofferdams; see Recommendation R 69 for elastic dolphin piles.

10.7 Design and construction of excavations in water (R 64)

1. Only sufficiently impermeable retaining walls may be employed for excavations in open water or in groundwater, e.g. sheet pile walls, diaphragm walls and secant pile walls. If the normal sealing properties of sheet pile interlocks are not sufficient, sheet pile sections with factory-fitted interlock seals may be employed. If it is anticipated that the sheet piles will run out of the interlocks to a large extent because of obstructions in the ground, it is expedient to carry out soil replacement in the driving region before driving the piles. If soil replacement extends down to the embedment depth of the wall, it shall be incorporated in the analysis, in particular when analysing the safety against hydraulic heave.

2. If individual sheet piles, diaphragm wall slices or piles cannot be installed to the projected design depth, additional measures shall be provided for or additional analyses performed to guarantee safety against hydraulic heave failure.

3. With regard to analysis of buoyancy safety according to R 62 (Section 10.5) of an excavation with walls embedded in a practically impermeable layer, the walls shall form a watertight unit with this layer. Sufficient wall embedment in the sealing layer depends on the ground and construction method and shall also be controlled by on-site inspections.

4. A practically impermeable base seal compliant with R 62, Paragraphs 1 c) and 1 e) (Section 10.5) shall generally be at least 0.8 m thick. The piping safety of the grouting or jet-grouting medium shall be demonstrated.

   It shall be demonstrated locally by means of suitability tests that the planned diameter of the jet grouted columns or sufficient grouting propagation can be achieved. The grid shall be configured such that the individual jet grouting columns or grouted sections safely overlap, including the anticipated diameters and tolerances.

   Deviations shall be taken into consideration by adapting the borehole grid.

5. The orientation of the jet axis and the diameter are especially relevant for the sealing properties of the base when constructing sealing bases using jet grouting methods. The orientation and verticality of the jetting or drilling axis shall therefore generally be monitored.

6. Defects in deep, non-anchored sealing bases generally only lead to greater quantities of residual water. In anchored, moderately deep or shallow jet grouted bases, larger defects can lead to hydraulic heave failure. Shallow jet grouted bases shall be provided with a second seal to protect against piping (redundant system). Moderately deep jet grouted bases shall be provided with sufficient cover as a function of the overburden thickness [144]. Cohesive soils are unsuitable as cover materials.

7. In excavations provided with an underwater concrete base, the water level within the retaining walls may be initially lowered as far as hydraulic heave safety considerations and the strength of the excavation allow. During installation of the concrete and the time until hardened the water level within the excavation may not be lower than outside. The DBV "*Unterwasserbeton*" (Underwater Concrete) Code of Practice [143] applies for installation of the underwater concrete, in conjunction with the regulations in DIN 1045-2 and EN 206-1, and EN 13670 in conjunction with DIN 1045-3, as well as [138].

8. Before excavating a watertight excavation or draining after installing an underwater concrete base, pump tests shall be performed to ensure correct sealing. During the pump test the retaining walls are subjected to water pressure and the resulting deformations monitored, in particular in excavations adjacent to structures.

The pump test is divided into 3 phases:

– drawdown;
– equilibrium;
– recovery.

The recovery phase may be dispensed with if the seal is sufficient.

9. The drawdown during the pump test shall be at least $0.5 \cdot h_w$ for water table differences $h_w \geq 6$ m as shown in Figure R 63-1.

10. Where soil is flushed through defects in the underwater concrete base and the walls, the defects shall be immediately sealed during the pump test's drawdown phase. Where different base seals are employed, defects in the walls shall be immediately sealed during excavation.

    Localisation of minor defects in moderately deep or deep sealing bases can be extremely difficult [146].

    Defects in the walls and the base can be localised more easily by using sealing bulkheads.

11. See [138, 141, 145] for details of executing the anchoring elements and of the anticipated deformations. If the vertical component of the earth pressure, the vertical component of the anchors and the self-weight of the retaining wall are taken into consideration for analysis of buoyancy safety, complete force transfer between the base and wall shall be guaranteed, e.g. by grooves in in-situ concrete walls or welded steel pieces on sheet pile walls.

12. As a safeguard against the sudden ingress of water through defects in the retaining wall and against the possibility of fissure development behind the retaining wall as discussed in R 58, Paragraph 3 (Section 10.1), configuration of a cofferdam as shown in Figure R 64-1 has proven useful. As a minimum measure, securing the bed with sandbags along the length of the retaining wall shall be planned. These measures are also suitable as a safeguard against piping failure according to R 58, Paragraph 5 (Section 10.1).

13. If initial piping phenomena (springs) are observed during excavation, securing measures shall be implemented immediately, e.g.

    – placement of coarse-grained soils;
    – partial flooding of the excavation;
    – if possible immediately, water pressure relief measures.

    Remediation measures can then be implemented.

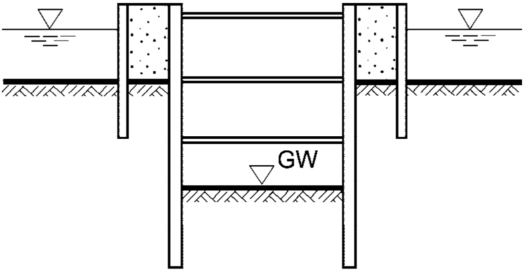


Figure R 64-1. Securing an excavation using cofferdams

14. In cases where a recognised hazardous condition cannot be otherwise eliminated, measures for purposely flooding the excavation shall be taken. When flooding, it shall be ensured that the inflowing water cannot cause any damage. In addition, elongated excavations with a large surface area shall be divided into sections by bulkheads in order to restrict sudden water ingress to limited sections of the excavation.
15. Occasionally it may be expedient for economical reasons to flood an excavation instead of designing for exceptionally high, rare water levels.
16. If surcharges and abnormal load conditions according to R 3, Paragraph 4 (Section 10.6) need to be avoided, the following measures may prove useful for excavations in open water:
    a) Configuration of dolphin piles to take up ship berthing shocks or placing a sandbank to keep ships at a distance.
    b) Continuous icebreaking along the retaining wall.
    c) Configuration of dolphin piles and floating beams to deflect ice floes and similar objects.
    d) Protecting the bed against scour formation according to the EAU, Recommendation R 83 [2].

10.8 Water management (R 65)

1. The following principal methods of water management may be considered:
    a) watertight excavation with residual water management;
    b) groundwater lowering using sump pumping;
    b) groundwater lowering using wells;
    d) groundwater relief.
2. It is expedient to construct a watertight excavation if, when lowering the groundwater, the amount of water extracted needs to be limited, settlement damage may occur as a result of groundwater lowering or groundwater

lowering using sump pumping or wells is impermissible as a result of groundwater contamination.

3. Sump pumping involves the water entering through the sides and bottom of the excavation being collected in drains, sent to pump sumps and pumped away. Sump pumping is suitable for small drawdown depths. Special measures are necessary in soils with a tendency to liquefy, e.g. soil replacement methods, whereby only small areas are laid free for short periods and are immediately covered by filter material.

4. Where water management using wells is employed, the water is collected in wells, which may be arranged inside or outside the excavation, and pumped away. In principle, two types are differentiated:

   a) Gravity wells are used if the water flows into the wells as a result of gravity, e.g. in sand and gravel.
   b) Use of vacuum-assisted dewatering is necessary if the gravity is not sufficient to allow the water to flow into the filter well, e.g. in fine-sand and coarse silt.

   Also see [1] and [64]. The lowered groundwater table shall generally be at least 0.5 m below the excavation level within the excavation.

5. Both sump pumping and wells are possible in watertight excavations. Soft gel injections can block pore spaces and reduce the inflow to wells and drains. Reserve wells and drains, for example, shall be held ready.

6. Groundwater relief may be necessary:

   a) If a slightly permeable layer below the excavation level is not capable of bearing the net resulting water pressure acting from below, see R 62 (Section 10.5).
   b) If the safety against hydraulic heave according to R 61 (Section 10.4) cannot be ensured in any other way.
   c) If cohesive soils are intercalated with permeable, cohesionless soils.

   In this case it may also be sufficient to arrange overflow wells with adequately small spacing within the excavation, where the groundwater can rise as far as the excavation level and then be collected and pumped away.

7. The following points shall be observed when employing overflow wells according to Paragraph 6:

   a) Similar to dewatering, the yield shall be computed for the prevalent hydrogeological situation, and the capacity and number of overflow wells adapted accordingly.
   b) Generally, overflow wells shall be fitted with screens, similar to dewatering wells. If, for minor yields, they are executed as gravel piles, filter stability shall be demonstrated.

c) Dissipation of the positive water pressure shall be monitored using porewater pressure transducers.
d) Overflow wells shall generally be sealed with appropriate material after abandonment.
e) The wells and relief wells shall be dimensioned sufficiently to reduce the risk of damage.

8. See [65] and [66] for groundwater reclamation by means of injection wells.

10.9 Monitoring excavations in water (R 66)

1. The following facilities shall be provided if the stability of the excavation is endangered or heavy economical losses are anticipated if water management facilities fail at short notice:

   a) Two independent power sources, e.g. from the public utility network and from emergency generators.
   b) Automatic switching facility for the pump power supply.
   c) If one pump fails, automatic switching to a non-operating well.
   d) Optical or acoustic signals.
   e) Display equipment for evaluation of pump performance.

   Facilities b) to e) are generally integrated into one switching and control centre. This control centre shall be monitored at all times, be equipped with a reliable warning system and have a sufficient supply of spare parts available.

   If short-term faults or interruptions do not pose a hazard, less complex facilities for power supply, switching and monitoring may suffice.

2. All influences relevant to an assessment of the water management facilities shall be regularly monitored and recorded, e.g.:

   – the water level of open water bodies;
   – the drawdown achieved within the excavation and in the immediate vicinity;
   – the amount of water pumped.

   Where there is a danger of violating water rights, the range of the drawdown shall also be monitored. The same applies if there is a danger of settlement. In this case, settlement measurements on buildings and on datum points shall also be provided for.

3. During excavation, it may prove useful to continuously measure the water level in the ground below the excavation level, or the porewater pressure in slightly permeable soil, in order to allow timely recognition of any irregularities.

11 Excavations in unstable rock mass

11.1 General recommendations for excavation in unstable rock mass (R 38)

1. Here, the rock mass is regarded as the rock and its joints. Stability is verified by means of rock mechanics investigations based on rigid body mechanisms. If these indicate that a rock mass cutting is unstable, supports are needed either:

   a) by means of stabilising individual rock masses in danger of slipping by targeted or uniformly distributed installation of rock nails or rock anchors, or;
   b) by means of a uniformly distributed, supported lining, in particular if a heavily fractured or weathered rock mass indicates that further fracture mechanisms may act in addition to the kinematics predetermined by the principal jointed structures.

   The force needed to support a rock mass cutting is known as the rock mass support force. The action of the rock mass on the retaining wall is known as the rock mass pressure.

   The following recommendations for excavations in unstable rock mass are based on the requirements for a supporting structure and therefore on the STR and GEO 2 limit states.

2. Although a wall displacement is necessary to allow the at-rest earth pressure to fall to the active earth pressure level when determining the active earth pressure according to R 8, Paragraph 4 (Section 3.1), it shall be assumed when determining the rock pressure that deformations are prevented as far as possible in order to retain the initial strength or the strength of the undisturbed rock mass. If displacements are allowed, the initial strength can be exceeded and a lower shear strength then governs, possibly leading to an increase in the rock pressure. The excavation lining and its supports shall therefore be designed to prevent displacement as far as is possible. All support components shall be installed immediately after cutting the rock mass and connected tightly to the exposed face. Struts and anchors shall generally be prestressed to the full characteristic anchor load P_k, immediately after installation.

3. In order to realistically estimate the rock mass properties for planning and construction of the excavation, the following shall be investigated in exposures according to the Eurocode 7 Handbook, Volume 2, and be continuously monitored during excavating, if possible in advance, e.g. by trenching:

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– extraction particulars, for example by loosening, excavating, chiselling, ripping, drilling, blasting;
– the rock types and their primary origin, grain size, mineralogical composition, porosity and voids;
– the geological structure, e.g. sedimentary, metamorphic and magmatic rocks;
– the joints, in particular the type of joint, spatial orientation, spacing, joint intensity, roughness, opening width, rock mass permeability, joint infill, number of joint sets, and size of joint infill;
– solid weathering, whether discoloured, slack, decomposed;
– variation of the solid when covered by water;
– the water conditions.

Also see EN ISO 14689-1.

4. Regardless of the supports and the type of retaining wall lining, the magnitude and distribution of the rock mass pressure are primarily dependent on:

   – the spatial orientation of the joints;
   – the joint intensity in the rock mass;
   – the roughness and nature of the discontinuity surfaces;
   – the extent and spacing of the discontinuities;
   – the degree of weathering;
   – the solid strength;
   – the shear strength of joints and the bedding plane or joint infill;

   and the resulting rock mass strength.

5. The following points apply in addition to Paragraph 4:

   a) The solid strength shall be determined on a sufficient number of samples using unconfined compression tests compliant with Recommendation No. 1 of the German Geotechnical Society's (*DGGT*) "Rock Testing Procedures" (*Versuchstechnik Fels*) Working Group [128], or using point load tests compliant with Recommendation No. 5 of the DGGT's "Rock Testing Procedures" Working Group [164]. The manner of determining the rock strength shall be given and identified as an estimate where necessary. Together with data on joint structure, this allows the rock mass strength to be estimated [129].
   b) Small-scale shear tests on joint samples can also provide valuable data on rock mass strength.
   c) The shear strength of the bedding plane or joint infill can be determined using soil mechanics methods. If the amount of soil sampled is not sufficient for this purpose the grain size composition of the bedding plane or joint infill shall be at least determined.

Large-scale tests according to Recommendation No. 4 of the DGGT's "Rock Testing Procedures" Working Group [78] are suitable to more precisely determine the shear resistance in potential sliding surface. This allows an assessment of the irregularities in the properties of the mass structure.

6. The properties of the undisturbed rock mass can be altered by external influences. For example:

   – vibrations from blasting;
   – slack or swelling phenomena caused by access of air or water or by relaxing movements of the rock mass;
   – alterations in porewater pressure in the joint infill and associated plastic flow caused by pressure redistribution;

   can all influence the magnitude and distribution of the rock mass pressure. Also note the provision in R 4, Paragraph 5 (Section 3.2).

7. In completely lined excavations, provisions shall be made for the discharge of perched and joint water. Otherwise, the water pressure shall be taken into consideration in addition to the rock mass pressure. Generally, the complete water pressure shall be adopted for the entire wall surface. If necessary, the rock mass shall be drained by means of horizontal drilling or by dewatering in advance of excavating – including for not completely lined excavations.

8. The elements of the excavation lining shall be designed for the rock mass pressure obtained according to R 39 (Section 11.2) and R 40 (Section 11.3), whereby the partial safety factors given in Table 6.1 of Appendix A 6 for the STR and GEO 2 limit states according to R 78, Paragraph 4 (Section 1.4) govern.

9. The struts or ground anchors required to support the rock mass cutting shall be designed for the design loads E_d obtained according to R 39 (Section 11.2) and R 40 (Section 11.3). The relevant regulations are:

   – R 52 (Section 13.7) for struts;
   – R 86 (Section 13.11) for ground anchors.

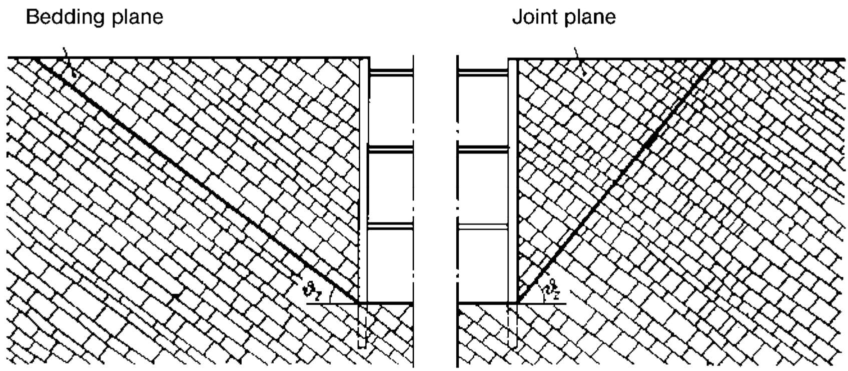
10. The length of ground anchors depends on the rock mass in which the grout bodies are embedded:

    a) If intact rock mass extends across the complete excavation height it is sufficient if the grout bodies are located behind the governing slip surface.
    b) If the grout bodies are located in soil or in a loosened, completely weathered or decomposed rock mass, the anchor length is given by analysis of lower failure plane according to R 44 (Section 7.3) or from the slip circle analysis according to R 45 (Section 7.4).

11.2 Magnitude of rock mass pressure (R 39)

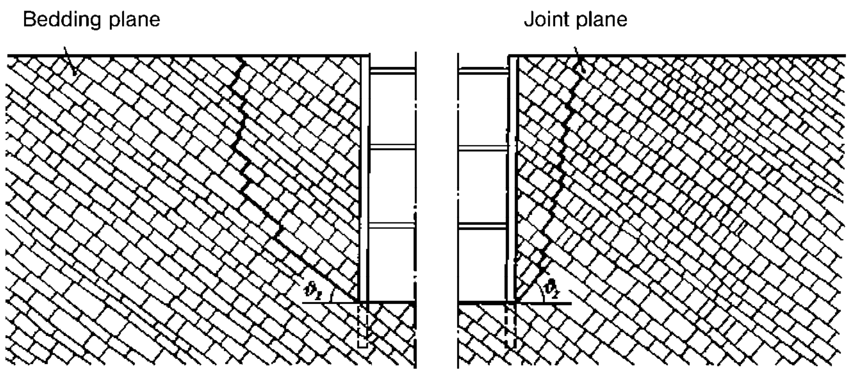
1. Generally, determination of the rock pressure is based on the existing joints. Three types of slip surface are differentiated:

   a) Slip surfaces in existing bedding planes (Figure R 39-1 a).
   b) Slip surfaces parallel to existing joint surfaces (Figure R 39-l b).
   c) Stepped slip surfaces in bedding planes and joints (Figures R 39-2 a) and R 39-2 b).



a) Slip surface in a bedding plane b) Slip surface parallel to joint plane

Figure R 39-1. Continuous slip surfaces in excavations in unstable rock mass



a) Sliding movement in bedding planes b) Sliding movement in joint planes

Figure R 39-2. Stepped slip surfaces in excavations in unstable rock

Where discontinuity spacing is small and joint intensity is high, consequently forming small blocks of rock relative to the size of the sliding body, it may be necessary to determine earth pressure as for soil.

2. In case of a bedding plane with a continuous slip surface, as shown in Figure R 39-1 a). the shear strength of the solid governs. The shear strength of the weaker layer governs if varying rock types are present. They may be only a few millimetres thick and be decomposed to soil, and may act as slip surfaces between the stronger rock strata. This also applies to stepped slip surfaces if sliding occurs in the bedding planes as shown in Figure R 39-2 a).

3. The following possibilities shall be differentiated for slip surfaces parallel to the jointing as shown in Figure R 39-1 b):

   a) If appropriate retaining wall linings and supports guarantee that no movements occur in the joints in all construction stages, and that therefore the material bridges have not cracked through, the rock shear strength in the material interfaces may be adopted as governing for determination of the rock pressure.
   b) If these conditions are not met it shall be assumed that the material bridges have cracked through as a result of unavoidable movements. The respective proportions of the shear strength of the joint infill in the existing joints and the shear strength of the material interfaces after cracking govern determination of the rock pressure. For high joint intensities the shear strength of the joint infill alone is governing.
   c) It shall be verified in both cases that the rock pressure from a stepped slip surface as shown in Figure R 39-2 b) can be accepted by the bracing. The shear strength of the joint infill is relevant for this purpose.

   If there is no joint infill in cases b) and c), and a high joint intensity, analysis may be based on the shear strength of the cracked solid.

4. The shear strength of the rock and the bedding plane or joint infill is generally determined according to R 38, Paragraphs 5 and 6 (Section 11.1). If the appropriate investigations have not been carried out, the characteristic value of the friction angle of the infill may be estimated as follows as a function of the granulometric composition:

   a) $\varphi'_k = 30°$ for sandy material;
   b) $\varphi'_k = 20°$ for silty material;
   c) $\varphi'_k = 10°$ for clayey material.

   Cohesion generally is not to be additionally adopted. Porewater pressure in slip surfaces shall be taken into consideration; it may also be necessary to adopt the shear strength at $\varphi_{u,k} = 0$. The undrained shear strength $c_{u,k}$ of the soil may only be adopted for these slip surfaces if based on separate investigations.

5. If the dip is not perpendicular or the strike is not parallel to the retaining wall as viewed in plan, the same analysis assumptions used to determine the earth pressure from soil self-weight and adoption of a given slip surface as shown in Figure R 6-1 b) (Section 3.4) may be adopted. An inclination angle between the orientation of the rock mass pressure and the normal to the wall may only be adopted if transfer of the vertical forces into the ground is guaranteed. See also R 84 (Section 4.8).

6. If the dip is not perpendicular or the strike is not parallel to the retaining wall as viewed in plan, the required rock mass support pressure is reduced. If the right angle is deviated from a force component parallel to the retaining wall occurs, the safe transfer of this component into the subsurface shall be demonstrated. In such cases additional investigations shall be performed to determine whether intersections occur, due to the existing joints, which dip perpendicular or obliquely to the lining. The partial sliding masses formed in this way can exert locally higher pressures on the retaining wall than were computed for the complete sliding mass. See [33] and [34], among others.

7. If more precise investigations have not been performed, or if sufficient local experience is not available, regardless of the numerical determination of the rock pressure according to Paragraph 2 or Paragraph 3, a computed minimum rock pressure on the excavation lining, which is obtained from the equivalent friction angle $\varphi'_{\text{Equiv,k}} = 40°$, according to the stipulations for earth pressure, shall be adhered to analogous to R 4, Paragraph 3 (Section 3.2). This also applies if the strike is oblique to the retaining wall. The equivalent friction angle may be adopted at $\varphi'_{\text{Equiv,k}} = 45°$ if the magnitude of the anticipated rock pressure is sufficiently well known from long-term measurements in similar conditions, and is checked in individual cases on the lining being installed.

8. If a greater rock mass pressure results on one side of a braced excavation than on the other, because of the different development of the slip surfaces, the higher load governs the design of the whole excavation structure, unless the computed minimum rock mass pressure is governing.

11.3 Distribution of rock pressure (R 40)

1. Because the magnitude and distribution of the rock mass pressure are a function of the rock mass disturbance level, concrete rules such as for the determination of earth pressure distribution in soil cannot be specified. The

load approach for the rock pressure shall be selected conservatively based on the respectively determined local conditions.

2. If intact rock mass extends across the complete excavation height, the rock mass pressure determined according to R 39 (Section 11.2) is generally adopted with rectangular distribution, due to the rigid body motion usually assumed for this case. In soil zones above the rock face or in completely weathered or decomposed rock mass an earth pressure distribution may generally be adopted according to the rules for soil. Because of the possible pressure redistribution, it is recommended to at least determine the support forces in the upper half of the wall or in the rock transition zone based on a rectangular pressure diagram.

3. To compensate for the relatively imprecise assumptions made for the distribution of the rock mass pressure, the action effects determined according to the stipulations of Paragraph 2 shall generally be increased by 30 % regardless of the type of excavation lining. These surcharges may only be dispensed with if the results of rock mass pressure distribution measurements were obtained under similar conditions and the pressure diagram based on them is confirmed by further measurements.

4. It is recommended:

   – to prestress all struts or anchors and to lock-off at the characteristic anchor load P_k according to R 38, Paragraph 2 (Section 11.1);
   – to carry out measurements in representative sections to facilitate timely recognition of deviations from the analysis assumptions and allow additional measures to be implemented.

11.4 Bearing capacity of rock mass for support forces at the embedment depth (R 41)

1. The resistance of the rock in front of the toe of a continuous retaining wall can be determined in analogy to determining the rock mass pressure. The governing slip surface is either a slip surface in a bedding plane or a slip surface parallel to the joint planes. An investigation according to R 39, Paragraph 2 governs in the one case and according to R 39, Paragraph 3 (Section 11.2) in the other. If groundwater can occur in the region of the wall toe, it may be necessary to take buoyancy and/or seepage into consideration.

2 . To prevent deformation, boreholes shall always be backfilled with hydraulically curing material, e.g. concrete, lime mortar or binding agents. The diameter of the borehole then governs determination of the rock resistance in

front of soldier piles. A three-dimensional effect may only be adopted if the joint intensity, number of joint sets, joint infill and joint orientation justify this. Without separate analysis, not more than half of the embedment depth, or a maximum of double the diameter of the concreted boreholes, may be adopted as the equivalent width for the three-dimensional effect.

3. Soldier pile walls and retaining walls with a similar support below the excavation level shall be examined for intersections of joints, running upwards from the concreted borehole to the excavation level. The partial sliding masses formed in this manner can govern determination of the rock mass resistance, in particular for shallow embedment depths.

4. A negative angle of inclination between the action axis and the normal to the wall may only be adopted for determination of the rock resistance inasmuch as this is allowed by the $\Sigma V_k = 0$ condition according to R 9 (Section 4.7).

5. The location of the support force for a retaining wall supported below the excavation level may be adopted as for cohesionless soil according to R 14, Paragraph 4 (Section 5.3) or R 19, Paragraph 4 (Section 6.3).

6. The partial safety factors given in Table 6.2 of Appendix A 6 shall be applied to the characteristic resistance of the rock mass when determining the design resistance.

12 Excavations in soft soils

12.1 Scope of Recommendations R 91 to R 101 (R 90)

1. Recommendations R 90 to R 101 apply to excavations in which soft, fine-grained soils, occasionally containing organic constituents, are prevalent in:

   a) favourable cases only above the excavation level;
   b) less favourable cases only below the excavation level;
   c) unfavourable cases both above and below the excavation level.

   The designation *soft soil* shall be regarded as a generic term, unrelated to the consistency index according to DIN 18122-1.

2. The soft soils discussed here are primarily layered, uniform, fine-grained soils according to DIN 18196, e.g. lacustrine clays and basin silts. In addition, softened boulder clays and flood plain loams, as well as organic soils such as lacustrine chalk, digested sludge, mud, tidal mud deposits and decomposed peat may be considered. These soils are generally normally consolidated but on occasion are still not completely consolidated under their own weight.

3. Each of the following soil properties taken on its own generally indicates the presence of a soft soil according to Paragraph 1:

   – very soft or liquid consistency corresponding to a consistency index $I_C < 0.50$ according to DIN 18122-11;
   – undrained shear strength of the soil $c_{u,k} \leq 20$ kN/m^2;
   – high vibration sensitivity, determined by the ratio of ultimate shear strength to residual shear strength in a vane test, or;
   – water content
     $w \geq 35$ % for soft soils without organic constituents or;
     $w \geq 75$ % for soft soils with organic constituents.

4. The following soil properties indicate the presence of a soft soil according to Paragraph 1:

   – soft consistency corresponding to a consistency index $0.75 > I_C \geq 0.50$ according to DIN 18122-1;
   – undrained shear strength of the soil 40 kN/m^2 $\geq c_{u,k} \geq$ 20 kN/m^2;
   – complete or almost complete saturation;
   – proneness to flow;
   – slightly plastic properties according to DIN 18 18196;
   – thixotropic properties, or;
   – organic constituent content.

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In individual cases, a decision to classify a soil as soft on the basis of these Recommendations shall not be solely dependent on a single criterion given here. However, if two of the criteria are met it can generally be assumed that a soft soil according to Paragraph 1 is present.

5. In all cases, the situation is aggravated if more permeable soil layers or bands, e.g. fine-sands, are intercalated with the soft soil and are subject to excess porewater pressure, regardless of whether this was already present before commencing construction measures or occurs as a result of excavation work or drawdown measures.

12.2 Slopes in soft soils (R 91)

1. Slopes in soft soils as defined in R 90 may be constructed without performing a stability analysis for excavation depths up to 3.00 m and slope angles up to $\beta = 45°$ if the following conditions specified in DIN 4124 "Excavations and Trenches" (*Baugruben und Gräben*) are adhered to:

   a) The undrained shear strength of the soil shall be $c_{u,k} \geq 20$ kN/m^2.
   b) If water-bearing layers or layers or bands subject to excess porewater pressure, are present in the soft soils, they shall be dewatered by means of vacuum.
   c) No heavy vibrations may occur, e.g. from traffic, driving work, compaction work or blasting.
   d) The ground behind the slope crest may not rise at more than 1 : 20 for a width up to five times the excavation depth, but for a maximum of twice the depth of the soft layer below the excavation level. A live load of $p_k = 10$ kN/m^2 at a distance of at least 1.50 m from the slope crest is permissible.
   e) On a horizontal ground surface, no earth fill inclined at more than 1 : 1 and higher than 1.50 m may be installed adjacent to a protective strip at least 1.50 m wide.
   f) Road vehicles and construction equipment up to and including 12 t gross weight shall adhere to a distance of at least 1.50 m between the outer edge of the contact area and the slope crest if load-bearing layers, e.g. a road pavement or natural ground with a total thickness of at least 0.50 m, are present above the soft soil or are built up to this level. Otherwise, the distance shall be increased to 2.00 m.
   g) Road vehicles and construction equipment of more than 12 t, and up to and including 40 t gross weight, shall adhere to a distance of at least 2.00 m between the outer edge of the contact area and the slope crest if load-bearing layers with a total thickness of at least 0.50 m are present above the soft soil or are built up to this level. Otherwise, the distance shall be increased to 3.00 m.

h) A berm immediately adjacent to the slope may not be subjected to loads from horizontal reaction forces from a retaining wall.
i) Any movement of the ground associated with construction of the slope shall remain within acceptable limits.

The additional engineering measures required to ensure stability shall be in accordance with Paragraph 2 to Paragraph 4.

2. If the ground:

a) is above the groundwater table at least as far as the excavation level;
b) is classified as soft according to DIN 18122-1 due to a consistency index of $0.75 > I_C \geq 0.50$;
c) is not classified as particularly difficult on the basis of any further criteria according to R 90 (Section 12.1) and;
d) does not display less favourable properties below the excavation level than above it;

no special measures are generally necessary for short-term construction stages. However, if the slope is exposed to weathering for an extended period, the slope surface shall be protected against erosion.

3. If the ground:

a) is above the groundwater table at least as far as the excavation level;
b) is classified as soft according to DIN 18122-1 due to a consistency index of $0.75 > I_C \geq 0.50$ and at least one further criteria according to R 90, Paragraph 3 or Paragraph 4 (Section 12.1) indicates particularly difficult soil conditions, or;
c) is classified as very soft according to DIN 18122-1 due to a consistency index of $I_C < 0.50$;
d) does not display less favourable properties below the excavation level than above it;

excavation may only proceed in short stages immediately followed by slope stabilisation, employing as a minimum slope toe stabilisation by means of a loaded filter or support element, e.g. of single-sized aggregate concrete on a geotextile base.

4. A slope that intersects the region below the groundwater table is generally only sufficiently stable if the soil is stabilised, e.g. by vacuum dewatering measures.

5. If the boundary conditions stipulated in Paragraphs 1 to 4 are not adhered to, slope stability shall be analysed using the shear strength parameter according to R 94 (Section 12.5) as described in DIN 4084 “Subsoil – Global Stability Analyses” (*Baugrund – Geländebruchberechnungen*). The safety factors for design situation DS-T are only valid if the anticipated deforma-

tions do not endanger buildings, pipelines, other structures or traffic areas. If such a risk cannot be ruled out because of local conditions, the partial safety factors for design situation DS-P shall be adopted and the utilisation factor limited to $\mu \leq 0.80$ when analysing global stability. It is recommended to adopt lower utilisation factors for highly organic materials. According to [110] and [165] a utilisation factor of $\mu \leq 0.75$ has proven reliable for North German tidal mud deposits with an ignition loss $V_{LOI} > 15$ % and a water content $w > 75$ %.

12.3 Wall types in soft soils (R 92)

1. If the execution of an excavation in soft soils using a slope according to R 90 (Section 12.1) is not possible because of space considerations, buildings, pipelines or other structures, or for other reasons, the excavation shall be supported by a wall system braced by struts, as far as this is possible, or tied back using anchors. Only walls that will not cause appreciable settlement in the surrounding soft ground or of other structures during construction may be utilised as excavation linings. A settlement hazard or danger of horizontal movement exists if the soil liquefies or is displaced during installation or construction of the wall. Generally, the following wall types are suitable for excavations in soft soils:

   a) sheet pile walls;
   b) bored pile walls;
   c) diaphragm walls.

   Also see Paragraphs 2 to 4. Soldier pile walls and bored pile walls with infilling installed between the piles during excavation are generally unsuitable as excavation linings in soft soils.

2. Care shall be taken to keep the effects of vibrations on neighbouring buildings to a minimum when installing sheet pile walls. The guide values for allowable vibration velocities according to DIN 4150-3 are generally too high for the soil conditions stipulated in R 90 (Section 12.1), because neighbouring buildings on shallow foundations in soft soil have often previously been subjected to deformations associated with an increased internal stress state and therefore only possess minor deformation reserves. Moreover, vibration-sensitive soils can suffer strength losses as a result of increases in porewater pressure, up to and including liquefaction. The hazard of ground liquefaction and therefore of settlement in neighbouring buildings is greater for vibratory techniques than for impact driving. The following demands shall be met by the planned installation methods:

a) When installing the sheet piles with a pile hammer, the driving energy per impact and the impact frequency shall be defined on the basis of previous piling tests according to Paragraph 5. Cautious installation in soft soils can generally be achieved if vibrations are allowed to fade between two separate impacts.
b) Vibration techniques are unsuitable if the soil is very vibration-sensitive, displays a proneness to thixotropic behaviour or has interbedded, saturated bands of fine sand. Sheet piles can only be vibrated-in in soft, highly plastic soils with low vibration sensitivity. Even when favourable conditions for the use of vibration techniques apply in this regard, vibration velocities shall be kept to a minimum. Driving tests according to Paragraph 5 are required for this purpose. Empirical values show that vibrations in neighbouring buildings are lowest at operating speeds greater than 2000 rpm. Moreover, particularly heavy vibration effects caused by switching on and off shall be prevented by using vibration hammers with variable balance weights.
c) The jacking method is particularly suitable in homogeneous, soft soils without obstructions. Top soil layers with large jacking resistances, e.g. made ground including construction wastes, shall be prepared for jacking by pre-drilling or by soil replacement.

3. EN 1536, DIN SPEC 18140 and [165] apply to the installation of bored pile walls. In addition, the following points shall also be observed:

   a) A low-vibration drilling method shall be selected to install the individual piles of a bored pile wall. Soil displacement caused by pile drilling shall be prevented, for example by:

      – selecting a larger pre-penetration of the casing tube than that demanded by EN 1536;
      – avoiding a drill bit that protrudes outside of the diameter of the casing tube;
      – using drill bits that possess a cutting edge instead of teeth;
      – using drilling tools that exert as small a suction effect as possible at the bottom of the borehole.

   It may also prove expedient to maintain a constant positive water pressure in the borehole as described in EN 1536.

   b) In principle, the following types of implementation may be considered:

      – bored pile walls using secant piles;
      – bored pile walls using sealing piles, i.e. small diameter unreinforced piles installed in the rear interstices of the neighbouring bored piles;
      – tangent bored pile walls with subsequent closing of the spaces during excavation;

- The unreinforced piles or sealing piles shall be extended to the depth below the excavation level obtained from analysis of the safety against base heave or against hydraulic heave.

c) The following points shall be observed when selecting the implementation method:

- Drilling the primary piles without a protruding drill bit is only possible on secant piles as long as the concrete is not completely set. Furthermore, pre-penetration of the casing tube below the bottom of the borehole is not possible.
- If the sealing piles are manufactured using drilling techniques, there is a danger of lateral displacement, in particular at localised projections on the wall piles. If they are manufactured using jetting techniques, the surrounding soil may be softened locally, thus presenting a settlement hazard to neighbouring structures. The cement slurry setting process is not reliable in organic soils.
- By driving wooden wedges, for example, squeezing of the soft soil through the unavoidable gaps between tangent bored piles can often be prevented, but cannot guarantee only limited groundwater drawdown outside of the retaining wall. Moreover, there is a danger of strong impacts and heavy vibrations if the drill bit catches on protrusions on a neighbouring pile. In addition, this type of pile installation requires that the soft soil is present above the excavation level only.

d) Uncased boreholes supported by a slurry shall be manufactured in accordance with the stipulations for diaphragm walls. Auger bored piles are less suitable due to the danger of uncontrolled soil displacement.

e) If the shear strength in an undrained shear test is $c_{u,k} \leq 15$ kN/m^2 or the consistency index is $I_C \leq 0.25$, direct concreting against the soil is not permissible according to EN 1536. This stipulation can be ignored if the pile wall above the excavation level is carefully examined for defects during excavation.

4. EN 1538 and DIN 4126 apply for constructing and analysing diaphragm walls. In addition, the following points shall also be observed:

a) Where possible, the distance to neighbouring buildings with foundations in soft soil, in particular heavily loaded gable foundations, shall be more than half of the trench depth, but at least 5 m, or the trench be located outside of the bearing failure zone.

b) When analysing trench stability, the fact that no arching effect can be assumed in soft soils shall be taken into consideration. Therefore, the slurry pressure:

$$\sigma_{s,k} = \gamma_F \cdot z$$

in regions with soft layers at a depth z shall be at least 10 % greater than the total horizontal pressure with $\delta_{a,k} = 0$:

$\sigma_{h,k} = e_{a,k} + w_k$

from active earth pressure $e_{a,k}$ and water pressure w_k. Here, where the initial condition of the unconsolidated soil:

$e_{a,k} = \sigma'_{z,k} - 2 \cdot c_{u,k}$

and the final condition of the consolidated soil:

$e_{a,k} = \sigma'_{z,k} \cdot K_{ag} - c'_k \cdot K_{ac}$

shall be investigated. The effective overburden pressure $\sigma'_{z,k}$ is obtained from the unit weight of the wet or buoyant soil. The water pressure is obtained from the unit weight of water and the depth below the groundwater table using the equation:

$w_k = \gamma_w \cdot z.$

The slurry pressure shall be increased where required, e.g. by deepening the guide walls or by using a high-density suspension. However, it shall be taken into consideration that a higher slurry pressure can push out soft soils.

c) The safety against slippage of single grains or grain groups, the safety against slipping of a failure wedge into the trench and the appropriate composition of the slurry shall be tested in a trial trench.
d) In addition, the trench execution sequence, the wet concrete pressure and the concreting technique may affect the loads on the soft soil, see [139].

5. Although not previously discussed in Paragraphs 2, 3 or 4, the selected installation or construction method shall always be tested on the site in question before starting work, but at a reasonable distance from existing, neighbouring structures. This allows the process to be optimised on the basis of parallel investigations on such things as concrete requirements, integrity tests, and vibration and settlement measurements.

6. In principle, bracing is less flexible than anchors. If anchors are used nevertheless, the grouted sections shall be located in soil of sufficient bearing capacity. The same applies to the grouted sections of anchors anchoring a concrete base slab. The anchor installation method shall ensure that soil displacement, softening or loosening is prevented.

7. Regardless of the type of wall system selected, the working level shall be constructed so that the soft soil does not lose its bearing capacity when

construction equipment is operating on it, and it does not begin to flow. If an existing layer of fill cannot be used for this purpose, the soft soil shall be protected as deemed necessary or be replaced by a load-bearing layer. Furthermore, only construction equipment exerting small loads, e.g. from bearing pressure or vibrations, shall be utilised to install or construct walls or, if applicable, to manufacture wall or base slab anchorages. If necessary, load distributing excavator mattresses shall be used.

12.4 Construction procedure in soft soils (R 93)

1. Because the anticipated displacements in excavations in soft soils only allow:

   – a fixed-earth support of the retaining wall in the initial advancing stage with a very small excavation depth;
   – a limited free-earth support of the retaining wall below the excavation level;

   the following procedures are useful [100], depending on the excavation depth and dimensions, and the soil and groundwater conditions. They assume the most unfavourable case of soft soil from ground level to far below the excavation level. If more favourable conditions prevail in places, the measures described may be correspondingly adjusted.

2. Regardless of the excavation depth, a continuous head beam executed as waling or a wale runner, which is capable of redistributing the earth pressure from the area of the excavated strip to neighbouring areas, shall be installed for sheet pile walls. This also serves to limit the anticipated head deflections. In this regard it is particularly useful to arrange this head beam as waling for a row of struts at ground level. The same applies to in-situ concrete walls, if constructive measures are not taken to ensure that the individual diaphragm slices or individual piles cannot move separately.

3. The following procedure can be adopted for shallow excavations, generally up to 3 m, and small plan dimensions:

   a) Within a daily shift:

      – an approximately 2 m to 3 m wide, laterally sloped trench, parallel to the narrow end of the excavation at the elevation of the planned excavation level is excavated and;
      – a stiffening strip of blinding concrete is installed below the planned foundation level of the new building.

   b) By continuing the blinding concrete in strips as shown in Figure R 93-1, a lateral support is provided to the retaining walls at the excavation

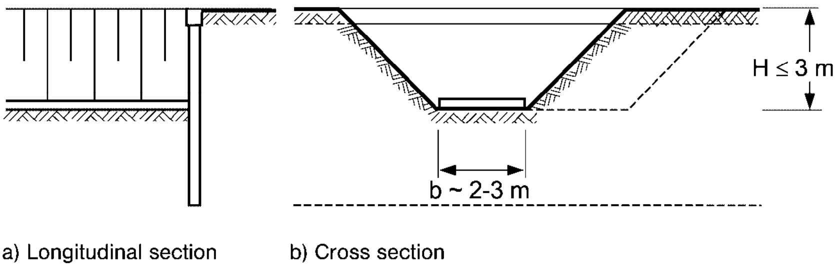

Figure R 93-1. Unsupported retaining wall in soft soil after installing the first strip of blinding concrete

level. It may be expedient to arrange the blinding concrete strips diagonally in the corners of the excavation.

c) If the retaining wall head displays unacceptable deflections when using this method:
   - the trench shall be supported by vertical walls;
   - bracing shall be installed at ground level according to Paragraph 4, or;
   - the core method as described in Paragraph 6 shall be selected.

   It may be expedient and sufficient to manufacture several blinding concrete strips at large intervals in lined trenches and to thus achieve an effective bracing of opposing retaining walls, before supplementing them with the intermediate blinding concrete strips in trenches sloped on one side only.

4. For medium depth excavations, generally 3 m to 5 m, a low-deformation bracing structure shall be installed in two or more excavation stages. If the excavation plan allows, bracing can be installed directly against the opposite wall. In principle, the construction procedure is then as follows:

   a) If the uppermost row of struts is not already installed approximately at ground level as part of the head beam installation according to Paragraph 2, but lower instead, the soil can be excavated in an initial stage according to Paragraph 3 a) in strips as far as the excavation level of this stage and a strut installed as appropriate.
   b) The soil can also be excavated in strips to the final excavation level according to Paragraph 3 a) and a stiffening blinding concrete strip manufactured at each strip.
   c) If a second or more rows of struts are planned, the procedure according to Paragraph a) repeats, before the final stage according to Paragraph b) concludes the excavation phase.

   Figure R 93-2 shows a single-propped wall after installation of the first strip of blinding concrete.

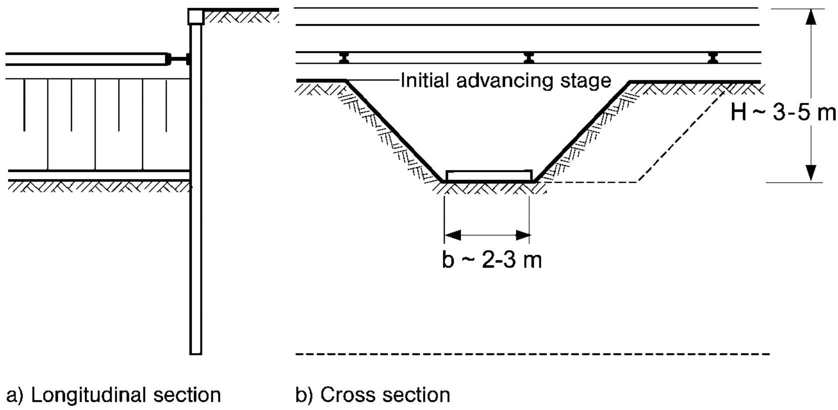

Figure R 93-2. Single-propped retaining wall in soft soil after installing the first strip of blinding concrete

5. The following points for manufacturing the blinding concrete strips according to Paragraph 3 or Paragraph 4 shall be observed:

   a) The stiffening blinding concrete shall be manufactured using fast hardening cement to allow the work schedule to be quickly completed.
   b) The thickness of the blinding concrete depends on the structural analysis, but shall not be less than 0.20 m.
   c) The blinding concrete strips shall be reinforced where necessary.

6. If the construction procedure according to Paragraph 3 or Paragraph 4 is not possible due to the large plan dimensions of the excavation, the following procedure applies:

   a) In an initial stage the central section of the foundation slab and the basement are manufactured in a sloped or supported excavation. In a sloped excavation, sufficiently wide berms shall be provided to enable reliable and low-deformation support of the retaining wall.
   b) In a second stage, intermediate bracing is installed against the completed central foundation, the berms removed in strips and the bracing base slab extended to the retaining walls. In this manner, each wall is gradually provided with a continuous support at the excavation level.

7. In deep excavations, generally more than 5 m, in deep, soft soils, it may be necessary to create a toe support for the retaining wall by means of a bracing base, e.g. using jet grouting techniques [102, 104]. During the course of excavation the retaining wall is supported above the excavation level by means of struts, and where necessary by anchors, or using the core or top-down methods according to Paragraph 6.

8. Soft soils are particularly sensitive to dynamic loading and to changes in the initial stress condition as a consequence of excavation. In order to minimise the risk of boiling the soft soil at the excavation level may not be traversed by vehicles and in extreme cases may not even be traversed unprotected on foot. Excavation shall always be carried out from a higher level. If this is not possible in all areas a sufficiently thick working subgrade shall be installed to protect the soft soil.
9. Soft soils, and especially fine-sand- and silt-banded soils below the groundwater table, are particularly susceptible to boil. The construction procedures described in Paragraphs 3, 4 and 6 using temporary slopes and berms can often only be implemented after the soft soil has been stabilised, e.g. by means of vacuum wells or vacuum lances [103]. Also see R 100, Paragraph 3 and Paragraph 4 (Section 12.11).
10. Because soil arching cannot reliably develop in soft soils and stress redistribution in the soil is directly associated with wall displacements, all strut removal and strut redistribution measures shall be executed with little or no deformation, generally with the help of hydraulic presses and in small stages.
11. There is a permanent risk of heave or hydraulic heave in excavations in soft soils. This can lead to substantial heave at the excavation base and to large settlements behind the retaining wall. Depending on the respective boundary conditions, one or more of the following measures shall be provided for in order to minimise the associated danger of settlement damage to neighbouring structures:
    a) Construction in small stages according to Paragraph 4.
    b) Downward extension of the wall beyond that required for a toe support.
    c) Installation of blinding concrete in stages from wall to wall, as a vault or a reinforced flexural beam.
    d) Anchoring of blinding concrete installed in stages, a base slab installed in stages or a previously manufactured, jet grouted base utilising tension piles.
12. Because the behaviour of retaining structures in soft, cohesive soil and the deformations of the ground outside of the excavation cannot always be predicted with the required reliability, it is imperative that the individual components of the retaining structure, the ground and the neighbouring structures are monitored and measured from the outset. Also see R 31 to R 37 (Section 14) and DIN 4123. If measurements indicate that unacceptably large movements are anticipated, with regard to neighbouring buildings, pipelines, other structures or traffic areas, a different construction procedure shall be adopted or additional measures implemented.

12.5 Shear strength of soft soils (R 94)

1. Geotechnical Category GC 3 according to the Eurocode 7 Handbook, Volume 1 shall be adopted for the site investigation in conjunction with excavations in soft soils according to R 90 (Section 12.1) if:

   a) the soft soil is excavated for a height of more than 3 m;
   b) the excavation depth is more than 5 m or;
   c) impacts on neighbouring buildings, pipelines, other structures or traffic areas are anticipated.

   If any one of these conditions apply a geotechnical expert shall be consulted.

2. Knowledge of the prevalent and the anticipated porewater pressure conditions is particularly important when designing retaining structures in soft soil. The geotechnical investigation shall therefore clarify and detail:

   a) Whether the soft soil has already consolidated under its own weight or whether excess porewater pressure generated during previous construction measures still exists.
   b) Whether excess porewater pressure resulting from changes in governing regions of the ground during excavation is anticipated.

   Based on these findings, it shall be decided in each individual case whether analysis shall be based on the drained or the undrained shear strength of the soil, or on a shear strength lying between these two limits. Also see Paragraph 10.

3. The criteria for anticipated excess porewater pressure in a soil normally consolidated under self-weight and therefore subject to undrained conditions are given in [105, 111] and [119]. Approximately drained boundary conditions can often be anticipated if:

   a) The investigations described in [111] demonstrated that drained conditions can very often be anticipated for the boundary conditions usually prevalent in practice.
   b) Investigations of the effective stress paths described in [119, 166] and [167] demonstrated that an unloading situation occurs in most areas in the ground and that despite deflection of the wall no excess porewater pressure occurs in front of the wall toe, or that only minor excess porewater pressure occurs as a function of the flexural stiffness of the retaining wall, and generally dissipates completely within a few days.

   It is therefore not generally necessary to consider undrained conditions mathematically. If excess porewater pressure development according to Paragraph 2 is anticipated and the governing conditions are therefore undrained, local experience shall also be drawn upon for the evaluation or investigations coupled with consolidation analyses shall be performed.

4. The following situations are differentiated according to the shear test boundary conditions:

   a) The drained shear strength of the soil with the parameters φ'_k and c'_k; however, also see R 95, Paragraph 3 (Section 12.6).
   b) The drained angle of total shear strength $\varphi'_{s,k}$ according to DIN 18137-1 including friction and cohesion components.
   c) The undrained shear strength of the soil with the parameters $\varphi_{u,k}$ and $c_{u,k}$, where $\varphi_{u,k} = 0$ is generally assumed.

   The shear parameters φ'_k and c'_k, and the angle $\varphi'_{s,k}$ of the total shear strength of the drained soil are generally obtained from triaxial tests according to DIN 18137-2 or from direct shear tests according to DIN 18137-3. These tests are only of limited suitability for determining the shear strength of very soft**,** slightly plastic soils, see Paragraph 5.

5. Determination of the drained and undrained shear strength of soils in laboratory tests can be heavily influenced by both random and systematic errors:

   a) Sampling errors and errors installing the sample in the shear box or triaxial cell can lead to strength reductions.
   b) Increases in apparent strength can be suggested in direct shear tests as a result of frictional resistance in the shear box.
   c) The resistance of the rubber membrane in triaxial tests may lead to an apparent increase in strength.

   For these reasons, the values determined in laboratory tests, in particular for the cohesion c'_k of the drained soil and for the shear strength $c_{u,k}$ of the undrained soil, shall be carefully assessed when stipulating the calculation values. A cohesion of $c'_k \approx 0$ is normally anticipated in any case for normally consolidated soft soils without organic constituents, giving $\varphi'_k \approx \varphi'_{s,k}$. With regard to the influence of anisotropy on the shear strength, also see Paragraph 9.
6. Failing the relevant experience for excavations in soft soils, the in-situ undrained shear strength $c_{u,k}$ of the soil shall be determined by vane shear tests according to EN ISO 22476-9, in addition to the usual site investigation measures and laboratory tests. These tests shall be carried out to a depth at which the soil strength noticeably improves, but to at least three times the depth of the excavation for deep, soft soil layers. The value $\tau_{f,k}$ is obtained from the vane shear test. In order to take the differing loading rates during the shear vane test into consideration, and the shear stresses

during excavation, the associated value $c_{u,k}$ shall be determined with the aid of a correction factor μ from the relationship:

$c_{u,k} = \tau_{f,k} \cdot \mu$.

Short-term and long-term construction stages may be differentiated:

a) The relationship between the factor μ and the plasticity index I_P according to DIN 18122-1 as shown in Figure R 941 (lower curve) applies for long-term construction stages according to [113].
b) The corrected upper curve as shown in Figure R 94-1 represents the relationship between the factor μ and the plasticity index I_P according to DIN 18122-1 for short-term construction stages according to [116] in [132].

Construction stages in which a locally limited critical situation is redressed on the same day by the rapid installation of effective stabilisation measures are considered to be short-term.

7. If carrying out vane shear tests does not appear to promise success, e.g. in fibrous, organic soils, the undrained shear strength $c_{u,k}$ of the soil may be alternatively estimated in the course of geotechnical investigations as follows:

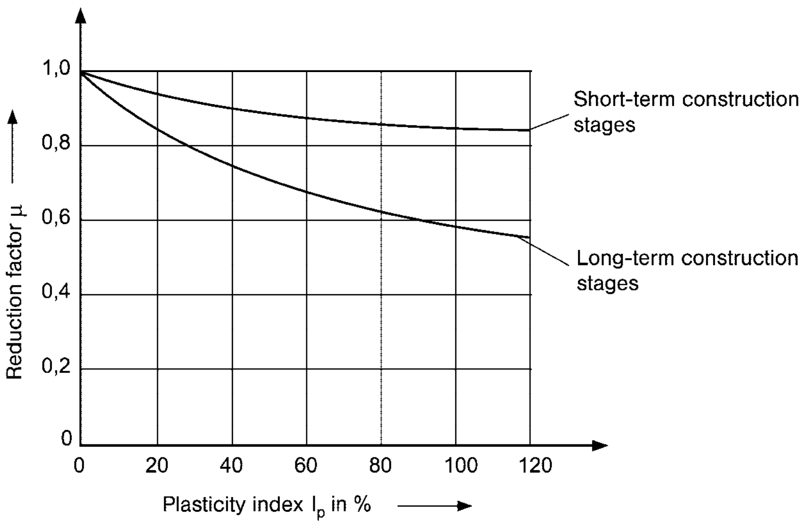

Figure R 94-1. Reduction factor μ when using the shear vane to determine the shear strength $c_{u,k}$

a) If relevant regional experience or reliable correlations are available [112], the shear strength $c_{u,k}$ can be derived with the aid of a coefficient λ_{cu} and the effective overburden pressure:

$c_{u,k} = \lambda_{cu} \cdot \sigma'_{v,k}$.

Where:

$\sigma'_{v,k} = \gamma' \cdot z$

if the groundwater table is at ground level. If it is lower, the unit weight γ of the wet soil or the unit weight γ_r of the saturated soil shall be adopted for the soil layer above groundwater.

b) In addition, indirect determination of the shear strength $c_{u,k}$ from cone penetration tests according to [120], using:

$c_{u,k} = (0.05 \text{ to } 0.10) \cdot q_c$

may be considered. For further information on the relationship between the shear strength $c_{u,k}$ and the cone resistance q_c see [114, 115].

Deriving the shear strength $c_{u,k}$ by means of correlations against the consistency index I_C is not recommended [106].

8. Taking the variance of the measurement results into consideration, the governing calculation value for the shear strength $c_{u,k}$ shall adopted so that analyses provide conservative results. In this way it represents a cautious estimate of the mean value in the respective soil region.

9. Because of the anisotropy of the soil as a consequence of sedimentation and the rotation of the principal stress directions as a consequence of soil excavation, the undrained shear strength $c_{u,k}$ of the soil shall normally be increased when determining the earth pressure and reduced when determining the passive earth pressure [105, 113]. As this influence can only be estimated numerically with difficulty, but both effects partly cancel each other out and an analysis using two different shear strengths would lead to difficulties, it is recommended to not consider it in the analysis and the impacts thus not assessed be compensated for by increasing the safety factor when adopting the passive earth pressure, see R 96, Paragraph 3 (Section 12.7). In contrast, taking the governing effective stress paths for excavations and the anisotropy of the soil into consideration, no substantial deviation for the angle of total shear strength $\varphi'_{s,k}$ was determined [166].

10. If excess porewater pressure is anticipated in specific situations, the shear strength $c_{u,k}$ for each layer shall be converted to an equivalent friction angle equiv. $\varphi_{s,k}$ as follows:

a) If the shear strength $c_{u,k}$ increases approximately linearly with depth as shown in Figure R 94-2 a), then:

$$\sin(\text{ers}\,\varphi_{s1,k}) = \frac{c_{u1,k}}{\sigma'_{v1,k}}$$ above the groundwater table, where $\sigma'_{v1,k} = \gamma_1 \cdot z_1$.

$$\sin(\text{ers}\,\varphi_{s2,k}) = \frac{\Delta c_{u2,k}}{\sigma'_{v2,k} - \sigma'_{v1,k}}$$ below the groundwater table, where $\sigma'_{v2,k} = \gamma_1 \cdot z_1 + \gamma'_2 \cdot z_2$.

b) If the shear strength $c_{u,k}$ is approximately constant both above and below the groundwater table as shown in Figure R 94-2 b), then:

$$\sin(\text{ers}\,\varphi_{s1,k}) = \frac{c_{u1,k}}{\sigma'_{vm1,k}}$$ above the groundwater table, where $\sigma'_{vm1,k} = \tfrac{1}{2} \cdot \gamma_1 \cdot z_1$.

$$\sin(\text{ers}\,\varphi_{s2,k}) = \frac{c_{u2,k}}{\sigma'_{vm2,k}}$$ below the groundwater table, where $\sigma'_{vm2,k} = \gamma_1 \cdot z_1 + \tfrac{1}{2} \cdot \gamma'_2 \cdot z_2$.

An additional layer boundary shall be introduced at the excavation level if this substantially improves correlation for the best-fit curve. In all cases, further analysis may be performed using the equivalent friction angle equiv. $\varphi_{s,k}$ on the basis of effective stresses, although the undrained shear strength of the soil is assumed.

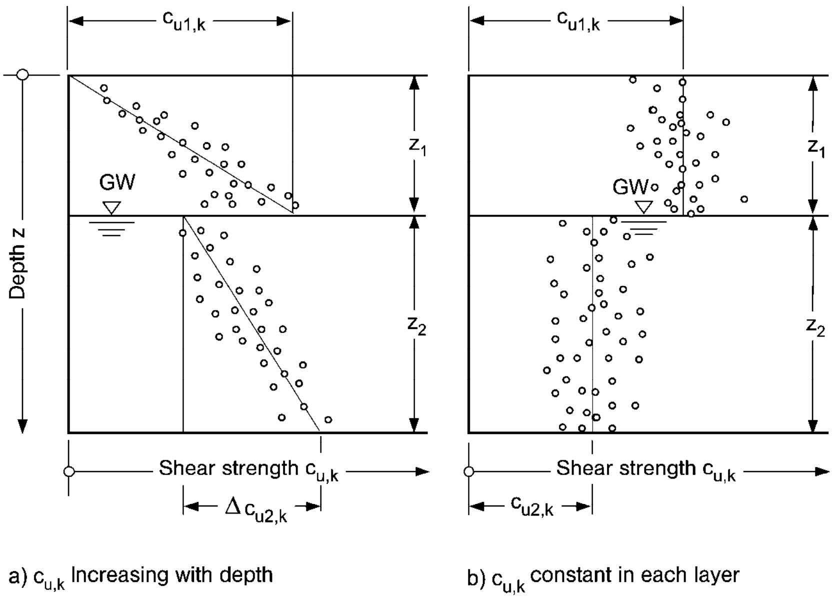

Figure R 94-2. Determination of the equivalent friction angle equiv. $\varphi_{s,k}$

11. If excess porewater pressure can be ruled out in specific situations and the effective shear strength with the shear parameters φ'_k and c'_k was not determined in the laboratory, the angle of total shear strength $\varphi'_{s,k}$ for the drained condition may be determined from the undrained shear strength $c_{u,k}$ of the soil as defined in Paragraph 8, based on [119].

12. Angles of total shear strength larger than $\varphi'_{s,k} = 27.5°$ or equivalent friction angles larger than equiv. $\varphi_{s,k} = 27.5°$ may only be adopted if the author of the design draft or the technical planner possess the requisite knowledge and experience.

12.6 Earth pressure on retaining walls in soft soils (R 95)

1. The same principles apply for determination of the magnitude of the earth pressure load and for the earth pressure distribution on retaining walls in soft soils as for excavations in firm to semi-solid soils, if no other stipulations are made below. The same applies accordingly for the stipulations for excavations adjacent to structures.

2. According to [105] and [112], the shear parameters of the undrained soil $\varphi_{u,k}$ and $c_{u,k}$ may be adopted for determination of the earth pressure acting on retaining walls in soft, unconsolidated soils. In addition, the total stresses in the soil may be assumed and thus earth pressure and water pressure be applied in a single computation. The following points speak against this approach:

   a) Merely determining the earth pressure and passive earth pressure alone, taking the water pressure into consideration separately, often returns unreliable results because:

      – mathematically, no earth pressure acts to a depth depending on the magnitude of the parameter $c_{u,k}$;
      – active and passive earth pressure increase equally with depth, therefore leading to decreasing mathematical safety against failure of the toe support with increasing embedment depth.

   b) In stratified ground, an analysis using total stresses may only be adopted for the soft layers, but not for the stiffer layers. Different procedures are therefore used for a single computation.

   Accordingly, this approach will not be further pursued here.

3. The following regulations assume that the shear strength is always adopted as a friction angle according to R 94 (Section 12.5), either:

a) as the angle of the total shear strength $\varphi'_{s,k}$ determined in a shear test according to R 94, Paragraph 4 b), or;
b) as an equivalent friction angle equiv. $\varphi_{s,k}$ according to R 94, Paragraph 10, based on the shear strength $c_{u,k}$, or;
c) as an angle of total shear strength $\varphi'_{s,k}$ according to R 94, Paragraph 11, based on the shear strength $c_{u,k}$.

Analysis shall be performed using effective stresses in all cases. See R 97 (Section 12.8) for adopting positive water pressure and, if necessary, excess porewater pressure.

4. The at-rest earth pressure is the starting point for investigating a retaining wall in soft soil, similar to firm or semi-solid soil:

$e_{0g,k} = \gamma \cdot K_0 \cdot z_a$.

In saturated soil γ is replaced by:

- γ_r above the groundwater table;
- γ' below the groundwater table.

The following empirical approximations are available for the at-rest earth pressure coefficient of a soil consolidated under self-weight:

a) The usual approach as a function of the friction angle:

$K_0 = 1 - \sin \cdot \varphi'_{s,k}$.

b) As a function of the plasticity index the:

$K_0 = 0.24 + 0.310 \cdot \log I_P$

approach after *Lee* and *Jin* (1979) is adopted [112]. The plasticity index I_P is given in %.

As a function of the water content w_L at the liquid limit the:

$$K_0 = 10^{0,00275 \cdot (w_L - 20) - 0,2676}$$

approach after *Sherif* and *Koch* (1970) is adopted [112]. The water content w_L is given in %.

An evaluation of the given empirical equations gives the following at-rest earth pressure coefficients for normally consolidated, cohesive soils:

a) $K_0 = 1 - \sin \varphi'_{s,k}$		b) *Lee* and *Jin*		c) *Sherif* and *Koch*	
φ_s	K_0	I_P	K_0	w_L	K_0
30°	0.500	5 %	0.456	10 %	0.507
25°	0.577	15 %	0.605	20 %	0.540
20°	0.658	25 %	0.673	30 %	0.575
15°	0.741	35 %	0.719	40 %	0.613
10°	0.826	45 %	0.752	50 %	0.653

Approach a) is regarded as the most reliable. If the author of the design draft or the technical planner possess the requisite knowledge and experience, approaches b) and c) may also be used in an assessment.
Any excess porewater pressure shall be adopted according to R 97, Paragraph 6 (Section 12.8).

5. The following points apply when adopting at-rest earth pressure:

   a) At-rest earth pressure may only be adopted above the excavation level as shown in Figure R 96-3 b) (Section 12.7) if wall deflections toward the excavation at the top or at the excavation level are almost completely prevented as a result of the construction procedure selected. This can be the case if, e.g.:

      – a low-deformation wall is installed;
      – a stiffening base slab is manufactured from ground level by jet grouting or by soil stabilisation and;
      – the top supports are installed and prestressed without appreciable excavation.

   b) At-rest earth pressure may only be adopted below the excavation level as shown in Figure R 96-3 (Section 12.7) if wall deflections towards the excavation at the toe or at the excavation level are almost completely prevented as a result of the construction procedure selected, or if wall deflection against the ground is anticipated. This can be the case, for example, if a stiff wall is installed and a stiffening base slab is manufactured from ground level using jet grouting techniques.
   c) Because of their deformability, it may be practical to only adopt an increased active earth pressure for sheet pile walls above and, if necessary, below the excavation level as shown in Figure R 96-4 a) (Section 12.7) in the sense of R 22 (Section 9.5), for cases in which the at-rest earth pressure is adopted for low-deformation walls. If the supports are also heavily prestressed, a large deflection of the top of the wall towards the soil may develop, which in turn leads to a rotation of the wall toe towards the excavation below the stiffening base slab, in particular in conjunction with the earth pressure from building loads and water pressure, so that only the active earth pressure still acts in this zone as shown in Figure R 96-4 b) (Section 12.7).

It shall be ensured at all times that the selected earth pressure approach approximately conforms to the computed deformations and deflections of the wall. At the least, it shall not obviously contradict the determined deformations and displacements.

6. The following points apply for determination of the active earth pressure:

   a) Similar to firm or semi-solid soils, the magnitude of the active earth pressure in soft soil is obtained from:

   $e_{ag,k} = \gamma \cdot K_a \cdot z_a$.

   In saturated soil γ is replaced by:

   – γ_r above the groundwater table;
   – γ' below the groundwater table.

   b) In soft soils it may be assumed that adhesion acts between the retaining wall and the ground. In simplification, it is permissible to adopt the wall friction angle $\delta_{a,k} = {}^1/_3 \cdot \varphi_k$ in place of this adhesion, where $\varphi_k = \varphi'_k$ or $\varphi'_{s,k}$ according to R 94, Paragraph 4 or φ_k = equiv. $\varphi_{s,k}$ according to Paragraph 10 (Section 12.5).

7. The following points apply for adopting the active earth pressure:

   a) The active earth pressure shall be adopted if the measures discussed in Paragraph 5 a) are not implemented. This is particularly the case:

   – if the top support is installed relatively deep;
   – if a wall support is subject to a ground reaction at the wall toe;
   – if a stiffening base slab according to R 93, Paragraph 3 (Section 12.4) is installed in strips.

   b) If undrained conditions are assumed when determining the earth pressure and the equivalent friction angle equiv. $\varphi_{s,k}$ is determined according to R 94, Paragraph 10 (Section 12.5), the active earth pressure may be greater than the at-rest earth pressure for very low shear strength values. In this case, the at-rest earth pressure may be adopted for determination of the actions on the wall.

8. A classical earth pressure distribution shall generally be assumed for excavations in soft soils, in particular if the wall can undergo greater displacements at the top than at the excavation level as a result of the planned construction procedure. However, if an upper support is prestressed on the one hand, but a support at the wall toe is subject to ground reaction on the other, earth pressure redistribution shall be assumed. The earth pressure from ground level down to the excavation level shall then be converted to a trapezoid or, at the most, a rectangle.

9. These stipulations apply to a homogeneous soil and a groundwater table at or below ground level. The following points shall be observed:

a) These stipulations only apply for determination of the earth pressure below the groundwater table in conjunction with R 97 (Section 12.8).
b) See R 99, Paragraph 6 (Section 12.10) for consideration of changes in stratified ground.

12.7 Ground reactions for retaining walls in soft soils (R 96)

1. The ground reactions below the excavation level can assume any value between the active earth pressure and the passive earth pressure in the limit state, depending on the wall displacements. The following cases are differentiated when adopting the ground reactions:

   a) Construction stages without a stiffening base slab as shown in Figure R 96-1.
   b) Construction stages where a stiffening base slab is installed in strips in the course of excavation as shown in Figure R 96-2.
   c) Construction stages with a stiffening base slab previously installed from ground level as shown in Figures R 96-3 and R 96-4.

   The load diagrams assume that not only the active earth pressure and the at-rest earth pressure, but also the passive earth pressure for accepting the ground reaction according to R 95, Paragraph 3 (Section 12.6), were determined using the angle of total shear strength $\varphi'_{s,k}$ or the equivalent friction angle equiv. $\varphi_{s,k}$.

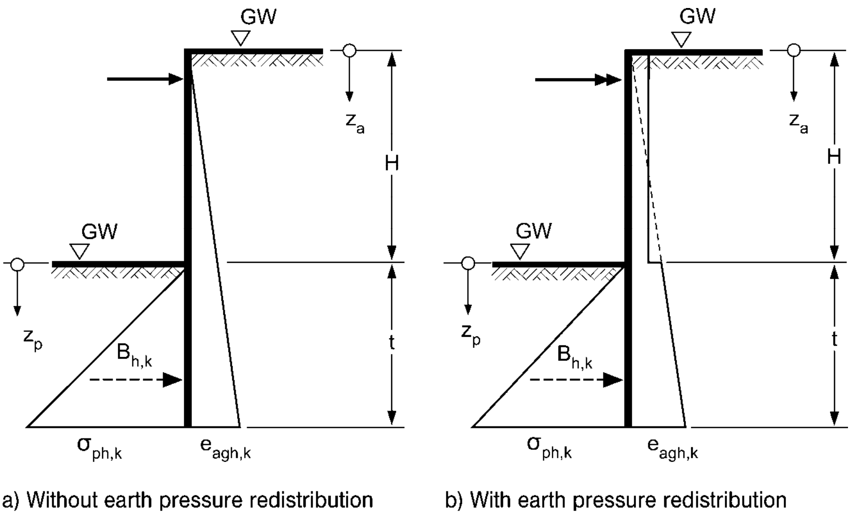

Figure R 96-1. Possible load diagrams for single-propped walls without support at the excavation level

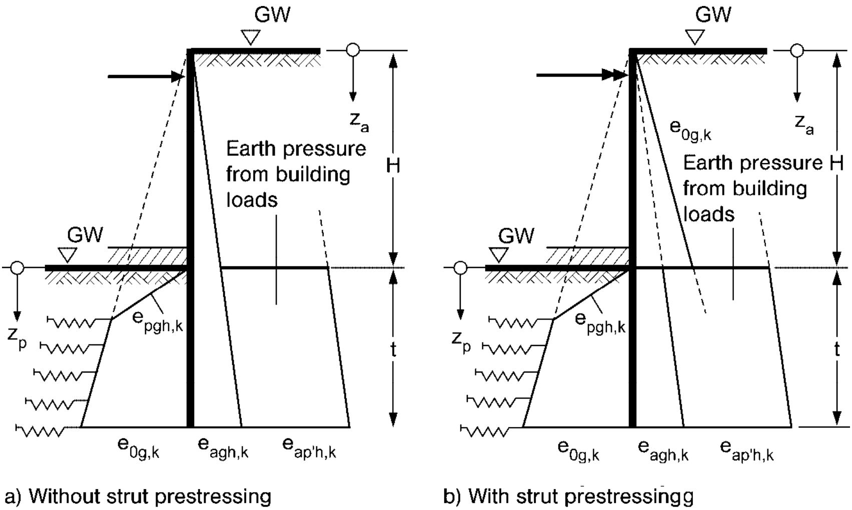

Figure R 96-2. Possible load diagrams for single-propped walls where a base slab is installed in stages at the excavation level

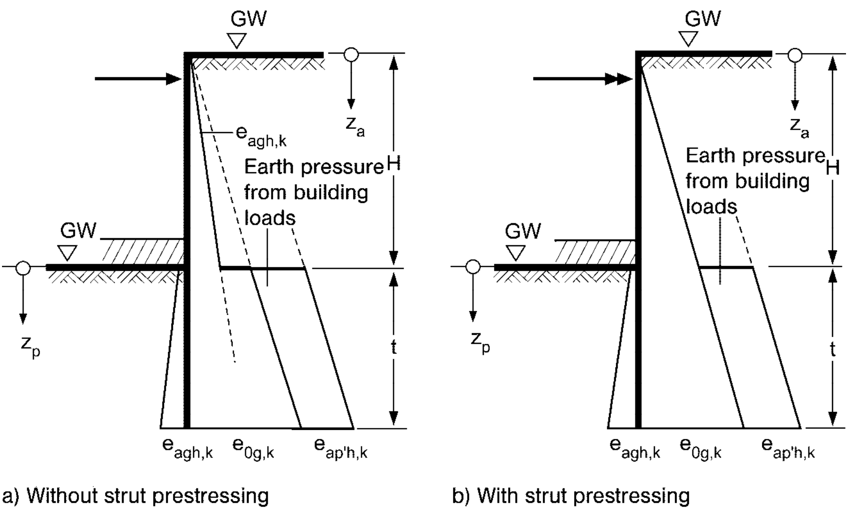

Figure R 96-3. Possible load diagrams for single-propped, low-deformation walls where a base slab is installed by jet grouting at the excavation level

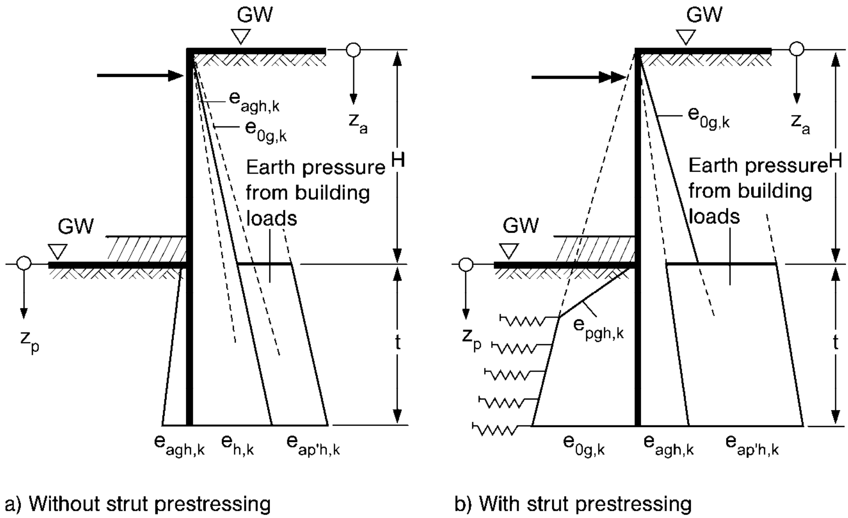

Figure R 96-4. Possible load diagrams for single-propped, flexible sheet pile walls where a base slab is installed by jet grouting at the excavation level

2. In construction stages without a stiffening base slab as shown in Figure R 96-1, equilibrium of horizontal forces can only be achieved if a ground reaction is mobilised in front of the wall toe. The ground reaction may be adopted as increasing linearly with depth, similar to the passive earth pressure. Assuming a buoyant soil, the passive earth pressure utilised as the reference value for the allowable ground reaction according to Paragraph 3 is obtained in the limit state from:

   $e_{pgh,k} = \gamma' \cdot K_{pgh} \cdot z_p.$

   According to R 95, Paragraph 6 b) (Section 12.6), the wall friction angle $\delta_{p,k} = -{}^1/_3 \cdot \varphi_k$ may be adopted in simplification instead of adhesion, if the special case shown in Figure R 99-3 (Section 12.10) does not apply.

   Otherwise, the following shall be observed:

   a) Due to the anticipated deflections of the top of unsupported walls, a fixed-earth support in soft soils is only possible for shallow excavation depths.
   b) Because of the large difference between the stiffness of a supported retaining wall and the stiffness of the ground, a fixed-earth support shall not be adopted in soft soils under any circumstances.

3. The following analyses shall be performed to guarantee adherence to the ultimate and serviceability limit states for the cases shown in Figure R 96-1:

   a) Taking the influence of anisotropy into consideration, see R 94, Paragraph 9 (Section 12.5), the following condition shall be met using the calibration factor $\eta_p \leq 0.50$:

      $B_{h,k} \leq E_{ph,k} \cdot \eta_p$.

   b) It shall be demonstrated that the design value of the reaction force is only as large as the design value of the passive earth pressure:

      $B_{h,d} \leq E_{ph,d}$;

      see R 80, Paragraph 5 b) (Section 4.3).

   c) Taking the serviceability state for limiting wall deflection into consideration, the following condition shall be met when using the calibration factor $\eta_p \leq 0.75$:

      $B_{h,k} \leq E_{0g,k} + (E_{ph,k} - E_{0g,k}) \cdot \eta_p$.

      The calibration factor shall be defined on-site based on local experience or on preliminary field tests such that the anticipated deflections of the wall in the embedment zone are acceptable. Expertise and experience in the geotechnical field are required. However, a reduction of the reaction force $B_{h,k}$ to a value corresponding to a coefficient of passive earth pressure of $K_{ph,mob} \leq 1.00$ is not expedient.

4. In construction stages where a stiffening base slab is installed in strips as shown in Figure R 96-2, equilibrium of forces is primarily ensured by the bearing capacity of the base slab. Otherwise, the following procedure may be used:

   a) Because the base slab is generally already installed before appreciable wall deflections occur at this depth, it may be assumed, in approximation, that the original at-rest earth pressure is largely retained below the base slab even after excavation is complete. Assuming an originally buoyant soil, it follows that:

      $e_{0g,k} = \gamma' \cdot K_0 \cdot (H + z_p)$.

      However, only the limit value of the passive earth pressure $e_{ph,k}$ determined using $\delta_{p,k} = 0$ may be effective in the zone directly below the base slab.

   b) If only the active earth pressure from soil self-weight is effective on the exterior of the wall below the excavation level as shown in Figures R 96-2 a) and R 96-2 b), the effective at-rest earth pressure determined according to Paragraph a) shall be reduced to a value equal to that which acts on the other side of the wall. If, on the other hand, the sum of the actions below the base slab is greater than the at-rest earth pressure determined according to Paragraph a), e.g. as a result of the impact of building loads or of water pressure, the ground reaction in excess of the at-rest earth pressure may be determined as follows:

- A subgrade reaction is adopted below the intersection of $e_{0g,k}$ and $e_{ph,k}$.
- The modulus of subgrade reaction is determined according to R 102 (Section 4.5).
- The characteristic ground reaction mobilised by the modulus of subgrade reaction shall meet the condition:

$$B_{Bh,k} \leq (E_{ph,k} - E_{V,k}) \cdot \eta_p.$$

Where:

$B_{Bh,k}$ the resultant of the mobilised characteristic subgrade reaction from the subgrade stress $\sigma_{h,k}$;
$E_{V,k}$ the characteristic resultant of the remaining at-rest earth pressure in the excavated state, taking the original preloading condition into consideration, see R 102, Paragraph 10 (Section 4.5).
η_p the calibration factor; here $\eta_p \leq 0.75$.

If the intersection of $e_{0g,k}$ and $e_{pgh,k}$ lies below the base of the wall, analysis using the modulus of subgrade reaction method is not possible, because the greatest possible ground reaction $e_{pgh,k}$ is already available to transfer reaction forces without noticeable displacement.

5. In construction stages where a stiffening base slab is installed using jet grouting techniques as shown in Figures R 96-3 and R 96-4, equilibrium of forces is primarily ensured by the bearing capacity of the base slab, similar to Paragraph 4. However, the following points shall also be observed:

a) Because the retaining wall moves towards the soil when installing the base slab, the ground below the base slab can relax so that only the active earth pressure is effective as shown in Figures R 96-3 and R 96-4 a). Assuming a buoyant soil and taking the surcharge p_k of the base slab into consideration, it follows that:

$$e_{ah,k} = \gamma' \cdot K_{agh} \cdot z_p + p_k \cdot K_{agh}.$$

b) The adoption of a ground reaction in excess of the at-rest earth pressure determined according to Paragraph 3 a), as shown in Figure R 96-4 b), or an at-rest earth pressure determined according to Paragraph 3 b), can only be justified if a flexible sheet pile wall is installed, the struts or anchors are heavily prestressed and the sum of the actions below the base slab is so great that the wall bends back towards the excavation, e.g. as a result of the effect of building loads or water pressure.

6. These stipulations apply to a homogeneous soil and a groundwater table at or below ground level. The following points shall be observed:
   a) These stipulations only apply for determination of the passive earth pressure below the groundwater table in conjunction with R 97 (Section 12.8).
   b) See R 99, Paragraph 6 (Section 12.10) for consideration of changes in stratified ground.

7. The approaches discussed are suitable for determination of the action effects, but not for determination of the required embedment depth below the stiffening base slab. The following points apply for analysis of sufficient embedment depth:
   a) In construction stages where a stiffening base slab is installed in strips in the course of excavation it may generally be assumed that the total length of the wall is sufficient, as obtained from the state prevalent before installing the stiffening concrete base according to Paragraph 3, in conjunction with R 98, Paragraph 2 (Section 12.9).
   b) In construction stages utilising soil stabilisation below the excavation level or a stiffening base slab installed by jet grouting, the necessary minimum embedment depth is obtained from analysis of the safety against base heave according to R 99, Paragraph 2 (Section 12.10), analysis of the safety against hydraulic heave according to R 99, Paragraph 3 or, if applicable, analysis of the safety against global failure according to R 99, Paragraph 4. It may be expedient to increase the embedment depth in individual cases, if this leads to a more favourable magnitude and distribution of the action effects.

12.8 Water pressure in soft soils (R 97)

1. If it is not possible, or no measures are taken, to dewater a deep, permeable layer, it shall be assumed that saturated, soft soil is buoyant and hydrostatic water pressure acts. If the wall is embedded in a load-bearing, permeable layer, the procedure is as follows:
   a) The water pressure on the outside of the wall as shown in Figure R 97-1 a) is obtained from:

   $$w_{a,k} = \gamma_w \cdot z_a$$

if the groundwater table is at ground level, or from:

$w_{a,k} = \gamma_w \cdot z'_a$

if the groundwater is below ground level.

b) The water pressure on the inside of the wall as shown in Figure R 97-1 a) is obtained from:

$w_{ü,k} = \gamma_w \cdot z_p$

if the groundwater table below the excavation level is not lowered.

c) The water pressure on both sides of the wall is determined separately and subsequently superimposed, so that only the positive water pressure $w_{ü,k}$ needs to be taken into consideration. Also see Figure R 97-1 b).

2. If the wall is not embedded in an impermeable layer, different approaches are adopted for the treatment of water pressure according to R 63 (Section 10.6):

a) In approximation, the simplified approach assumes that the wall is embedded in an impermeable, load-bearing layer. The true seepage around the wall is not considered. The governing approach is therefore that according to Paragraph 1 (Figure R 97-1).

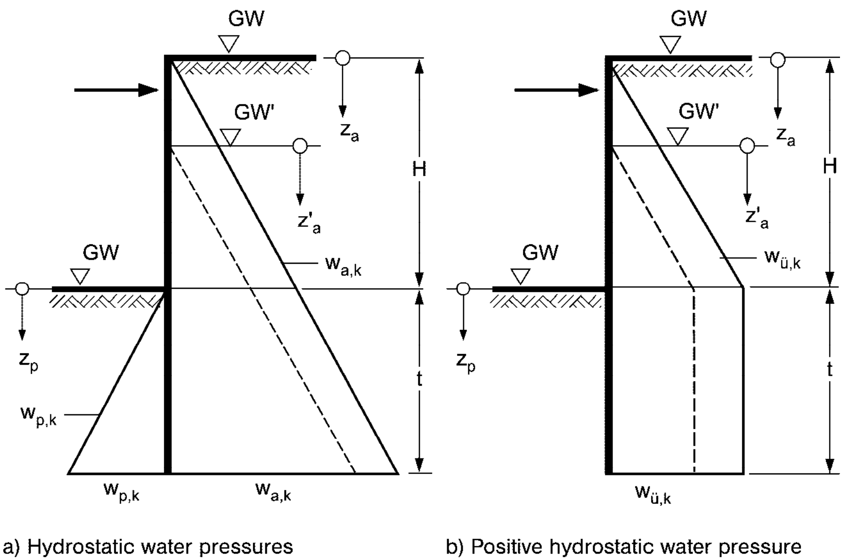

Figure R 97-1. Water pressure for a wall embedded in an impermeable layer

b) In more precise procedures the seepage around the wall is considered. Also see R 63, Paragraph 2 (Section 10.6). For a water level at ground level and one at the excavation level, the water pressure is obtained from:

$w_{a,k} = (\gamma_w - i_a \cdot \gamma_w) \cdot z_a$ on the outside and;

$w_{p,k} = (\gamma_w + i_p \cdot \gamma_w) \cdot z_p$ on the inside of the wall.

The two components are superimposed so that only the positive water pressure $w_{p,k}$ is taken into consideration for further analysis. Also see Figure R 97-2 b).

c) The procedures discussed in R 59 (Section 10.2) for determination of the flow pressure provide differing results depending on the simplification they are based on. The simplified case is shown in Figure R 97-2:

$i = i_m = i_a = i_p = \Delta h/l = H/(H + 2 \cdot t)$.

Also see R 63, Paragraph 2 (Section 10.6).

3. The following stipulations apply for determination of the earth pressure below the groundwater table when the water pressure is taken into consideration:

a) In the approximation solution according to Paragraph 2 a) the active earth pressure and the at-rest earth pressure are determined using the unit weight γ' according to R 95 (Section 12.6).

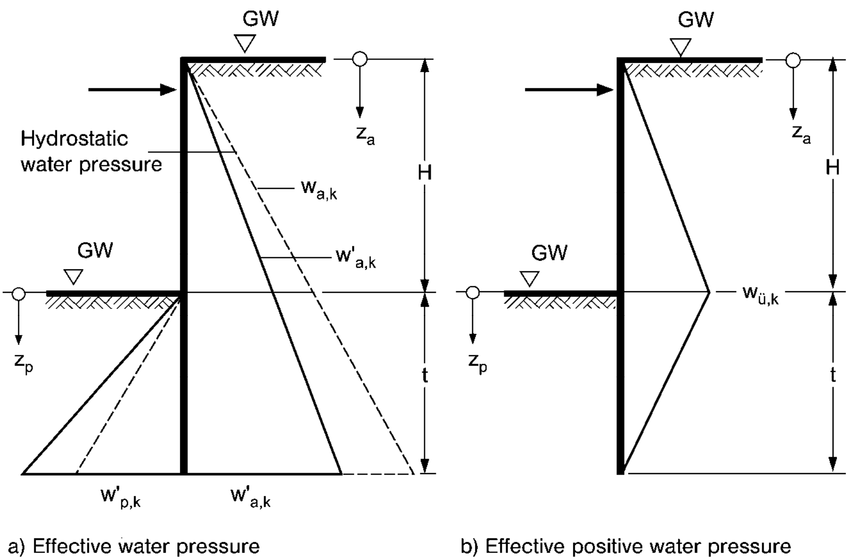

Figure R 97-2. Water pressure with seepage around the wall toe (simplified representation)

b) In the more precise analysis according to Paragraph 2 b) the effective unit weights deviate from R 95 (Section 12.6) and:

$\gamma'_a = \gamma' + i_a \cdot \gamma_w$ on the outside, where the governing flow is from top to bottom of the wall:

$\gamma'_p = \gamma' - i_p \cdot \gamma_w$ on the inside, where the governing flow is from bottom to top

of the wall.

Similar approaches are also discussed in the following two Paragraphs for the passive earth pressure.

4. If the simplified water pressure approach without consideration of seepage according to Paragraph 2 a) is adopted, the following analysis approaches apply for the ground reactions on the inside of the wall:

a) Construction stages without a stiffening base slab:

$\sigma_{ph,k} = (\gamma' \cdot K_{pgh} \cdot z_p) \cdot \eta_{eff}$ as shown in Figure R 97-3 a).

The effective calibration factor η_{eff} represents a substitute for determination of the mobilised characteristic ground reaction according to R 96, Paragraph 3 (Section 12.7).

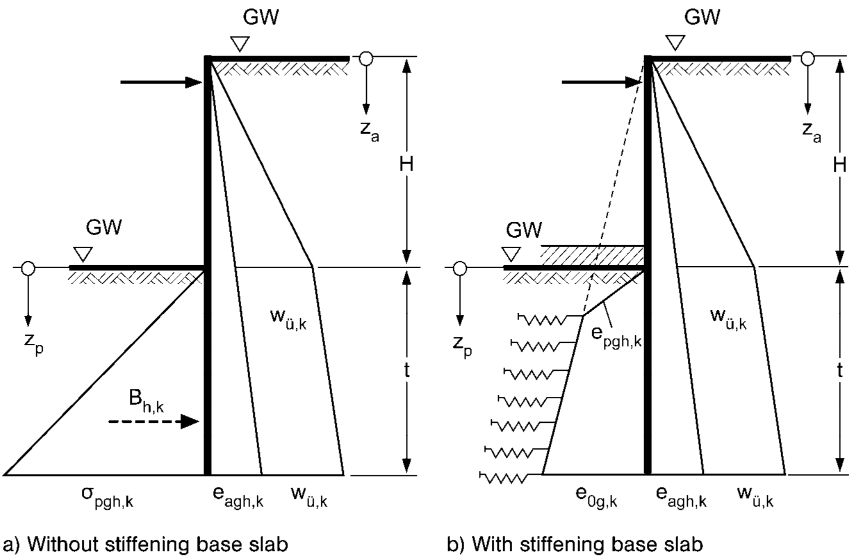

Figure R 97-3. Load diagrams for single-propped walls with positive hydrostatic water pressure

b) Construction stages where a base slab is installed in strips or a stabilised soil layer is installed below the excavation level:

$e_{0g,k} = \gamma' \cdot K_0 \cdot (H + z_p)$, but a maximum of

$e_{pgh,k} = \gamma' \cdot K_{pgh} \cdot z_p$ as shown in Figure R 97-3 b)

and not greater than the total characteristic load from earth pressure and water pressure on the outside of the wall below the excavation level. If the total load is greater, a subgrade reaction according to R 96, Paragraph 4 b) (Section 12.7) may additionally be adopted.

c) Construction stages where a base slab is installed by jet grouting:

$e_{ah,k} = \gamma' \cdot K_{agh} \cdot z_p + p_k \cdot K_{agh}$.

for a low-deformation wall; otherwise, R 96, Paragraph 5 b) (Section 12.7) applies.

5. If a more precise water pressure approach including consideration of seepage according to Paragraph 2 b) is adopted, the following analysis approaches apply for the passive earth pressure or the at-rest earth pressure on the inside of the wall:

a) Construction stages without a stiffening base slab as shown in Figure R 96-1:

$\sigma_{ph,k} = (\gamma' \cdot K_{pgh} \cdot z_p) \cdot \eta_{eff}$

and as shown in Figure R 97-3 a). The effective calibration factor η_{eff} represents a substitute for determination of the mobilised characteristic ground reaction according to R 96, Paragraph 3 (Section 12.7).

b) Construction stages where a base slab is installed in strips or a stabilised soil layer is installed below the excavation level as shown in Figure R 96-2:

$e_{0g,k} = \gamma'_p \cdot K_0 \cdot (H + z_p)$, but a maximum of

$e_{pgh,k} = \gamma'_p \cdot K_{pgh} \cdot z_p$ as shown in Figure R 97-4 b)

and not greater than the total load from earth pressure and water pressure on the outside of the wall below the excavation level. If the total load is greater, e.g. as the result of additional earth pressure from a building load, a subgrade reaction according to R 96, Paragraph 4 b) (Section 12.7) may be additionally adopted.

c) Construction stages where a base slab is installed by jet grouting as shown in Figure R 96-3:

$e_{ah,k} = \gamma'_p \cdot K_{agh} \cdot z_p + p_k \cdot K_{agh}$

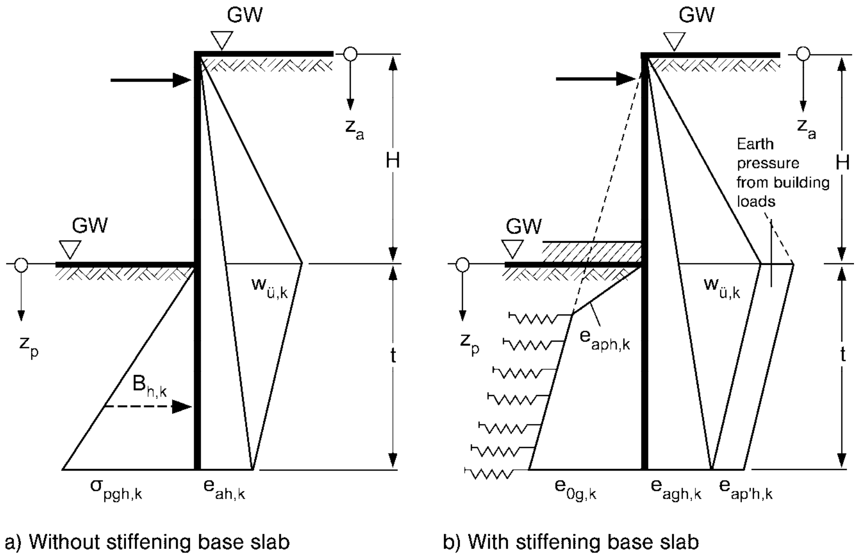

Figure R 97-4. Load models for single-propped walls when adopting flow pressure (simplified representation)

for a low-deformation wall; otherwise, R 96, Paragraph 5 b) (Section 12.7) applies.

The drainage layer, which is usually required in case of seepage at the excavation level, is not shown in Figures R 97-2 and R 97-4.

6. If the settlements caused by a fill or a building foundation adjacent to the planned excavation are not complete and porewater pressure therefore acts, the hydrostatic water pressure shall be increased by the value of the porewater pressure over the complete effective height. In addition, a natural excess porewater pressure may also occur, e.g. from extensive bands of sand, or of artesian origin.

7. In Paragraphs 2 to 5 and in the corresponding Figures R 97-2 to R 97-4 it has been assumed in simplification that the groundwater table is at ground level. This is generally not the case. In addition, the effect of ring drainage as recommended in R 100, Paragraph 2 (Section 12.11) shall be considered. The ordinates of the earth pressure, at-rest earth pressure and water pressure shown in Figures R 97-3 and R 97-4 change accordingly.

12.9 Determination of embedment depths and action effects for excavations in soft soils (R 98)

1. All construction stages occurring when excavating and backfilling the excavation shall be investigated according to R 11, Paragraph 1 (Section 4.2). The following shall be observed:

   – R 95 (Section 12.6) for determination of earth pressure;
   – R 97 (Section 12.8) for determination of water pressure;
   – R 96 (Section 12.7) for adopting the ground reactions.
   In contrast to R 11, Paragraph 2 (Section 4.2), the computed deformations of the retaining wall at intermediate stages and their impacts, in the form of support point displacements at the height of the subsequent support in the following construction stage, shall generally be taken into consideration due to their great influence on the action effects.

2. The following points apply for analysis of the construction stages that are both locally and temporally limited according to R 93, Paragraphs 3 and 4 (Section 12.4):

   a) Two conditions shall be investigated:

      – the condition in which the first trench is sloped on both sides;
      – the condition in which the excavated strip is bounded by blinding concrete on one side and sloped on the other.

   b) For analysis:

      – a temporary arching effect in the soil;
      – the load distribution by the head beam according to R 93, Paragraph 2 (Section 12.4) and;
      – the load-bearing effect of parts of the retaining wall already supported by a strip of blinding concrete;

      may be taken into consideration.

      The deflection of the wall during excavation shall also be monitored, in addition to this analysis. If the results are unsatisfactory, the originally selected trench width shall be reduced or one of the construction procedures discussed in R 93, Paragraph 3 c) (Section 12.4) employed.

3. The analysis according to Paragraph 2 may be dispensed with if the following procedure is adhered to:

   a) The retaining wall shall have a minimum embedment depth obtained from:

      – the stability analysis assuming an equivalent level according to Paragraph 4;

- the analysis of base heave safety according to R 99, Paragraph 2 (Section 12.10), without the subsequent blinding concrete surcharge;
- the analysis of safety against hydraulic heave according to R 99, Paragraph 3 and, if applicable;
- the analysis of safety against global failure according to R 99, Paragraph 4.

b) Work shall commence at a non-critical location. A small trench width shall initially be selected and can then be optimised in the course of work based on the results of monitoring and measurements.
c) The deflections, settlements and heave of the wall and its surroundings shall be carefully monitored whilst manufacturing the blinding concrete strip or installing additional bracing.
d) If it is not possible to install the blinding concrete strip or other additional bracing on the basis of daily capacity according to R 93, Paragraphs 3 and 4 (Section 12.4), due to unforeseen circumstances, a condition that is mathematically demonstrated as being safe shall be achieved by other means before work ends, e.g. by reinstating the condition prevalent before commencement of the daily capacity.

If the results are unsatisfactory, one of the construction methods discussed in R 93, Paragraph 3 c) (Section 12.4) shall be employed.

4. Regardless of whether stability is demonstrated according to Paragraph 2 or Paragraph 3 for the short-term construction condition before installing a blinding concrete strip, the construction condition after excavation of the respective first trench as shown in Figure R 93-1 or Figure R 93-2 (Section 12.4) shall be analysed for a computed equivalent level located at two thirds of the depth of the intended trench depth as shown in Figure R 98-1. In this manner, determination of the necessary embedment depth takes into consideration that:

   - on the one hand, the first trench or its strip-wise extension has the full excavation depth;

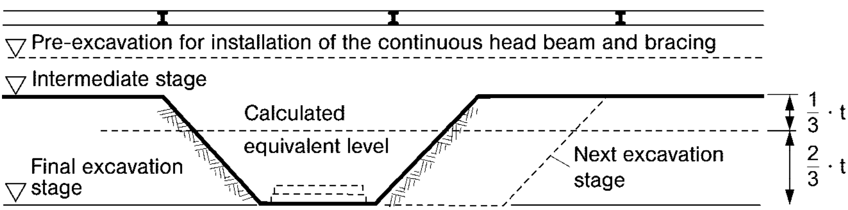

Figure R 98-1. Equivalent excavation level for the intermediate stage with trench

– on the other hand, that lateral regions exist that are either still supported by soil or already supported by the stiffening blinding concrete.

The groundwater level within the excavation shall be adopted at the planned final excavation level for the purpose of this analysis.

5. For a single-propped wall with a free-earth support, both the classical earth pressure distribution and an earth pressure redistribution can govern according to R 95, Paragraph 7 (Section 12.6). If there is any doubt whether analysis shall be performed with or without earth pressure redistribution, both cases shall be investigated. However, the additional determination of action effects and embedment depths with redistributed earth pressure as shown in Figure R 96-1 (Section 12.7) may generally be dispensed with if the reaction force determined for the row of struts or anchors is increased by 30 %.
6. See R 99 (Section 12.10) for further stability analyses, in particular for base heave, hydraulic heave and global failure, as well as the additional investigations for stratified ground.

12.10 Additional stability analyses for excavations in soft soils (R 99)

1. The most unfavourable case is dealt with in Recommendations R 95 to R 98 assuming soft soil from ground level to the base of the wall or deeper as shown in Figure R 99-1 a). More favourable conditions are prevalent if load-bearing soil is present in the upper layer and soft soil only deeper, as shown in Figure R 99-1 b). Even more favourable conditions are prevalent if only soft soil is present in the upper layer and load-bearing soil deeper, as shown in Figure R 99-1 c). The change in layers may either be at the excavation level, or higher, or lower. Additional stability analyses to those described in R 98 (Section 12.9) are required in a number of the cases mentioned. Also see Paragraphs 2 to 6.

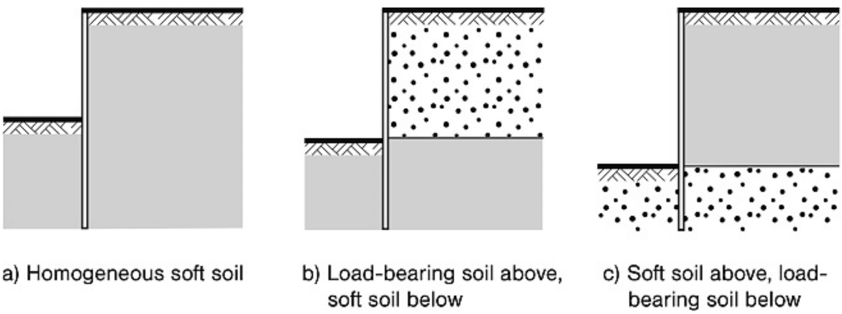

Figure R 99-1. Excavations in stratified ground (without representation of supports)

2. Excavations in homogeneous soft soil as shown in Figure R 99-1 a) are seriously threatened by base heave, also see R 10, Paragraph 1 (Section 4.9). The same applies to a lesser extent for excavations in stratified ground as shown in Figure R 99-1 b), see [52] and [117]. Analysis of base heave at the excavation level is generally performed using the undrained shear strength $c_{u,k}$ of the soil. The following individual points shall be observed:

   a) For excavations of depth H and width $B > 0.20 \cdot H$ in homogeneous, saturated soil as shown in Figure R 99-2, the limit state condition analogous to R 10, Paragraph 1 (Section 4.9):

   $$G_{B,d} + G_d + Q_d \leq R_{n,d} + T_d$$

   shall be adhered to [130]. The partial safety factors γ_G for permanent and γ_Q for variable actions shall be taken into consideration for the soil self-weight $G_{B,k}$ and the surcharge G_k or Q_{Rep}, where unbounded distributed loads at $p_k \leq 10$ kN/m^2 are dealt with as permanent actions.

   b) The design value of the vertical resistance from cohesion is obtained from:

   $$T_d = \frac{c_{u,k} \cdot \left(H + t_g\right)}{\gamma_{R,v}}$$

   and the design value of the bearing capacity from:

   $$R_{n,d} = \frac{b_g \cdot \left(\gamma \cdot t_g + 5{,}14 \cdot c_{u,k}\right)}{\gamma_{R,v}}$$

   The unit weight γ above the groundwater table is adopted at γ and at γ_r below the groundwater table.

   c) If effective stresses are adopted to calculate $G_{B,d}$ and $R_{n,d}$, i.e. the effective unit weight γ' for the buoyant soil, the differential hydrostatic water pressure shall also be taken into consideration. Alternatively, an analysis method taking flow into consideration in conjunction with the effective unit weight according to R 59 (Section 10.2) may also be adopted.

   d) The width b_g is obtained as follows:

   – The governing width $b_g = B$ is obtained without lateral surcharges if the undrained shear strength of the soil $c_{u,k}$ with depth is known.
   – For lateral surcharges and variable shear strength $c_{u,k}$ the width shall be varied in order to identify the maximum utilisation factor [130].

   The base heave hazard is reduced where excavations have a width $B \leq 0.20 \cdot H$. Also see [52] and [117].

e) Because of the anisotropy of the soil as a consequence of sedimentation and the rotation of the principal stress directions as a consequence of soil excavation, the undrained shear strength $c_{u,k}$ of the soil shall normally be increased when determining the earth pressure and reduced when determining the bearing capacity [105, 113]. Because this can only be estimated with difficulty, but both effects partly cancel each other out, it is recommended to disregard it according to common practice.

f) Where excavation bases are anchored the limit state condition according to Paragraph 2 a) is expanded for analysis of heave safety by the design value of an equivalent surcharge Z_d imposed by the tension piles in the region of the heave body as shown in Figure R 99-2 b):

$G_{B,d} + G_d + Q_d \leq R_{n,d} + T_d + Z_d$.

The design value Z_d is obtained from:

$Z_d = \Sigma\, Z_{i,k}/\gamma_{R,v}$

as shown in Figure R 99-2b).

In design for the equivalent surcharge imposed by the n tension piles, the limit state condition:

$\Sigma\, Z_{i,k} \cdot \gamma_G \leq (\Sigma\, R_{t,i,k})/\gamma_{s,t}$

shall be met. Only the pile skin friction areas below the heave mass may be adopted to determine the characteristic tension pile resistances $R_{t,i,k}$.

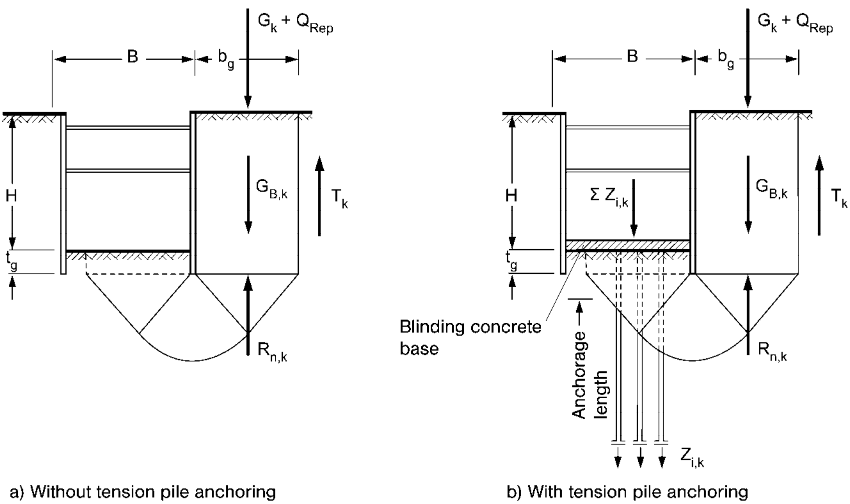

Figure R 99-2. Excavation base heave

3. For high groundwater levels in particular, excavations in stratified ground as shown in Figure R 99-1 b) are very seriously threatened by the possibility of hydraulic heave. The same applies to a lesser extent for excavations in homogeneous soil as shown in Figure R 99-1 a). Also see R 61 (Section 10.4).

4. An analysis of global stability shall be performed for excavations in homogeneous soft soil as shown in Figure R 99-1 a) and excavations in stratified soil as shown in Figure R 99-1 b). The following points shall be observed:

   a) In particular, slip surfaces that terminate within the excavation as shown in Figures R 101-1 b) and R 101-1 c) (Section 12.12) shall be investigated for braced retaining walls.
   b) With regard to serviceability, lower utilisation factors shall be adopted in soft soils than in load-bearing soil types. Also see R 91, Paragraph 5 (Section 12.2). However, the lower utilisation factors are not required for the load-bearing soil layers involved in a slip mechanism.
   c) The normal force and the shear resistance of a stiffening base slab may be taken into consideration favourably in the analysis.

5. Deep-seated stability is analysed according to R 44 (Section 7.3) for anchored retaining walls. The following points shall be observed:

   a) The starting point of the lower failure plane is generally the toe of the retaining wall.
   b) In excavations with a change in soil layering at the excavation level as shown in Figure R 99-1 b), the anchored block shall generally be supported by a low set of struts, blinding concrete installed in stages or by a base slab installed by jet grouting.
   c) In excavations with alternating layers of soft and load-bearing soils, a lower failure plane may develop, the course of which is not a straight line from the centre of gravity of the grouted section to the wall toe, but instead is interrupted by a lengthy horizontal slip plane in one of the soft layers.
   d) If the soil layer below the wall toe is soft, the origin of the governing lower failure plane may be located below the toe of the wall.

6. In excavations in which:

   – the soft layer as shown in Figure R 99-3 is below the wall toe and therefore;
   – the development of a fixed-end support in the cover layer is possible;

   the following points shall be observed:

   a) When determining the active earth pressure the wall friction angle shall be adopted at $\delta_{a,k} = 0$, because transfer of the vertical loads to the ground is not guaranteed.

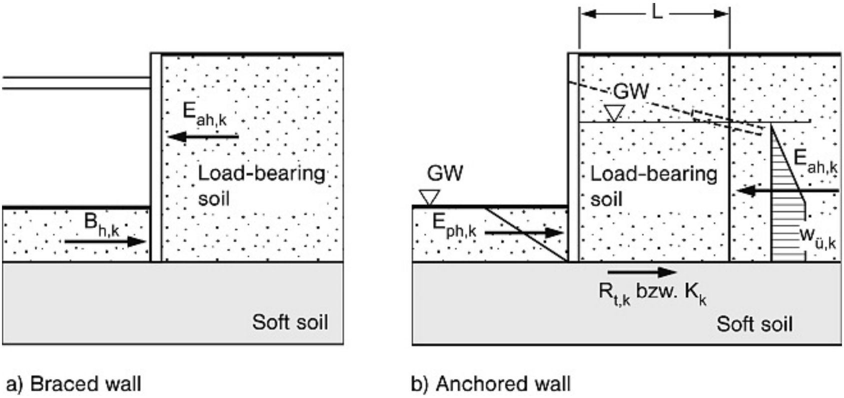

Figure R 99-3. Excavation with soft soil below the wall toe

b) In braced excavations as shown in Figure R 99-3 a), sufficient embedment depth in the load-bearing cover layer shall be demonstrated. The wall friction angle shall be adopted at $\delta_{p,k} = 0$ to ensure that no computed slip surface through the soft soil becomes governing. It shall be demonstrated that:

$B_{h,d} = E_{ph,d}$.

c) Sufficient sliding safety shall be demonstrated for anchored retaining walls as shown in Figure R 99-3 b):

$E_{ah,d} + W_{ü,d} \leq E_{ph,d} + R_{t,d}$ or K_d.

A utilisation factor $\mu < 1.0$ may be necessary in order to limit the anticipated deflections. The passive earth pressure in the cover layer is determined using the wall friction angle $\delta_{p,k} = 0$. For the sliding resistance either:

$R_{t,k} = G_k \cdot \tan \varphi_k$

where $\varphi_k = \varphi'_{s,k}$ or $\varphi_k = \text{equiv. } \varphi_{s,k}$ according to R 95, Paragraph 3 (Section 12.6) or

$K_k = c_{u,k} \cdot L$.

The smaller value governs analysis.

12.11 Water management for excavations in soft soils (R 100)

1. Substantial settlements are anticipated in soft soils if the groundwater is lowered far enough that the buoyant effect is lost and the weight of the saturated soil can act. Lowering or relief of the groundwater table is therefore

only permissible within strict limits. Extensive sand banding shall be taken into consideration.

2. The groundwater table is generally subject to seasonal fluctuations. Because of the extremely unfavourable influence of water pressure on determining the embedment depth and action effects of the retaining wall, it is recommended to lower the groundwater table to the lowest known previous level by arranging a ring drainage system around the outside of the excavation. It may generally be assumed that the soil is consolidated at this level.

3. It is generally permissible to dewater intercalated bands of fine-sand or coarse silt, or to lower an existing confined groundwater table within an excavation lined according to R 92, Paragraph 1 (Section 12.3). The wells shall terminate above the toe of the retaining wall in order to limit the effects of dewatering outside the excavation. Vacuum filter wells shall be employed if gravity dewatering is insufficient or if additional densification is aimed for.

4. The localised use of vacuum lances for stabilising slopes, e.g. when manufacturing trenches for installing blinding concrete strips according to R 93, Paragraph 3 or Paragraph 4 (Section 12.4), generally presents no problems with regard to neighbouring structures.

5. Residual perched water and surface water shall be collected in filter-stable surface drains according to DIN 4095 at all times and sent to pump sumps. The pump sumps shall be operated long enough to rule out flooding of the base of the building.

6. The effects of dewatering measures inside and outside of the excavation shall be constantly monitored.

12.12 Serviceability of excavation structures in soft soils (R 101)

1. The serviceability of excavation structures depends on:

   – accurate investigation and assessment of the given situation;
   – selection of a suitable wall and base slab;
   – selection of a suitable construction method;
   – realistic analysis and design approaches;
   – technically correct implementation and monitoring of construction work.

   If deficits in only one of these points occur it shall be assumed that grave impacts on the surroundings will result and may extend much further than the depth of the excavation, in contrast to excavations in load-bearing ground. The following points shall be observed in addition to the stipulations in the previous Recommendations.

2. The demands on the serviceability of excavations in soft soils shall be defined with the construction of the structure within the excavation and the effects on the surroundings in mind:

   a) If it is certain that no structures in the vicinity of the excavation are affected, it is sufficient to draft the design according to R 95 to R 99 (Sections 12.6 to 12.10) and to guarantee, by implementing a working space or by selecting a sufficiently large degree of tolerance when installing the retaining wall in excavations without a working space, that large ground movements do not compromise the serviceability of the retaining wall.
   b) In excavations in the vicinity of settlement- and deformation-sensitive structures, it is extremely important to limit ground movements in addition to wall deformations. In soft soils the only option is to maintain, as far as possible, the primary stress state of the ground. It is critically important to limit relaxation and loosening of the soft soil below the excavation level.

3. The most obvious measures for maintaining the primary stress state are selection of a stiff retaining wall and implementation of stiff supports at ground level. In addition, stiff support of the wall toe and measures to prevent base heave may be considered. In principle, the following impacts can be anticipated:

   a) In large excavations with a load-bearing cover layer extending to the wall toe as shown in Figure R 101-1 a) there is a hazard of the toe support in the cover layer slipping on the soft soil and the excavation level being subject to strong heave, leading to extensive relief of the ground behind the retaining wall and thus to settlement and deformation. This is not substantially influenced by using struts instead of tie-back anchors.
   b) If an additional stiffening base slab is installed as shown in Figure R 101-1 b) the toe support remains largely free from deformation, but even with a mathematically adequate safety against base heave it is possible for the excavation floor to heave, leading to settlement behind the retaining wall.
   c) If additional stabilisation of the excavation level against heave is implemented using ballast, floor doming, or tension piles or anchors, to a sufficient depth as shown in Figure R 101-1 c), base heave can be prevented to the extent that settlement behind the retaining wall is greatly reduced or unloading heave occurs.

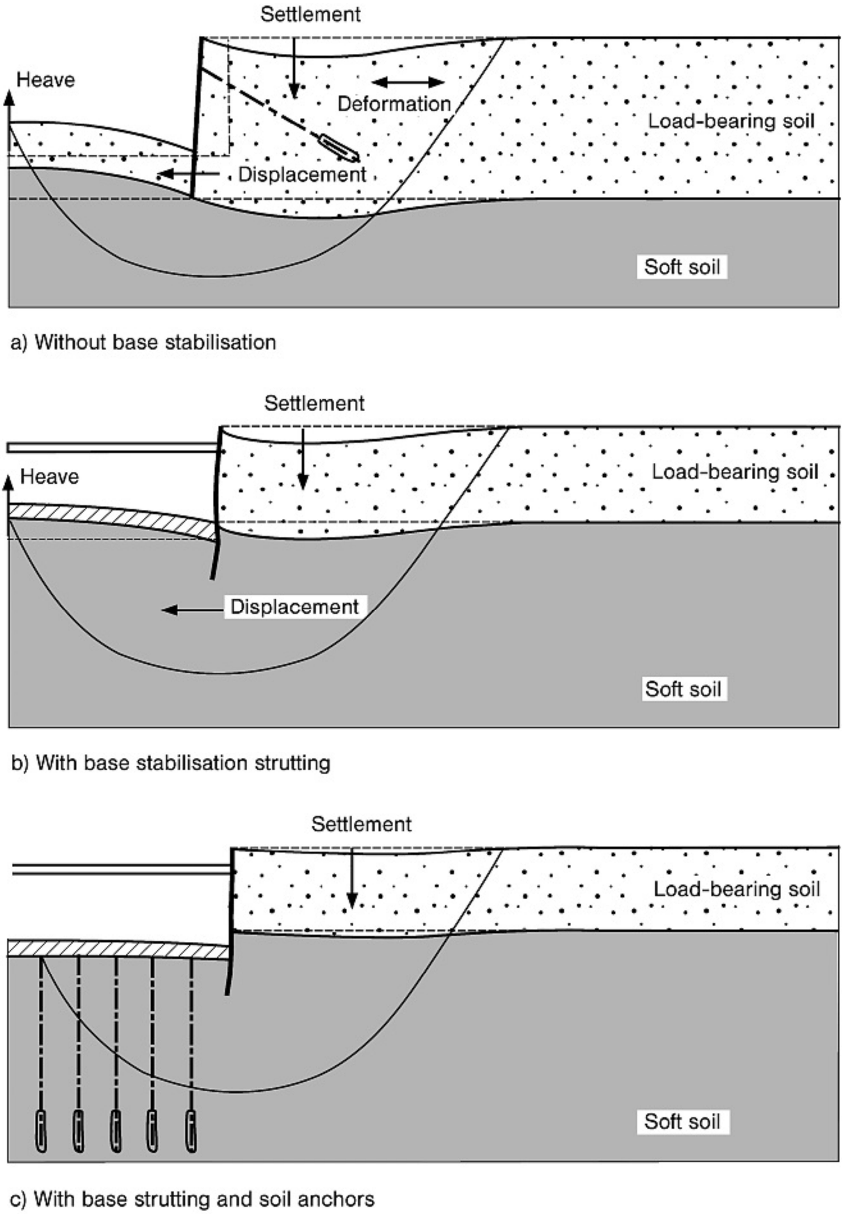

Figure R 101-1. Ground movements as a function of base slab stabilisation

The favourable impact of a stiffening base slab or ground anchors is increased if they are installed before excavation begins instead of in stages after reaching the excavation level.

Three cases with varying depth of the layer boundary between load-bearing soil and soft soil are shown in Figure R 99-1 (Section 12.10), representing increasing demands on stabilisation measures.

4. Conservation of evidence measures shall be carried out on existing structures at an adequate radius around the planned excavation before starting construction work, and the groundwater level recorded. All subsequent work phases impacting the soft soil shall be accompanied by local settlement measurements at sufficiently short intervals during the course of work. It is necessary to repeatedly inspect the structures in the immediate vicinity of the respective work areas while installing retaining walls and during excavation work. As soon as critical settlements occur in the vicinity, or wall deformation or cracks in neighbouring structures are perceptible to the naked eye, excavation work shall cease and, for advanced excavations, supporting berms tipped or the excavation partly backfilled until the settlement process ceases.

5. It is only possible to construct excavations in soft soil without settlement impacts on neighbouring buildings in conjunction with highly favourable boundary conditions. Deformation resulting from the excavation process is generally unavoidable, in particular with increasing excavation depth. These deformations cannot be determined with sufficient precision using classical analysis methods. In contrast, numerical methods, e.g. based on finite element methods (FEM) according to R 103 (Section 4.6), can provide approximately correct deformation figures when realistic material properties are adopted. If possible, the FEM model employed shall be calibrated using measurement results taken from an excavation in similar ground conditions. The use of FEM analyses is particularly valuable if the deformation-reducing effect of any additional support measures needs to be visualised. The plausibility of the adopted earth pressure distribution or earth pressure redistribution can also be visualised in an FEM analysis.

13 Analysis of the bearing capacity of structural elements

13.1 Material parameters and partial safety factors for structural element resistances (R 88)

1. The material parameters and partial safety factors for structural element resistances in the ultimate limit states STR and GEO 2 are given in:

   – EN 1992-1-1 and the corresponding NA for concrete or reinforced concrete structural elements;
   – EN 1993-1-1 and the corresponding NA for steel structural elements;
   – EN 1995-1-1 and the corresponding NA for timber structural elements.

2. Note the following points on adopting the partial safety factors given in the regulations in Paragraph 1:

   a) See R 24 (Section 2.1) and R 79 (Section 2.4) for definitions of design situations.
   b) Because the regulations discussed in Paragraph 1 do not differentiate between permanent and temporary structures or between permanent and temporary situations, the partial safety factors used also apply to design situations DS-T, DS-T/A and DS-A, if not otherwise stated in individual cases.
   c) The partial safety factors for design situation DS-P are reproduced in Tables 6.1 and 6.2, Appendix A 6, but are given in brackets because they do not generally govern retaining walls according to R 79, Paragraph 1 (Section 2.4).

3. The material parameters and partial safety factors given in EN 1992-1-1 for concrete or reinforced concrete structural elements are summarised in Appendix A 7.

4. The material parameters and partial safety factors given in EN 1993-1-1 for steel structural elements are summarised in Appendix A 8. Note the following individual points:

   a) The numerical value of the shear strength has been included in the table of material parameters.
   b) The data for sheet pile wall steel is taken from EN 1993-3.
   c) Any weakening of steel sections caused by drilling, transverse welding or significant corrosion shall be taken into consideration when analysing bearing capacity.

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   d) See EN 1993-1-8 for loads and allowable loads on connections.
   e) The full table values may be used when stipulating the stiffnesses of the steel sections.

5. The material parameters and partial safety factors given in EN 338 for timber structural elements are summarised in Appendix A 9. Note the following individual points:
   a) Quality classes C 24 and C 30 correspond approximately to the previous quality classes S 10/MS 10, GK II, S 13 and GK I.
   b) The given material parameters and partial safety factors assume that new or practically new timber is used.
   c) The modification factor for taking into consideration the utilisation class and load action duration class for solid timber may be adopted at $k_{mod.} = 1.00$.

13.2 Bearing capacity of soldier pile infilling (R 47)

1. The safety against structural failure of soldier pile wall infilling according to the limit state condition:

   $E_d \leq R_d$

   shall be analysed for the design action effects determined according to Section 5. The design value E_d consists of the loads imposed by the most unfavourable combination of action effects, the design value R_d of the resistance of the structural element. The individual analyses depend on the material used.

   a) In the case of analysis of the normal bending stresses of timber planks with uniaxial bending according to Paragraph 5 the general design equation obtained from

   $$E_d = \sigma_{m,d} = \frac{M_d}{W_{y,n}} \quad \text{and} \quad R_d = f_{m,d} = \frac{k_{mod} \cdot f_{m,k}}{\gamma_M}$$

   is:

   $\sigma_{m,d} \leq f_{m,d}$.

   Where:

   $f_{m,k}$ the characteristic bending strength according to Appendix A 9;
   γ_M the partial safety factor according to Appendix A 9;
   $k_{mod.}$ the modification coefficient, here $k_{mod.} = 1.00$ according to R 88, Paragraph 5 (Section 13.1);
   M_d the design moment according to Paragraphs 2 to 4;
   $W_{y,n}$ the net resisting moment.

b) For steel infilling see R 48 (Section 13.3) and R 49 (Section 13.4).
c) For reinforced concrete infilling see R 50 (Section 13.5).

2. The governing characteristic earth pressure for determining the bending stresses is obtained as follows:

a) When adopting active earth pressure from soil self-weight, unbounded distributed load $p_k \leq 10$ kN/m^2 and, if applicable, cohesion according to R 4 (Section 3.2), the pressure diagram according to R 69 (Section 5.2) used to determine the action effects in the soldier piles governs. If a triangular or classical earth pressure distribution is selected, either the tip, as shown in Figure R 47-1 a), or the maximum value, as shown in Figure R 47-l b), may be truncated. However, the remaining earth pressure ordinate shall equal at least two thirds of the original.
b) If a building load acts in addition to the actions given in Paragraph a) the pressure diagram obtained when adopting the active earth pressure according to R 28 (Section 9.3) or R 29 (Section 9.4), or when adopting the increased active earth pressure according to R 22 (Section 9.5), governs.
c) If live loads greater than $p_k = 10$ kN/m^2 act in addition to the actions given in Paragraph a) and, if applicable, Paragraph b), the characteristic earth pressure from live loads may be superimposed on the pressure diagram according to Paragraph a) or Paragraph b), such that a uniform load develops in the load distribution boundary zones according to R 7, Paragraph 2 (Section 3.5) or R 28, Paragraph 3 (Section 9.3), as shown in Figure R 47-2.

The thickness of the infilling may be adjusted to fit the respective diagram.

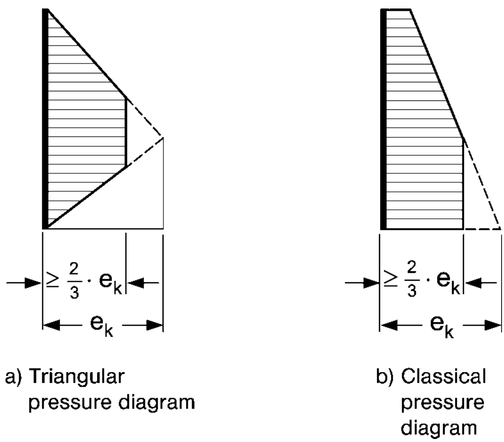

Figure R 47-1. Reduction of earth pressure from soil self-weight, unbounded uniform load and, if applicable, cohesion when designing infilling

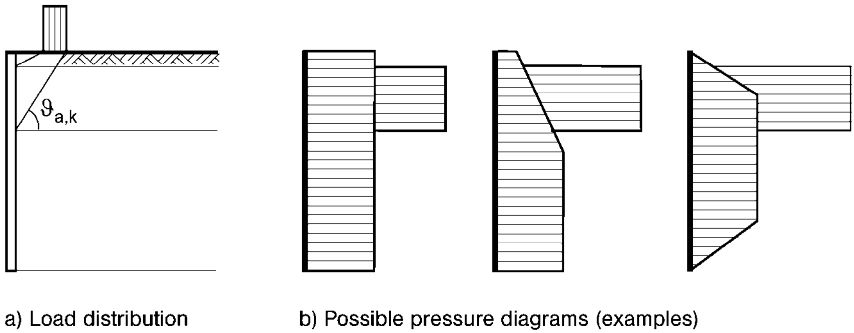

Figure R 47-2. Earth pressure from live loads for infill wall design

3. The characteristic earth pressure shall generally be adopted from soldier pile to soldier pile as a uniform load. The impact of the arching effect of the ground between soldier piles, and the resulting decrease in the loads on the infilling in mid-span, may be taken into consideration if:

   – either medium-dense or dense, cohesionless soil or at least stiff, cohesive soil is present;
   – the soldier piles are driven or vibrated or, in the case of soldier piles set into boreholes, the backfill material is compacted in such a way that a tight bond is developed between soldier piles and the native soil and;
   – the infilling is installed behind the flanges on the excavation side, without pre-bending.

   Also see [53].

   When using triangular and classical pressure diagrams the impact of the arching effect may be related to the remaining earth pressure ordinate as shown in Figure R 47-1. If the arching effect is not taken into consideration during analysis the bending moment determined using the uniform load may be reduced by 20 %.

   In order to take the arching effect into consideration when designing the infilling an earth pressure redistribution consisting of two triangles with the zero ordinate in the centre of the infilling may be adopted.

4. In general, the vertical earth pressure component may be disregarded when designing infilling composed of individual elements, e.g. timber planks, prefabricated reinforced concrete elements or trench sheet piles. However, this does not apply:

   a) if the impact of the arching effect is taken into consideration according to Paragraph 3 or;

b) if individual infill elements are arranged vertically, supported by nogging pieces.

It may be necessary to consider the vertical earth pressure component obtained from the horizontal component multiplied by the tangent of the characteristic wall friction angle $\delta_{a,k}$, determined according to Paragraph 2 or Paragraph 3.

5. The following routes may be taken to determine the design loads:

   a) In approximation, the greatest load ordinate determined according to Paragraph 2 or Paragraph 3 is divided into one component from permanent actions and one from variable actions. The characteristic bending stresses $M_{G,K}$ and $M_{Q,K}$ are obtained from this. The design load is then obtained from:

   $M_d = M_{G,k} \cdot \gamma_G + M_{Q,k} \cdot \gamma_Q$.

   b) The characteristic earth pressure from live loads is multiplied by the factor f_Q according to R 104, Paragraph 5 b) (Section 4.11) before it is superimposed by the earth pressure from soil self-weight, unbounded uniform load $p_k \leq 10\ kN/m^2$ and, if applicable, cohesion according to R 4 (Section 3.2), as well as building loads. The design load is then obtained from the characteristic load M_K using:

   $M_d = M_k \cdot \gamma_G$.

6. An analysis of the loads on the infill occurring when testing, overstressing or loosening anchors or struts may be dispensed with. However, the behaviour of the infilling should be monitored while this work is carried out.

7. The characteristic material parameters and the partial safety factors are given in:

   – EN 1992-1-1 for concrete or reinforced concrete infilling, see Appendix A 7;
   – EN 1993-1-1 for steel infilling, see Appendix A 8, or EN 1993-5 for horizontal trench sheet piles and vertical lightweight sheet pile walls;
   – EN 1995-1-1 for timber infilling, see Appendix A 9.

13.3 Bearing capacity of soldier piles (R 48)

1. The safety against structural failure of soldier piles according to the limit state condition:

   $E_d \leq R_d$

   shall be analysed for the design action effects determined according to Section 5. The design value E_d consists of the loads imposed by the most unfavourable combination of action effects, the design value R_d of the resistance of the structural element.

The steel sections are divided into 4 classes according to the width to thickness ratio (c/t) of the cross-sections under compressive stresses and as a function of the yield strength, also see EN 1993-1-1, Section 5.5. This classification aims to define the stress limits using local bulging of sections.

These classes determine the applicability of the various analysis methods used to determine the action effects or loads and adopting the cross-sectional bearing capacity.

- Class 1 cross-sections:
  Plastic-plastic methods.
  Plastic analyses and design are allowable. However, in order to adhere to the c/t limit value, sufficient rotation capacity shall also be demonstrated.
  Generally, the plastic-plastic case shall not be adopted when designing retaining walls.

- Class 2 cross-sections:
  Elastic-plastic methods.
  Elastic analysis is necessary. Utilisation of the plastic cross-section values is allowable.

- Class 3 cross-sections:
  Elastic-elastic methods.
  Elastic analysis is necessary. Only the elastic cross-section values may be adopted.

- Class 4 cross-sections:
  Elastic-elastic methods with local bulging failure.
  Elastic analysis is necessary. A reduction of the elastic resistance due to local bulging in the elastic range shall be taken into consideration.

For example, in the simplest case of a double-symmetry steel soldier pile with uniaxial bending according to Paragraph 3, Clause 1 and Paragraph 6, Clause 1, an elastic-elastic (steel section ≤ class 3) analysis of the capacity of the cross-section produces the following conservative approximation:

$N_{Ed}/N_{Rd} + M_{y,Ed}/M_{y,Rd} \leq 1.$

Where:

N_{Ed} design value of the acting normal force;
N_{Rd} design value of the normal force capacity;
($N_{c,Rd}$ for compression, $N_{t,Rd}$ for tension);

$M_{y,Ed}$ design value of the acting moment around the y axis;
$M_{y,Rd}$ design value of the bending moment capacity around the y axis;

and:

$$N_{Rd} = A \cdot f_y/\gamma_{M0}$$

$$M_{y,Rd} = W_{el,min} \cdot f_y/\gamma_{M0};$$

and:

A net cross-sectional area;
$W_{el,min}$ smallest elastic resisting moment;
f_y nominal value of yield strength according to Appendix A 8;
γ_{M0} partial safety factor according to Appendix A 8.

This analysis applies to single-propped soldier piles with a free-earth support, where the field moment governs design. If the support moment or fixed-earth moment govern, the analyses detailed in the following paragraphs shall be followed.

2. When designing soldier piles, the dead-load of the retaining wall may be disregarded. In addition to normal forces and moment loads, however, shear stresses and the interactions of the various loads shall be analysed (see EN 1993-1-1, Section 6.2).

3. If no vertical forces other than the dead-load of the retaining wall and the vertical earth pressure component need to be transferred, a bearing capacity analysis according to EN 1993-1-1, Section 6.2 will suffice. In the case of other vertical forces to those previously discussed, e.g. from excavation covers, provisional bridges or inclined anchors, a stability analysis according to EN 1993-1-1, Section 6.3 shall be undertaken, in particular for single-propped retaining walls and for the individual retreating states of multiple-propped retaining walls.

4. In analogy to EN 1993-1-1, Section 5.4.1 (4), a limited plastic moment redistribution may be taken into consideration for multiple-propped walls in the elastic-plastic analysis, if the supporting moments exceed the plastic moment capacity by less than 15 %.

   The conditions specified in EN 1993-1-1, Section 5.4.1 (4), i.e.:

   – redistribution of the excess moment peaks;
   – adherence to the equilibrium conditions;
   – cross-sections in classes 1 or 2;
   – lateral torsional buckling is prevented;

   shall be adhered to.

5. The girder spacing should be as uniform as possible. If the spacing differs greatly between neighbouring girders special measures shall be taken to prevent rotation of the girders as a result of variable loading from the infilling.

6. In the case of soldier pile wall infill wedged behind the front compression flange faces, it can be assumed that these flanges are secured against deflection by the lagging whilst compression flanges on the soil side are protected against deflection by the surrounding soil, assuming the soil is sufficiently stable. Otherwise, additional continuous waling shall be linked to soldier piles in such a way that it provides sufficient strength to counteract lateral torsional buckling of the compression flanges.

7. See R 51 (Section 13.6) for details of configuring and dimensioning waling and tie rods in front of soldier pile walls.

8. Double I- and U-sections shall be connected by battens both on the excavation and on the ground side, ensuring the battens are positioned close enough together. A torsional load capacity analysis may be dispensed with if the batten spacing does not exceed 1.5 m. Stability analysis is not required if sections are fully embedded in concrete, with the exception of their facing sides.

9. Analysis of flange bending as a result of the lagging support forces can generally be dispensed with for single and double I- and U-sections with flange widths ≤ 300 mm. Flange bending for larger flange widths can be analysed using EN 1993-5, Annex D.

10. The characteristic material parameters and the partial safety factors are given in EN 1993-1-1, see Appendix A 8.

11. R 50 (Section 13.5) applies accordingly to designing reinforced concrete piles with infilling installed in the spaces.

12. See R 85 (Section 13.10) for analysis of external bearing capacity, i.e. transfer of vertical forces to the subsurface.

13.4 Bearing capacity of sheet piles (R 49)

1. The safety against structural failure of sheet piles according to the limit state condition:

   $E_d \leq R_d$

   in the form adopted by EN 1993-5:

   $M_{Ed} \leq M_{V,N,Rd}$

shall be demonstrated for the design action effects determined according to Section 6. The design value M_{Ed} comprises the loads resulting from the unfavourable combination of actions, and the design value $M_{V,N,Rd}$ the reduced resisting moment of the sheet pile cross-section resulting from shear force and normal force.

Because the descriptions of the steel sections dealt with in the earlier standards DIN 4114, DIN 18800 and EN 1993-1-1 do not correspond to the forms of the particular sheet pile sections, it was necessary to compile a design standard for sheet piles.

The analysis method is specified in EN 1993-5 and the corresponding NA, and the following points shall be noted:

a) The characteristic material parameters and the partial safety factors are taken from EN 1993-5, see Appendix A 8.
b) The sheet pile sections are divided into 4 classes in analogy to the methods described for steel sections in Section 13.3, Paragraph 1. The parameter for classifying the sheet pile sections is the ratio of the flange width to the flange thickness (b/t_f).
   However, the plastic-plastic case shall not generally be adopted when designing the retaining wall.
c) If necessary, when determining the flexural stiffness and the elastic and the plastic moment resistance for the continuous wall, a reduction factor β_D or β_B shall be taken into consideration for the impact of any reduction in the transfer of shear forces to the sheet pile wall interlocks.

   red. $I_y = \beta_D \cdot I_y$ red. $W_{el,y} = \beta_B \cdot W_{el,y}$ or red. $W_{pl,y} = \beta_B \cdot W_{pl,y}$.

   For Z-sections and triple U-sections these reduction factors shall be adopted at 1.0; they are taken from the National Annex to EN 1993-5 for single- and double-U sections. Where walls consist of double-U sections, which are rigidly connected in at least every second interlock, no reduction factor is adopted for the elastic-elastic analysis method.
d) The impact of normal forces, shear forces and, if applicable, lateral bending due to high positive water pressures and transfer of concentrated loads, e.g. from anchors, shall be taken into consideration for determining the bending limit bearing capacity $M_{V,N,Rd}$. See EN 1993-5, Sections 5.2.2 to 5.2.4 and 7.4.3.
e) A stability analysis (buckling) is only required if the acting normal force is greater than 4 % of the critical normal force. See EN 1993-5, Section 5.2.3 (4).

2. When designing soldier piles, the dead-load of the retaining wall may be disregarded.

3. Where sheet pile walls consist of U-sections, shear force transfer in the zero line (generally the location of the interlocks) shall be demonstrated according to EN 1993-5. It is sufficient to limit this analysis to the following cases for retaining walls:
   a) the sheet pile wall is located in open water or a significant portion of it is driven through peat, tidal mud deposits, mud or soils with a high clay content;
   b) grease or sealant is applied to lubricate against interlock friction prior to the driving process, or interlocks are appropriately protected against penetration of soil particles or;
   c) connecting elements between individual sections exceed the tolerances specified in the EAU, Recommendation R 67 [2].

   However, the reduction factors β_D and β_B shall be adopted for analysis of the cross-sectional capacity according to EN 1993-5 and the corresponding NA, regardless of these specifications.
4. See R 85 (Section 13.10) for analysis of external bearing capacity, i.e. transfer of vertical forces to the subsurface.

13.5 Bearing capacity of in-situ concrete walls (R 50)

1. The safety against structural failure of in-situ concrete walls according to the limit state condition:

   $E_d \leq R_d$

   shall be analysed for the design action effects determined according to Chapter 6. The design value E_d consists of the loads imposed by the most unfavourable combination of action effects, the design value R_d of the resistance of the structural element.
2. With regard to the structural element resistances, the resistances of the concrete and the steel are differentiated:

   a) Concrete: $\sigma_{cd} \leq f_{cd}$ where $f_{cd} = \alpha_{cc} \cdot f_{ck}/\gamma_c$
   and γ_c according to Appendix A 7.
   b) Reinforcing steel: $\sigma_{sd} \leq f_{sd}$ where $f_{sd} = f_{yk}/\gamma_s$
   and γ_s according to Appendix A 7.

   Where:

   σ_{cd} design value of the concrete compressive stress of the actions;
   f_{cd} the design value of the concrete compressive strength;
   σ_{sd} design value of the reinforcing steel stress of the actions;
   f_{sd} design value of the yield stress of the reinforcing steel;

f_{ck} characteristic value of the concrete compressive strength according to Appendix A 7;
f_{yk} characteristic value of the yield stress of the reinforcing steel according to Appendix A 7;
α_{cc} reduction factor according to EN 1992-1-1/NA ($\alpha_{cc} = 0.85$ for reinforced normal strength concrete and $\alpha_{cc} = 0.70$ for unreinforced concrete).

3. EN 1992-1-1 applies for the design and construction of in-situ concrete walls. With regard to reinforcement positioning and concrete cover, the requirements of EN 1538 shall be met for diaphragm walls and those of EN 1536 for pile walls.

4. In addition to reducing the computed maximum reaction moment according to R 11, Paragraph 6 (Section 4.2), the moment diagram may be smoothed out at each data point if concealed beams or reinforced concrete waling are installed. For rolled section waling, the full flange width may be considered as support only if web stiffeners are designed to sufficiently prevent flange deflection and the space between waling and retaining wall is filled with concrete.

5. When determining shear reinforcement, diaphragm wall slices with thicknesses greater than one fifth of their width shall be treated as beams unless individual slices are tightly fitted using dowels. Diaphragm wall elements consisting of several reinforcement cages in a single element length, and which are seamlessly concreted in a single operation, are regarded as tightly dowelled.

   Sufficient dowelling may be achieved by suitable profiling of the joints, for example, for separately manufactured diaphragm wall elements.

6. When analysing anchoring lengths according to EN 1992-1-1, Section 8.4.2, the bonding properties of the horizontal rebars are always classified as moderate, those of the vertical rebars as good.

7. Generally, an analysis to restrict the crack width in in-situ concrete walls is not necessary if the necessary minimum reinforcement according to EN 1992-1-1, Section 9.2.11 is observed. An analysis is necessary if:

   a) the ambient conditions for exposure class XA 3 according to EN 1992-1-1, Table 4.1 need to be taken into consideration;
   b) the ambient conditions for exposure class XS and XA according to EN 1992-1-1, Table 4.1 need to be taken into consideration and the construction stage governing reinforcement design is planned to last more than 2 years.
   c) the in-situ concrete walls form part of a permanent structure.

8. The characteristic material parameters and the partial safety factors are given in EN 01/01/1992, see Appendix A 7.
9. See R 85 (Section 13.10) for analysis of external bearing capacity, i.e. transfer of vertical forces to the subsurface.

13.6 Bearing capacity of waling (R 51)

1. The safety against structural failure of waling according to the limit state condition:

   $E_d \leq R_d$

   shall be analysed for the design action effects determined according to Chapter 5 and Chapter 6. The design value E_d consists of the loads imposed by the most unfavourable combination of action effects, the design value R_d of the sum of the resistances of the structural elements.
2. Analysis of the safety against structural failure of steel beam waling corresponds to the methods given in Section 13.3, Paragraph 1. The adopted material parameters and partial safety factors are given in EN 1993-1-1, see Appendix A 8. Where waling comprises sheet pile sections, EN 1993-5 shall be observed.
3. In analogy to EN 1993-1-1, Section 5.4.1 (4), a limited plastic moment redistribution may be taken into consideration for multiple-propped waling in the elastic-plastic analysis. The information in Section 13.3, Paragraph 4 applies here accordingly.
4. If steel waling subject to bending stresses is utilised to transfer axial forces, a stability analysis shall be performed, where necessary, according to EN 1993-1-1, Section 6.3. Only deflections on the excavation side need be taken into consideration when determining the buckling length.
5. Shear stresses and the interactions between bending/shear/normal forces shall be analysed where steel section waling is subject to bending.
6. If a cantilever effect is considered when determining bending moments, the impact of unintentional displacement of load transfer or reaction points shall be assessed.
7. If web stiffeners are welded on at load transfer or reaction points of steel section waling, or if waling is concreted, it can be assumed that the flanges are sufficiently protected against deflection. This also applies to tightly fit steel plate or timber web stiffeners, provided they are installed with reasonable care and accuracy.

8. If no more precise analysis is performed, bracing elements are required at the load transfer and reaction points and, if applicable, at intermediate points, for waling consisting of single sheet piling (U-sections), in order to retain shape stability.
9. Waling designed to prevent collapse of the retaining wall only for a limited period following complete failure of an anchor or strut may be designed for design situation DS-A according to R 24, Paragraph 5 (Section 2.1), if such an analysis is required in exceptional cases, taking the reserves inherent in the supporting structure (e.g. plastic/plastic design method) and arching effects into consideration and, in contrast to Appendix A 8, fully utilising the yield stress of the steel.
10. In order to ensure the girder spacing of soldier piles, to prevent girder rotation and as a structural measure against the failure of a strut or anchor, at least one waling shall be located in the upper region of the retaining wall and be subject to tension for its length. This is also the case for unsupported soldier pile walls that have only a fixed-earth support. If the uppermost waling is not utilised for this purpose, a lightweight steel section shall be located near the top of the wall or near the uppermost row of struts or anchors, connecting the soldier piles in a straight line and tightly bonded to them by welding or bolting. For excavation depths up to 5 m a tieback of 5 cm^2 cross-section will generally suffice; in addition, a minimum cross-section of 10 cm^2 shall be adopted.
11. The characteristic material parameters and partial safety factors for reinforced concrete waling are given in EN 1992-1-1, see Appendix A 7.

13.7 Bearing capacity of struts (R 52)

1. The safety against structural failure of struts according to the limit state condition:

   $E_d \leq R_d$

   shall be analysed for the design action effects determined according to Section 5 and Section 6. The design value E_d consists of the most unfavourable combination of action effects, the design value R_d of the resistance of the structural element. If the struts also serve as components of a provisional bridge or excavation cover, R 54 (Section 13.9) shall be observed.
2. With regard to exposure to stresses and the risk of failure, struts constitute the most sensitive elements of a retaining wall. Design shall therefore always be based on conservative assumptions. In case of any doubt as to whether the pressure diagram selected for a specific row of struts provides safe support reactions, the support forces shall be appropriately increased.

3. Generally, strut design shall take eccentric force transfer into consideration in addition to the normal force and bending moment. For steel and reinforced concrete struts, deflection due to dead and live loads shall also be considered, see R 56 (Section 2.7). Rolled steel waling shall also be analysed with regard to lateral torsional buckling according to EN 1993-1-1, Paragraph 6.3.
4. If no specific force transfer eccentricity is defined and ensured by appropriate procedures, the stability analysis according to EN 1993-1-1, Paragraph 6.3 performed for steel struts must include the following additional vertical eccentricities:

   a) in cases without end centring, an eccentricity of one sixth of the soldier pile height for rolled sections or one sixth of the tube diameter for tubes;
   b) in cases with end centring, an eccentricity of one sixth of the height of the contact surface.

   The eccentricity shall be added to the bending caused by dead-loads and live loads.
5. If it is necessary to reduce the buckling length of struts, the waling and bracing required for this purpose shall be installed at the top and bottom of the struts. Constructions acting similarly to this may be installed in place of the bottom bracing. If buckling safeguards are undesirable or shall be avoided as far as possible for operational reasons, the use of tubular sections or connected I-sections is recommended.
6. The buckling length is defined as the length of the strut excluding wedges, packing pieces and waling. If the strut ends are not restrained according to design, it shall be assumed that they can rotate freely. Where applicable, this is also valid for points where the buckling length is shortened by an anti-buckling element.
7. The impact of temperature increases shall generally be taken into consideration according to [92]:

   – at long-term construction sites with large seasonal temperature fluctuations;
   – when using slender I-beam struts without anti-buckling elements in sufficiently close spacing;
   – when using short steel struts with anti-buckling elements and relatively stiff abutments, such as provided by rocky ground or in-situ concrete walls;

   except for the cases discussed below. According to [92] analysis may be dispensed with for:

a) steel struts for soldier pile walls;
b) trench sheeting with shoring struts;
c) timber struts.

8. Frost action shall be taken into consideration for narrow excavations if frost-susceptible soils lead to the assumption that the strut forces may increase considerably if the soil freezes.

9. Constructions that serve to reduce the buckling length of struts, such as central supports, waling and bracing, shall be designed for loads perpendicular to these struts, which may be adopted at 1/100 of the sum of the normal forces occurring in the struts. If two or more of these constructions are arranged side-by-side, each one shall be designed for the given load. The same applies to common bracing. Rigid connections, e.g. welding and high-strength screw connections, shall be designed for twice the computed loads, taking possible constraining forces into consideration.

10. The stability analysis (buckling, lateral torsional buckling) shall not be restricted to the individual supporting elements, but shall also address the spatial relationships of the individual components according to EN 1993-1-1.

11. Timber struts may not be subject to impacts. Round timber struts shall display linear growth and no spiral graining.

12. In contrast to the standards discussed in Paragraph 13 below, the partial safety factors for design situation DS-P according to EN 1997-1 and DIN 1054 shall be adopted for determining the design action effects or the design action effects determined for a different load case shall be increased by 15 %.

13. The characteristic material parameters and the partial safety factors are given in:

    – EN 1992-1-1 for struts or stiffening concrete or reinforced concrete base slabs, see Appendix A 7;
    – EN 1993-1-1 for steel struts, see Appendix A 8;
    – EN 1995-1-1 for timber struts, see Appendix A 9. The modification coefficient may be adopted at 1.0, as for structural elements subject to bending loads.

14. When analysing the load-bearing capacity of shoring struts, the Principles for Construction and Working Safety Checks of Adjustable Bracing Elements for Use in Utility Trenches ("*Grundsätze für den Bau und die Prüfung der Arbeitssicherheit von in der Länge verstellbaren Aussteifungsmitteln für den Leitungsgrabenbau*"), published by the German Professional Association for the Civil Engineering Industry shall be adhered to.

15. The use of structural measures shall ensure that the failure of a strut cannot lead to the failure of the structural element secured by the strut. Any conditions presenting a hazard when manufacturing the excavation and to its later use, e.g. from crane operations or material transport, shall be taken into consideration. It may be necessary to implement special protective measures, e.g. deflectors or covers.

16. If, in special cases, strut failure is mathematically analysed, the analysis may take the reserves inherent in the supporting structure and the ground, e.g. arching effects, into consideration and adopt the partial safety factors for the structural element resistances at $\gamma_M = 1.00$. Also see R 51 (Section 13.6).

13.8 Bearing capacity of trench lining (R 53)

1. The safety against structural failure of trench lining according to the limit state condition:

   $E_d \leq R_d$

   shall be analysed for the design action effects determined according to Chapter 5 and Chapter 6. The design value E_d consists of the loads imposed by the most unfavourable combination of action effects, the design value R_d of the resistance of the structural element.

2. The following points apply for horizontal lining:

   a) The pressure diagram for designing the timbers of horizontal trench lining can be stipulated according to R 47 (Section 13.2).
   b) R 47 (Section 13.2) applies for timber design.
   c) R 51 (Section 13.6) applies accordingly for designing soldier beams for horizontal sheeting.

3. The following points apply for vertical lining:

   a) R 49 (Section 13.4) applies accordingly for designing trench sheet piles, driven sheet plates, curtain sections or lightweight sheet pile walls used in vertical trench sheeting.
   b) R 51 (Section 13.6) applies for designing steel section waling.
   c) Timber waling may be designed as for soldier beams according to Paragraph 2 c).

4. Regardless of the specific material used, the cantilever effect of projecting ends and the continuous beam effect may be taken into consideration in the case of multiple-propped elements of horizontal or vertical trench sheeting.

5. R 52 (Section 13.7) applies for strut design.
6. EN 13331-1 and EN 13331-2 shall be observed when designing trench lining systems.

13.9 Bearing capacity of provisional bridges and excavation covers (R 54)

1. The safety against structural failure of provisional bridges and excavation covers according to the limit state condition:

   $E_d \leq R_d$

   shall be analysed for the governing design action effects. The design value E_d consists of the loads imposed by the most unfavourable combination of action effects, the design value R_d of the resistance of the structural element. The partial safety factors for design situation DS-T given in Appendix A 6 apply for determining loads.

2. Determination of the action effects of individual elements of provisional bridges and excavation covers shall take the following loads into consideration in addition to dead-loads:

   a) For provisional bridges and excavation covers designed to accommodate public road and rail traffic; loads according to R 55 (Section 2.6).
   b) For provisional bridges and excavation covers for site traffic, as well as for excavation covers provided to create storage or work spaces; loads according to R 56 (Section 2.7).
   c) For the operating areas of excavators and lifting equipment; loads according to R 57 (Section 2.8).
   d) For pipe bridges; dead loads of cables, pipes, protective elements and, if applicable, materials or substances inside pipes, including resultant deflection and surge forces.
   e) For protective covers; the characteristic values of wind loads according to EN 1991-1-4, the characteristic values of snow loads according to EN 1991-1-3 and, if applicable, loads resulting from the build-up of water pockets on sheet coverings.

   If the main girders of provisional bridges or excavation covers also act as stiffening elements, R 52 (Section 13.7) shall also be observed.

3. The characteristic material parameters and the partial safety factors are generally given in:

   – EN 1992-1-1 for concrete or reinforced concrete structural elements, see Appendix A 7;

– EN 1993-1-1 for steel structural elements, see Appendix A 8;
– EN 1995-1-1 for timber structural elements, see Appendix A 9;

unless, as in the case of rail traffic, for example, the respective transport company's regulations prevail.
For construction measures where the DIN Technical Reports 101 "Actions on Bridges" (*Einwirkungen auf Brücken*), 102 "Concrete Bridges" (*Betonbrücken*), 103 "Steel Bridges" (*Stahlbrücken*) and 104 "Composite Bridges" (*Verbundbrücken*) form a component of the contract, i.e. in general for construction measures within the field of responsibility of the (German) Federal Ministry of Transport, Building and Urban Development (*Bundesministerium für Verkehr, Bau- und Stadtentwicklung (BMVBS)*), the stipulations made therein shall be observed.

4. In addition to the standard analyses prescribed in generally accepted regulations and guidelines, e.g. ultimate limit state analyses, the following analyses shall generally be undertaken for provisional bridges and excavation covers:

   a) Transfer of vertical and horizontal loads from road pavements into the ground via the supporting structure and retaining wall and, if necessary, by means of intermediate supports and load distributing bearing structures.
   b) Safety of road pavements and supporting structures against uplift, also with a view the anticipated noise impacts caused by pavement detachment.

5. For analysis of the serviceability limit state according to R 78, Paragraph 9 (Section 1.4) it may be necessary to limit bending in provisional bridges and excavation covers and to select their dimensions as a function of the tolerable deflection. The following criteria may serve to determine such dimensions:

   a) For provisional bridges and excavation covers designed for road and rail traffic; maximum permissible speed, potential hazards to the pavement, driving comfort, or impact on vehicles.
   b) For pipe bridges for rigid pipes; strength and deformation behaviour of pipes and sleeves if deflection cannot be compensated for by appropriate structural design.
   c) For protective covers; the amount of water drainage required to prevent ponding.

   For provisional bridges and excavation covers designed for road and rail traffic, it is widely accepted practice to restrict live-load related deflection to 1/500 of the structure's span. Moreover, dead-load related structural deflection is often compensated for by appropriate superstructure design, which is of particular relevance if the structure will accommodate rail traffic. In points areas, it may be necessary to reduce deflection even further and to restrict the potential rotation angle at the ends of main girders to a tolerable level.

13.10 External bearing capacity of soldier piles, sheet pile walls and in-situ concrete walls (R 85)

1. For analysis of the vertical bearing capacity as demanded by R 84 (Section 4.8), determination of the characteristic resistances between the retaining wall elements and the ground in the ultimate limit (ULS) state is required; here, this is referred to as the "external" bearing capacity.
2. The characteristic resistances shall be determined on the basis of load tests, regardless of the type of retaining wall elements. If no load tests are carried out the characteristic resistances of the retaining wall elements against vertical loads may be based on empirical data.
   Taking the demands on the ground into consideration for the respective situation, the following points apply for the characteristic base resistance and the characteristic skin friction:
   a) for in-situ concrete walls, soldier piles placed in boreholes and concreted at the base, and driven soldier piles, the information given in [165];
   b) for sheet pile walls, the information in Appendix A 10.
3. The following points apply for determining the characteristic base resistances according to R 84, Paragraph 2 c) (Section 4.8):
   a) The actual toe and base areas shall be adopted for in-situ concrete walls and concreted soldier piles according to Paragraph 2 a).
   b) The governing toe or base area of a sheet pile wall is obtained from the cross-sectional area of the steel as shown in Figure R 85-1 a).
   c) The full girder cross-section may be adopted for driven soldier piles as shown in Figure R 85-2 a).

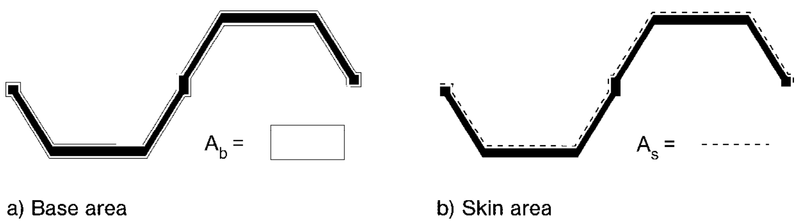

Figure R 85-1. Effective base area and skin area of driven sheet pile walls

a) Base area   b) Skin area

Figure R 85-2. Effective base area and skin area of driven soldier piles

4. The following points apply for determining the characteristic skin resistances according to R 84, Paragraph 2 d) (Section 4.8):

   a) The skin areas below the excavation level shall be adopted for in-situ concrete walls, sheet pile walls and concreted soldier piles.
   b) The developed surface of the rolled section may be adopted for driven soldier piles.

   Generally, the skin area may be adopted to transfer vertical forces on the excavation side only. This is represented by the dashed area as shown in Figure R 85-2 b). The skin surface on the earth side may only be adopted:

   – if the wall friction angle is adopted at $\delta_a = 0$ or with a negative sign;
   – if the wall is embedded deeper than calculated necessary; however, only in the zone of the additional embedment depth.

5. Because vertical displacements can generally be allowed in excavations, the upper characteristic values given in [165] for both the tip pressure and the skin friction of in-situ concrete walls and soldier piles may be adopted. These values are already included in Appendix A 10 for sheet pile walls. Where settlement-prone structures are located adjacent to excavations, it shall be examined whether a reduction should be applied for in-situ concrete walls and soldier piles, and sheet pile walls according to Appendix A 10.

6. Without further analysis an embedment depth of 1.5 m is generally sufficient for excavations up to 10 m deep and favourable ground conditions, if only the dead-loads loads of the wall and the vertical earth pressure component are transferred to the subsurface. Otherwise, the following points apply:

   a) Transfer of vertical forces shall always be analysed if:

      – the excavation is deeper than 10 m;
      – there is no sufficiently load-bearing ground below the excavation level or;
      – other vertical forces act, e.g. from anchorages or from provisional bridges and excavation covers.

b) Shallower embedment depths than:

   – $t_g = 3.00$ m for driven sheet pile walls and soldier piles or;
   – $t_g = 2.50$ m for in-situ concrete walls and concreted soldier piles;

   are not permissible for transferring additional loads, in addition to the dead-load of the retaining wall and the vertical component of the earth pressure, without more precise analysis.

7. If the 3.00 m and 2.50 m embedment depths stipulated in Paragraph 5 are not adhered to when analysing transfer of vertical forces from dead-weight and earth pressure, in particular for an excavation depth of more than 10 m, the toe resistance determined according to Paragraph 3 shall be reduced by the calibration factor η_t. This calibration factor may be determined as follows:

   $$\eta_t = \frac{t_g - 0{,}50\ \text{m}}{2{,}50\ \text{m}}$$ for driven sheet pile walls and soldier piles;

   $$\eta_t = \frac{t_g - 0{,}50\ \text{m}}{2{,}00\ \text{m}}$$ for in-situ concrete walls and concreted soldier piles.

8. See R 84, Paragraph 6 (Section 4.8) for determining the design values $R_{c,d}$ from the characteristic resistances $R_{c,k}$.

13.11 Bearing capacity of tension piles and ground anchors (R 86)

1. Tension piles are deployed in retaining walls to anchor excavation bases in water according to R 62 (Section 10.5) and to anchor retaining walls according to R 43 (Section 7.2). Generally, only displacement piles, shaft-grouted displacement piles or grouted micropiles are employed. Ground anchors are used in retaining walls to anchor them according to Chapter 7 and, where applicable, to anchor excavation bases in water according to R 62 (Section 10.5).

2. Sufficient failure safety according to the Eurocode 7 Handbook, Volume 1, is given if the limit state conditions:

   $F_{t,d} \leq R_{t,d}$ for tension piles

   and

   $P_d \leq R_{a,d}$ for ground anchors

   are met. The design forces of the actions or loads are obtained from the determination of action effects according to R 11 (Section 4.2), R 81 (Section 4.1) and R 82 (Section 4.4), or according to R 62 (Section 10.5) for the

GEO 2 limit state, using the partial safety factors according to Table 6.1 given in Appendix A 6.

According to the Eurocode 7 Handbook, Volume 1, the characteristic tension pile resistance $R_{t,k}$ determined from pile load tests or, in exceptional cases, from empirical data, is divided by the partial safety factor $\gamma_{s,t}$, multiplied by the model factor $\eta_M = 1.25$, regardless of the pile inclination, when defining the design value of the pile resistance $R_{t,d}$ for grouted pile systems (grouted micropiles to DIN EN 14199 and grouted displacement piles to DIN EN 12699), such that:
$R_{t,d} = R_{t,k}/(\gamma_{s,t} \cdot \eta_M)$.

3. See the Eurocode 7 Handbook, Volume 1, for determining the characteristic tension pile resistances $R_{t,k}$. The Eurocode 7 Handbook, Volume 1, also governs determination of the governing anchor resistance and the necessary anchors tests.

4. The use of structural measures shall ensure that the failure of a ground anchor or a tension pile does lead to the failure of the secured structural element. If a stability analysis is performed in this case, it may take all the reserves inherent in the supporting structure and in the ground into consideration.

5. The implementation of structural measures or a calculation of the possible failure of a ground anchor may be dispensed with if the following conditions are met:

   a) Each anchor (temporary anchors) is tested to $P_p = 1.5 \cdot P_k$ during the acceptance test. The internal bearing capacity of the tendon shall be analysed with regard to the tensioning process.
   b) Strand or bar anchors are used. It shall be demonstrated for strand anchors that if one strand fails the remaining strands are capable of transferring the anchor force P_d, whereby the design situation DS-A may be adopted. This condition may be regarded as met for anchors with four or more strands.
   c) The load-bearing components of the anchor head shall be installed as deep as possible behind the front edge of the soldier piles, sheet pile wall, pile wall or diaphragm wall, if hazards from site operations cannot otherwise be ruled out.

14 Measurements and monitoring on excavation structures

14.1 Purpose of measurements and monitoring (R 31)

1. Measurements serve as a means of quality control and thus demonstrate technically faultless planning and construction. In this context, measurements may also represent a component of conservation of evidence against third parties, e.g. authorities or neighbours.

   Hence, where excavation structures are classified as GC 3, it is necessary to always design and implement a suitable monitoring concept. Where excavation structures are classified as GC 2, the necessity for monitoring is decided on a project-specific basis.

   The establishment of a proper monitoring concept forms part of draft and approval planning, in which the type and extent of measurements are specified. The monitoring concept shall be expanded and adapted as required during construction, corresponding to the construction method employed.

2. The following points present the objectives of measurements and monitoring on excavation structures:

   a) To check the design parameters and the results of the structural analyses, see Paragraph 3.
   b) To examine the effects of design changes and deviations from the design during construction, see Paragraph 4.
   c) To optimise the design and the construction process, see Paragraph 5.
   d) To apply the observational method according to the Eurocode 7 Handbook, Volume 1, see Paragraph 6.

3. Examination of the projected behaviour initially concerns the planning fundamentals directly entered in the analyses. For excavation structures these are initially:

   – the adopted ground characteristics, primarily determined by the stratigraphy and the associated soil properties;
   – the groundwater levels;
   – the loads from adjacent buildings, traffic or other actions.

   Examination of the calculation results achieved using these planning fundamentals generally consists of the following points:

   – the design loads, i.e. the magnitude and distribution of earth and water pressures;

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- the calculated displacements of the excavation structure;
- the projected forces in anchors or struts.

4. The effects of planning changes or the deviations to plans made or identified during construction can be identified by measurements and assessed based on the measurement results. For example, deviations from planning can include:

   - ground properties (stratigraphy, soil parameters, etc.);
   - water levels (tides, perched water, groundwater);
   - construction progress;
   - duration of the construction project;
   - embedment of the retaining wall;
   - loads on the retaining wall;
   - anchor resistance;
   - water volumes for dewatering.

5. Some examples of optimisation options that may be implemented during the building process based on measurement results are given below:

   - increasing the spacing of the load-bearing elements of a soldier pile wall in linear excavations or adopting a smaller soldier profile;
   - reducing retaining wall embedment;
   - reducing the extent of dewatering.

6. Measurements carried out using the observational method according to the Eurocode 7 Handbook, Volume 1, represent a principal component of ultimate limit state and serviceability limit state design. They should therefore be closely integrated in the geotechnical analyses and numerical projections. Also see R 33, Paragraph 5 (Section 14.3).

14.2 Measurands and measuring methods (R 32)

1. Measurements generally involve determination of the following measurands:

   - lengths;
   - displacements (e.g. bending, settlement, heave, horizontal displacement);
   - spatial orientation or position in a global coordinate system;
   - tensile and compressive strains;
   - angular distortions;
   - forces;
   - stresses and pressures;
   - water levels and porewater pressures;

- vibration velocities and accelerations;
- times;
- temperatures.

2. The selected measurands should provide the following information, for example, depending on the existing local conditions and the respective problem at hand:
   - orientation of the retaining wall and its load-bearing elements after construction;
   - wall and ground displacements, e.g. as a result of dewatering, excavation, anchor installation, installation of horizontal grouted cut-offs, building construction, anchor or strut removal, backfilling;
   - earth and water pressures;
   - anchor, strut and pile forces;
   - displacements and deformations of adjacent structures.
3. Measuring methods are principally differentiated into discrete and continuous measurements. In many cases discrete measurements can be carried out manually. Continuous measurements require automatic data collection and forwarding. Robust and reliable measurement methods, which meet the minimum demands on measurement accuracy, should be selected.
4. The following methods are commonly used to record the discussed measurands at excavation structures:
   a) Retaining wall displacements can be determined geodetically using analogue or digital levelling devices or total stations. Motorised instruments with automatic target detection allow automatic measurements with continuous evaluation by a central data logging system. In addition, methods based on laser scanning are available, allowing linear metrological recording of structures.
   b) Inclinations and horizontal displacements of a retaining wall can be measured using inclinometers.
   c) On the one hand, displacements in the neighbouring ground or of adjacent structures can be logged manually, e.g. using inclinometers, probe extensometers, sliding deformeters or settlement gauges, and on the other automatically, e.g. using chain inclinometers or rod extensometers. In addition, water level gauges are available for monitoring buildings adjacent to excavations.
   d) Strain gauges, vibrating string sensors and glass-fibre systems are available for strain measurements, e.g. on steel girders or reinforcing steel.
   e) Electrical or hydraulic load cells with electrical pressure transducers are commonly used to measure forces, e.g. strut or anchor forces. They also allow remote monitoring.

f) In special cases, direct determination of the earth pressure acting on retaining walls may be appropriate. Electrical or hydraulic earth pressure cells may be used for measuring earth pressures.
g) Conventional open standpipes are most commonly used for measuring the groundwater table and the head the groundwater in permeable soils. The piezometric heads are determined using sounding lights or pressure transducers. Pressure transducers generally allow more accurate measurements. They also allow the water pressure to be continuously recorded. Closed systems based on electrical or pneumatic pressure transducers have been developed in particular for low-permeability soils with only minor quantities of water available for measurement. For rapidly changing groundwater levels and changeable porewater pressures, e.g. in the course of consolidation processes, automatic data logging with continuous measurement results is beneficial.
h) Profile measuring devices can be employed to check the dimensional accuracy and verticality of boreholes and open trenches.
i) Seismic measuring devices allow dynamic loading on adjacent structures to be recorded, e.g. during driving sheet pile walls.
j) Geophysical measuring methods are employed to explore heterogeneities in the ground or the structural element, e.g. for localising obstructions, voids or leaks.

14.3 Measurement planning (R 33)

1. When planning measurements on excavation structures the following points must be considered or specified:
   - identification of possible hazard scenarios and a risk assessment;
   - identification of the required measurands and selection of suitable measuring methods, also see R 32 (Section 14.2);
   - definition of threshold, intervention and alarm levels to describe the limits of the structure’s behaviour, including instructions for action in an alarm plan in case the specified values are reached;
   - defining monitoring points, also see Paragraph 4 and R 34 (Section 14.4);
   - stipulation of the measurement process, with particular regard to the time of measurements and forwarding measurement results, also see R 35 (Section 14.5);
   - stipulation of the type and extent of measurement result interpretation and documentation, also see R 36 (Section 14.6).
2. Possible hazard scenarios and risks related to excavation structures can affect both bearing capacity and serviceability. A number of examples are given below:

- The occurrence of large displacements of a retaining wall, which can lead to unacceptable displacements of adjacent structures or an unacceptable reduction of the planned clear dimensions of the excavation.
- Loss of anchor resistance, failure of the anchored ground or loss of stability at low failure plane thus compromising stability of the retaining wall.
- Risks, avoidable or unavoidable, occurring during the excavation structure's construction process, e.g. softening of the ground due to the installation of cased boreholes for bored piles in the sand below the groundwater table, or settlement of adjacent buildings due to the installation of anchors or diaphragm walls.
- Occurrence of heave or failure by piping due to groundwater flow around a retaining wall, leading to a reduction in, or complete loss of, passive earth pressure or presenting a hazard to adjacent structures.

3. Threshold, intervention and alarm levels are defined as follows:

   a) The threshold level is reached when the measurands have a smaller margin to the intervention level as defined, i.e. the measured data critically approach the defined intervention level. During monitoring, heightened attention is paid to the behaviour of the structure, the structural element or the ground, including the groundwater conditions. This can be achieved by e.g. reducing the measurement intervals. Possible additional measures shall be prepared.
   b) The intervention level defines the boundary of the measurands upon which additional measures are immediately required. Close coordination between the parties involved in planning and construction is required to enable these additional measures.
   c) Reaching the alarm level indicates abnormal loading, which effects the bearing capacity of the excavation structure or the surrounding ground. Safety measures aimed at protecting persons and property shall be initiated immediately.

4. The configuration of monitoring points and the type and extent of measurements are defined jointly by the parties involved in the project and, if applicable, any affected third parties. Also see R 34, Section 14.4. The following criteria shall be observed in particular:

- potential hazards to life and limb;
- potential hazards to public safety and order;
- potential hazards to third party assets;
- potential hazards to completed works (time and financial expense involved in rectifying any damage and reinstating the planned condition);
- type of construction, foundations, size, distance and sensitivity of adjacent structures;

– ground and groundwater conditions;
– depth, size and complexity of the excavation structure;
– construction progress and duration of the project.

5. If the observational method according to the Eurocode 7 Handbook, Volume 1, is adopted, a monitoring programme shall be prepared in close cooperation with the structural engineer and the geotechnical designer, allowing the system behaviour to be examined for compliance with the defined boundaries, based on meaningful measurands. The following requirements shall generally be met:

   a) Failure of the structure or the ground surrounding the structure shall be recognisable by suitable measurements or make itself noticeable at an early stage such that structural countermeasures can be implemented in time. To facilitate this the reaction times of the transducers and the times until the data are evaluated shall be sufficiently short. The use of online-monitoring systems with an integrated alarm function activated if defined boundaries are reached is beneficial.
   b) With regard to failure mechanisms that cannot be ruled out, the structure shall facilitate retrofitting with suitable structural measures. These measures shall be planned and coordinated from the outset in the course of approval planning and detailed design.
   c) The measures discussed above shall be prepared as part of the construction process, allowing them to be implemented immediately if required.

6. Redundancy should be aimed for regardless of the selected measuring methods, i.e. it should be possible to monitor a measurand by measuring with a different measuring system or at a different measuring point, or at least to check for plausibility.

7. Once the type and extent of measurements are determined, the data format, the time and the persons to whom they shall be forwarded shall be stipulated, in addition to who shall interpret them.

14.4 Location of measuring points (R 34)

1. The locations of measuring points generally follow the criteria given below:

   a) Setting the measuring points shall primarily be located in areas or on elements of the excavation structure with a high hazard potential, e.g. adjacent to sensitive neighbouring buildings.
   b) The measurement results obtained should be representative for an area of the retaining wall as large as possible (homogeneous area). Homoge-

neous areas are primarily defined regarding the load-bearing structure, the selected construction method, ground and groundwater conditions, actions and, if applicable, adjacent structures.

c) In an extensive excavation, e.g. a linear excavation, the section of the excavation for which the measuring point is representative shall be defined before monitoring begins.

d) Where any impairment or hazard to adjacent buildings is possible the measuring points shall be arranged perpendicular to the front and rear of the retaining wall according to the anticipated impact of the excavation structure.

2. If measurements serve to verify calculated predictions, for example in the case of the observational method, the configuration of the measuring points shall be adapted to the analyses and the design criteria selected in the analyses. For example, if the retaining wall displacements govern the design of a sheet pile wall, they should be monitored during construction.

3. Sufficient options for verification of the measuring results should be planned for during configuration of the measurement points. This means, for example, that the configuration of two measuring points in one and the same homogeneous region should be given preference over a great number of homogeneous regions with only single measuring points. The aim is primarily to acquire reliable information on the structure-soil-interaction in general.

14.5 Carrying out measurements and forwarding measurement results (R 35)

1. Measurements should generally be made at the following times:

   – immediately after installing the measuring instruments;
   – before and after loading the structural element being measured;
   – before and after every construction stage;
   – before and after unloading the structural element;
   – before removing the measuring instruments.

2. Some measuring methods may require calibration or zero measurements respectively before installing the measuring instruments. As far as is technically possible, these should be repeated after measurements are complete in order to recognise any changes to the measuring device (e.g. sensor drift) during measuring and to compensate for them during evaluation.

3. Further measurements depend on the time-dependent behaviour of the material of both the structural element itself, e.g. creeping of an anchor, and of

the ground, e.g. due to changes in porewater pressure. Changes in groundwater conditions with time shall also be taken into consideration when stipulating measuring intervals.

4. Where practicable, the time of the measurements should be selected such that the external conditions, e.g. temperature or tide levels, are comparable for each respective measurement. Where required, the influence of varying boundary condition on the measurands shall be determined by supplementary measurements.

5. Measurement results are forwarded in accordance with the specification in R 33, Paragraph 7 (Section 14.3). In addition, forwarding and further processing of the measurement results are controlled individually for each excavation structure by the quality assurance system. In the case of exceptional events, e.g. unexpected large retaining wall displacements, the instructions for action described in the alarm plan shall also be adhered to.

6. To facilitate coherent evaluation of the measuring results the full significant information shall be forwarded, in addition to the actual measurement results (raw data). This includes such things as the excavation stages, groundwater levels and ambient temperatures, for example.

7. In order to give the construction project participants sufficient time to react, if necessary, the time between logging, forwarding and evaluation of the measured data shall be kept to the minimum possible. For example, if measurements are made automatically, the construction project participants may be allowed access to the measured data by establishing appropriate data logging systems (web-based) and using online networks.

8. In order to allow rapid reactions to abnormal or critical changes in the predicted conditions, threshold levels, intervention levels and alarm levels shall be defined in an alarm plan. If these levels are reached, previously defined instructions for action and work shall be followed. Also see R 33 (Section 14.3). For automatic data logging, alarms can be implemented within the logging software in case one of the defined levels is reached. Alarm notifications can then be sent automatically, e.g. by means of text and email notifications.

9. The levels discussed above can be adapted during the construction process in agreement with the parties involved in planning and construction.

14.6 Evaluation and documentation of measurement results (R 36)

1. The following points shall be noted, in particular, when evaluating measurement results:

a) The raw data shall be evaluated and processed in such a way that the measured data relevant to the assessment can be filtered out and visualised in a suitable format, together with the additional information governing the evaluation. See also Paragraph 2.
b) The measured data shall be checked for plausibility. For example, if measuring errors or unexpected measuring results are identified, it may be necessary to repeat the measurements or to collect additional information to facilitate the assessment.
c) When interpreting measuring results the actual site conditions shall be considered.
d) If information on the load bearing behaviour of the excavation structure is required to evaluate the measurement results, the structural engineer shall be consulted during assessment and evaluation of the measuring results.

2. Graphical visualisations are essential for assessing the measuring results, especially for large quantities of data. In most cases automatic data logging systems provide suitable options in the measuring and evaluation software.

3. When evaluating the measurands, the effect of temperature shall be considered as follows:

    a) The effect of temperature changes on the structural element being measured, e.g. a steel strut, shall be determined by parallel temperature measurements on the structural element.
    b) The effect of temperature changes on the measuring system or the sensor, e.g. the hydraulic pressure in a pressure cell, shall be compensated for.

4. Measurement reports shall be produced or updated at regular intervals. They shall include all the information pertinent to the measurements (for example construction stage, construction activities, groundwater levels). Once measurements are complete the data and measurement reports shall be summarised and documented in their entirety. These documents shall be treated as part of the as-built documents.

5. The unprocessed raw data and the processed or converted measured data are stored separately and independently of one another. The measured data (raw data and relevant, processed data) are archived together with the other as-built documents. The measured data shall be stored in file and database formats that are both universal and remain available in the long term. If this is not possible, a suitable export function should be available.

Annex

A 1: Relative density of cohesionless soils

Based on DIN 1054 'Verification of the Safety of Earthworks and Foundations'.

Table 1.1. Definition of relative density

$$D = \frac{\max n - n}{\max n - \min n} = \frac{\rho_d - \min \rho_d}{\max \rho_d - \min \rho_d} = \frac{\gamma_d - \min \gamma_d}{\max \gamma_d - \min \gamma_d}$$

Compaction	Relative density		Cone resistance of CPT MN/m^2
	$U \leq 3$	$U > 3$	
Very loose	$D < 0.15$	$D < 0.20$	$q_c < 5.0$
Loose	$0.15 \leq D < 0.30$	$0.20 \leq D < 0.45$	$5.0 \leq q_c < 7.5$
Medium-dense	$0.30 \leq D < 0.50$	$0.45 \leq D < 0.65$	$7.5 \leq q_c < 15$
Dense	$0.50 \leq D < 0.75$	$0.65 \leq D < 0.90$	$15 \leq q_c < 25$
Very dense	$0.75 \leq D$	$0.90 \leq D$	$q_c > 25$

Table 1.2. Criteria for medium-dense compaction

Soil class to DIN 18196	Uniformity coefficient	Relative density	Proctor density	CPT cone resistance
SE, SU GE, GU, GT	$U \leq 3$	$D \geq 0.3$	$D_{Pr} \geq 95\%$	$q_s \geq 7.5$ MN/m^2
SE, SW, SI, SU GE, GW, GT, GU	$U > 3$	$D \geq 0.45$	$D_{Pr} \geq 98\%$	$q_s \geq 7.5$ MN/m^2

Table 1.3. Criteria for dense compaction

Soil class to DIN 18196	Uniformity coefficient	Relative density	Proctor density	CPT cone resistance
SE, SU GE, GU, GT	$U \leq 3$	$D \geq 0.5$	$D_{Pr} \geq 98\%$	$q_s \geq 15$ MN/m^2
SE, SW, SI, SU GE, GW, GT, GU	$U > 3$	$D \geq 0.65$	$D_{Pr} \geq 100\%$	$q_s \geq 15$ MN/m^2

Recommendations on Excavations – EAB, 3rd edition.
Editors: German Geotechnical Society (Deutsche Gesellschaft für Geotechnik e. V.)

A 2: Consistency of cohesive soils

Definitions

The consistency depends on the water content w of the soil (see DIN 18121-1). With decreasing water content, cohesive soil changes its state from liquid to plastic to semi-solid to solid (hard). Transitions from one state to another were defined by *Atterberg* and are known as consistency limits:

a) The liquid limit w_L is the water content at the transition from liquid to plastic state.
b) The plastic limit w_P is the water content at the transition from plastic to semi-solid.
c) The shrinkage limit w_S is the water content at the transition from the semi-solid to the solid (hard) state.
d) The plasticity index I_P is the difference between liquid and plastic limit:

   $I_P = w_L - w_P$
e) The plastic range between the liquid and the plastic limit is sub-categorised into very soft, soft, and firm states.

Determination of consistency in laboratory tests

Based on the water content at the liquid limit w_L and at the plastic limit w_P, the consistency index is computed using the soil water content w:

$$I_C = \frac{w_L - w}{w_L - w_P} = \frac{w_L - w}{I_P}$$

The following I_C values correspond to the plastic state sub-categories:

a) $I_C = 0.00$ to 0.50: very soft consistency;
b) $I_C = 0.50$ to 0.75: soft consistency;
c) $I_C = 0.75$ to 1.00: firm consistency.

Determination of consistency in field tests

The following criteria shall be applied to field tests in order to determine the cohesive soil state:

a) A soil that is squeezed through the fingers when making a fist is **very soft**.
b) A soil that is easy to knead is **soft**.
c) A soil that is difficult to knead but can be formed to 3 millimetre thick rolls in the hand without cracking or crumbling is **firm**.
d) A soil that cracks and crumbles when attempting to form 3 millimetre thick rolls but is still moist enough to be re-formed to a clod is **semi-solid**.
e) A soil that has dried out and generally appears light-coloured is **solid** (hard). This soil can no longer be kneaded but only broken apart. Subsequent balling of individual pieces is no longer possible.

A 3: Soil properties of cohesionless soils

Table 3.1. Empirical values for the unit weight of cohesionless soils

Soil type	Abbreviation to DIN 18196	Compaction	Unit weight		
			Earth moist γ_k [kN/m^3]	saturated $\gamma_{r,k}$ [kN/m^3]	Buoyant γ'_k [kN/m^3]
Gravel, sand, uniformly graded	GE, SE with U < 6	Loose Medium-dense Dense	16.0 17.0 18.0	18.5 19.5 20.5	8.5 9.5 10.5
Gravel, sand, well or intermittently graded	GW, GI, SW, SI with $6 \le U \le 15$	Loose Medium-dense Dense	16.5 18.0 19.5	19.0 20.5 22.0	9.0 10.5 12.0
Gravel, sand, well or intermittently graded	GW, GI, SW, SI with U > 15	Loose Medium-dense Dense	17.0 19.0 21.0	19.5 21.5 23.5	9.5 11.5 13.5

The following points should be observed when adopting the table values:

a) The given empirical values of the unit weight are characteristic average values.
b) When analysing safety against heave, safety against hydraulic failure and safety against uplift, the unit weights are reduced:

   – by 1.0 kN/m^3 for an earth moist soil;
   – by 0.5 kN/m^3 for a saturated or a buoyant soil.

   The lower characteristic values of the unit weight are obtained.

Table 3.2. Empirical values for the shear strength of cohesionless soils

Friction angle			
Soil type	Abbreviation to DIN 18196	Compaction	Friction angle φ'_k [°]
Gravel, sand, Uniformly, well or intermittently graded	GE, SE, GI, SE, SW, SI	Loose Medium-dense Dense	30.0 – 32.5 32.5 – 37.5 35.0 – 40.0

Capillary cohesion		
Soil type	Designation to DIN 4022-1	Capillary cohesion $c_{c,k}$ [kN/m^2]
Sandy gravel Coarse sand Medium sand Fine sand	G, s gS mS fS	0 – 2 1 – 4 3 – 6 5 – 8

The following points should be observed when adopting the table values:

a) The empirical values given for the angle of friction φ'_k and for capillary cohesion $c_{c,k}$ represent conservative estimates of the average value according to DIN 1054. They apply to round and rounded grains.

b) If angular grains obviously dominate, the given friction angle values may be increased by 2.5°.

c) Adoption of the given bandwidths for the shear strength values assumes that the author of the draft and the technical planner posses expertise and experience in the geotechnical field. Otherwise, only the smallest values may be adopted.

d) The empirical values given for capillary cohesion $c_{c,k}$ shall be adopted as follows:

- the lower values apply for a saturation of 5% ≤ S_r ≤ 40% and loose compaction;
- the upper values apply for a saturation of 40% ≤ S_r ≤ 60% and dense compaction.

If required, interpolation between these values may be performed.

Capillary cohesion may only be taken into consideration if it cannot be lost by drying or flooding of the subsoil due to a rising groundwater table or water ingress from above during construction work.

A 4: Soil properties of cohesive soils

Table 4.1. Empirical values for the unit weight of cohesive soils

Soil type	Abbreviation to DIN 18196	Consis-tency	Unit weight		
			Earth moist γ_k [kN/m^3]	satu-rated $\gamma_{r,k}$ [kN/m^3]	Buoyant γ'_k [kN/m^3]
Silty soils					
Slightly plastic silts ($w_L < 35\%$)	UL	Soft Firm Semi-solid	17.5 18.5 19.5	19.0 20.0 21.0	9.0 10.0 11.0
Medium-plastic silts ($35\% \leq w_L \leq 50\%$)	UM	Soft Firm Semi-solid	16.5 18.0 19.5	18.5 19.5 20.5	8.5 9.5 10.5
Clay soils					
Slightly plastic clays ($w_L < 35\%$)	TL	Soft Firm Semi-solid	19.0 20.0 21.0	19.0 20.0 21.0	9.0 10.0 11.0
Medium-plastic clays ($35\% \leq w_L \leq 50\%$)	TM	Soft Firm Semi-solid	18.5 19.5 20.5	18.5 19.5 20.5	8.5 9.5 10.5
Highly plastic clays ($w_L > 50\%$)	TA	Soft Firm Semi-solid	17.5 18.5 19.5	17.5 18.5 19.5	7.5 8.5 9.5
Organic soils					
Organic silt Organic clay	OU and OT	Very soft Soft Firm	14.0 15.5 17.0	14.0 15.5 17.0	4.0 5.5 7.0

The following points should be observed when adopting the table values:

a) The given empirical values of the unit weight are characteristic average values.
b) For cohesive soils with particularly flat grading curves, such as boulder clay, with grain sizes ranging from clay to sand or gravel (mixed-grained soils of groups GU, GT, SU and ST or GU*, GT*, SU* and ST* according to DIN 18196), the empirical unit weights given shall be increased by 1.0 kN/m^3.
c) When analysing safety against heave, safety against hydraulic failure and safety against uplift, the unit weights are reduced:
   – by 1.0 kN/m^3 for an earth moist soil;
   – by 0.5 kN/m^3 for a saturated or a buoyant soil.

   The lower characteristic values of the unit weight are obtained.

Table 4.2. Empirical values for the shear strength of cohesive soils

Soil type	Abbreviation to DIN 18196	Consis-tency	Shear strength		
			Earth moist		Cohesion
			φ'_k [°]	c'_k [kN/m²]	c'_u [kN/m²]
Silty soils					
Slightly plastic silts ($w_L < 35\%$)	UL	Soft Firm Semi-solid	27.5 – 32.5	0 2 – 5 5 – 10	5 – 60 20 – 150 50 – 300
Medium-plastic silts ($35\% \leq w_L \leq 50\%$)	UM	Soft Firm Semi-solid	22.5 – 30.0	0 5 – 10 10 – 15	5 – 60 20 – 150 50 – 300
Clay soils					
Slightly plastic clays ($w_L < 35\%$)	TL	Soft Firm Semi-solid	22.5 – 30.0	0 – 5 5 – 10 10 – 15	5 – 60 20 – 150 50 – 300
Medium-plastic clays ($35\% \leq w_L \leq 50\%$)	TM	Soft Firm Semi-solid	17.5 – 27.5	5 – 10 10 – 15 15 – 20	5 – 60 20 – 150 50 – 300
Highly plastic clays ($w_L > 50\%$)	TA	Soft Firm Semi-solid	15.0 – 25.0	5 – 15 15 – 20 15 – 25	5 – 60 20 – 150 50 – 300
Organic soils					
Organic silt Organic clay	OU and OT	Very soft Soft stiff	17.5 – 22.5	0 2 – 5 5 – 10	2 – 20 5 – 60 20 – 150

The following points should be observed when adopting the table values:

a) The empirical values given for the shear strength are conservative estimates of the average value according to DIN 1054.
b) Only characteristic values of $c_{u,k}$ are given in the table as the shear strengths in the unconsolidated condition. The corresponding friction angle shall be adopted as $\varphi_u = 0$.
c) Adoption of the empirical values given for the cohesion c'_k of the consolidated or drained soil and for the shear strength $c_{u,k}$ of the undrained soil is only permissible if it is certain that the consistency will remain unchanged or when an unfavourable change is prevented.
d) Adoption of the given bandwidths for the shear strength values assumes that the author of the draft and the technical planner posses expertise and experience in the geotechnical field. Otherwise, only the smallest values may be adopted.

A 5: Geotechnical categories of excavations

Table 5.1. Geotechnical categories of excavations

Geotechnical Category 1	Geotechnical Category 2	Geotechnical Category 3
Ground Conditions		
At least medium-dense or firm soils Stable rock	No allocation to GC 1 or GC 3 possible	Layers of alternating, irregular cohesive and cohesionless soils Organic soils Very soft and soft, cohesive soils Creep-prone soils Rock with unfavourably oriented fault zones or discontinuities Mining subsidence or doline regions Heterogeneos fill
Groundwater		
≥ 0.5 m below the excavation level	≤ 2.0 m above the excavation level Can be lowered using normal measures Is retained by sheet pile walls and unanchored bases	> 2.0 m above the excavation level Percolation around the retaining walls Horizontal/vertical permeability > 3.0 Settlement-prone soils in the zone of influence of groundwater lowering measures
Retaining wall		
Unanchored or propped sheet pile or soldier pile walls at up to 3 m excavation depth Standard support system to DIN 4124 Slopes up to 3 m Underpinning to DIN 4123 with a free height ≤ 0.5 m	Retaining walls up to 10 m excavation depth Retaining walls as bore pile walls and diaphragm walls Underpinning to DIN 4123 with a free height > 0.5 m	Excavations adjacent to deflection- and settlement-prone buildings Retaining walls that must display minor deflections Retaining walls with more than two rows of struts or anchors Retaining walls as support structure facilitated by soil stabilisation Percolation around retaining walls

Geotechnical Category 1	**Geotechnical Category 2**	**Geotechnical Category 3**
Sealing base		
No sealing base	Unanchored underwater concrete bases Practically impermeable soil	Anchored underwater concrete and stabilised earth bases Deep sealing bases
Tied back retaining walls		
Retaining walls not tied back	One ground anchor row maximum	Head of tie located below the groundwater table Tied back using nails in accordance with EN 14490 or using micropiles Tied back into underpinning bodies
Anchored back base		
Base not anchored	Base not anchored	All types of base anchorage

A 6: Partial safety factors for geotechnical variables

Table 6.1. Partial safety factors γ_F^1 and γ_E^2 for actions and effects

Action or effect	Notation	Design situation			
		DS-P	DS-T	DS-T/A	DS A
HYD and UPL: Limit state of failure by hydraulic heave and buoyancy					
Destabilising permanent actions[a]	$\gamma_{G,dst}$	(1.05)	1.05	1.05	1.00
Stabilising permanent actions	$\gamma_{G,stb}$	(0.95)	0.95	0.95	0.95
Destabilising variable actions	$\gamma_{Q,dst}$	(1.50)	1.30	1.15	1.00
Stabilising variable actions	$\gamma_{Q,stb}$	(0)	0	0	0
Seepage force in favourable subsoil	γ_H	(1.35)	1.30	1.25	1.20
Seepage force in unfavourable subsoil	γ_H	(1.80)	1.60	1.50	1.35
STR and GEO-2: Failure of structures, structural elements and the ground					
Effects of permanent actions in general[a]	γ_G	(1.35)	1.20	1.15	1.10
Effects of favourable permanent actions[b]	$\gamma_{G,inf}$	(1.00)	1.00	1.00	1.00
Effects of permanent actions from at-rest earth pressure	$\gamma_{G,E0}$	(1.20)	1.10	1.05	1.00
Effects of unfavourable variable actions in general[a]	γ_Q	(1.50)	1.30	1.20	1.1
Effects of favourable variable actions[b]	γ_Q	(0)	0	0	0
GEO-3: Limit state of failure by loss of overall stability					
Permanent actions[a]	γ_G	(1.00)	1.00	1.00	1.00
Unfavourable variable actions	γ_Q	(1.30)	1.20	1.10	1.00
SLS: Serviceability limit state					
γ_G = 1.00 for permanent actions or effects					
γ_G = 1.00 for variable actions or effects					
[a] including permanent and variable water pressure					
[b] only for determining the design value of the tensile load on piles if a simultaneously acting compressive load resulting from favourable permanent actions is adopted to determine the design values.					

[1] The coefficient γ_F is a generic for the respective, individual cases of the partial safety factors relative to the actions F.

[2] The coefficient γ_E is a generic for the respective, individual cases of the partial safety factors relative to the effects E.

EN 1990 prescribes that all partial safety factors for the DS-E design situation are defined as 1.0.

Table 6.2. Partial safety factors γ_R^3 for resistances in the STR and GEO-2 limit states

Resistance	Notation	Design situation			
		DS-P	DS-T	DS-T/A	DS A
STR and GEO-2: Failure of structures, structural elements and the ground					
Ground resistances					
– Passive earth pressure and bearing capacity	$\gamma_{R,e}$, $\gamma_{R,v}$	(1.40)	1.30	1.25	1.20
– Sliding resistance	$\gamma_{R,h}$	(1.10)	1.10	1.10	1.10
Pile resistances from static and dynamic pile testing					
– Toe resistance	γ_b	(1.10)	1.10	1.10	1.10
– Shaft resistance (compression)	γ_s	(1.10)	1.10	1.10	1.10
– Overall resistance (compression)	γ_t	(1.10)	1.10	1.10	1.10
– Shaft resistance (tension)	$\gamma_{s,t}$	(1.15)	1.15	1.15	1.15
Pile resistances based on empirical data					
– Compression piles	γ_b, γ_s, γ_t	(1.40)	1.40	1.40	1.40
– Tension piles	$\gamma_{s,t}$	(1.50)	1.50	1.50	1.50
Pull-out resistances					
– Soil and rock nails	γ_a	(1.40)	1.30	1.25	1.20
– Grouted body of ground anchors	γ_a	(1.10)	1.10	1.10	1.10
– Flexible reinforcement elements	γ_a	(1.40)	1.30	1.25	1.20

[3] The coefficient γ_R is a generic for the respective, individual cases of the partial safety factors relative to the resistance.

The partial safety factors for the material resistance of the steel tendon consisting of prestressing steel and reinforcing steel is given in DIN EN 1992-1-1 for the limit states GEO-2 and GEO-3 as $\gamma_M = 1.15$.
The partial safety factors for the material resistance of flexible reinforcement elements is given in EBGEO [170] for the limit states GEO-2 and GEO-3.
EN 1990 prescribes that all partial safety factors for the DS-E design situation are defined as 1.0.

Table 6.3. Partial safety factors γ_M for geotechnical parameters

Resistance	Notation	Design situation			
		DS-P	DS-T	DS-T/A	DS A
GEO-3: Limit state of failure by loss of overall stability					
Friction coefficient tan φ' of the drained soil and friction coefficient tan φ_u of the undrained soil	γ_φ, $\gamma_{\varphi u}$	(1.25)	1.15	1.13	1.10
Cohesion c' of the drained soil and shear strength c_u of the undrained soil	γ_c, γ_{cu}	(1.25)	1.15	1.13	1.10

A 7: Material properties and partial safety factors for concrete and reinforced concrete structural elements

Table 7.1. Characteristic material properties for normal strength concrete to EN 1992-1-1, Table 3.1

Concrete strength class C $f_{ck}/f_{ck,cube}$	C12/15	C16/20	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
For analysis of bearing capacity									
f_{ck} [N/mm^2]	12	16	20	25	30	35	40	45	50
For analysis of serviceability									
f_{ctm} [N/mm^2]	1.6	1.9	2.2	2.6	2.9	3.2	3.5	3.8	4.1
$f_{ctk;0.05}$ [N/mm^2]	1.1	1.3	1.5	1.8	2.0	2.2	2.5	2.7	2.9
$f_{ctk;0.95}$ [N/mm^2]	2.0	2.5	2.9	3.3	3.8	4.2	4.6	4.9	5.3
E_{cm} [N/mm^2]	27,000	29,000	30,000	31,000	33,000	34,000	35,000	37,000	38,000

f_{ck} characteristic compressive cylinder strength of concrete after 28 days
$f_{ck,cube}$ characteristic compressive cube strength of concrete after 28 days
f_{ctm} mean value of central tensile strength of the concrete
$f_{ctk;0.05}$ characteristic value of the 5% quantile of the central tensile strength of the concrete
$f_{ctk;0.95}$ characteristic value of the 95% quantile of the central tensile strength of the concrete
E_{cm} mean Young's modulus for normal strength concrete (secant at $|\sigma_c| \approx 0.4\ f_{cm}$)

Table 7.2. Characteristic material properties for reinforcing steel to DIN 488-1, excerpt from Table 2

Abbreviation	B500A	B500B	B500A	B500A	Quantile p (%) at $W = 1 - \alpha$ (one side)
Material number	1.0438	1.0439	1.0438	1.0438	
Surface	Ribbed	Ribbed	Smooth (+G)	Profiled (+P)	
Half product/ supply format	Reinforcing steel in rings, decoiled products, reinforcing steel mats, lattice girders	Reinforcing bar steel, reinforcing steel in rings, decoiled products, reinforcing steel mats, lattice girders	Reinforcing wire in rings and bars, lattice girders		
Yield point R_e[a] [N/mm^2]	500	500	500	500	5.0 at W = 0.90
Yield ratio R_m/R_e	1.05[b]	1.08	1.05[b]	1.05[b]	10.0 at W = 0.90
Ratio $R_{e,actual}/R_{e,nom}$	–	1.30	–	–	90.0 at W = 0.90
Total percentage strain for highest force A_{gt} [%]	2.5[b]	5.0	2.5[b]	2.5[b]	10.0 at W = 0.90

[a] The yield point (and tensile strength) is calculated from the force applied when the yield point (and greatest force) is achieved, divided by the nominal cross-sectional area ($A_n = \pi\, d^2/4$). The yield point is represented by the upper yield point R_{eH}. If there is no pronounced yield point, the 0.2% elongation $R_{p0.2}$ shall be determined.

[b] $R_m/R_e \geq 1.03$ and $A_{gt} \geq 2.0$ for nominal diameter 4.0 mm to 5.5 mm.

Table 7.3. Partial safety factors

According to EN 1992-1-1/NA, Table NA.2.1, supplemented according to R 24 and R 79

Action combination according to R 24	Design situation			
	DS-P	DS-T	DS-T/A	DS-A
γ_c for determining the bearing capacity of concrete[1)]	(1.50)	1.50	1.50	1.30
γ_s for determining the bearing capacity of reinforcing steel	(1.15)	1.15	1.15	1.00
γ_c and γ_s for analysis of serviceability	(1.00)	1.00	1.00	1.00

[1)] For in-situ concrete bored piles with recovered casing the partial safety factor is generally multiplied by the coefficient k_f. Where bored piles are produced to EN 1536, $k_f = 1.0$, in all other cases $k_f = 1.1$.

A 8: Material properties and partial safety factors for steel structural elements

Table 8.1. Characteristic material properties (nominal values)

In the sense of EN 1993-1-1 and EN 1993-5, for product thicknesses < 40 mm

Material standard and steel grade	Yield point f_y [N/mm^2]	Tensile strength f_u [N/mm^2]	Shear strength τ_R [N/mm^2]	Young's modulus E [N/mm^2]	Shear modulus G [N/mm^2]
EN 10025-2					
S 235	235	360	136		
S 275	275	430	159		
S 355	355	490	205		
S 450	440	550	254		
EN 10027				210 000	81 000
S240GP	240	340	139		
S270GP	270	410	156		
S320GP	320	440	185		
S355GP	355	480	205		
S390GP	390	490	225		
S430GP	430	510	248		

Table 8.2. Partial safety factors

According to EN 1993-1-1 and /NA, supplemented according to R 24

Action combination according to R 24	Design situation			
	DS-P	DS-T	DS-T/A	DS-A
γ_M for analysis of bearing capacity				
a) Capacity of cross-sections γ_{M0}	(1.00)	1.00	1.00	1.00
b) Capacity of structural elements for stability limit state γ_{M1}	(1.10)	1.10	1.10	1.00
c) Capacity of cross-sections for rupture limit state as a result of tensile loading γ_{M2}	(1.15)	1.15	1.25	1.15
d) To compute the stiffnesses	(1.00)	1.00	1.00	1.00
γ_M for analysis of serviceability	(1.00)	1.00	1.00	1.00

See EN 1993-1-8 for partial safety factors used when analysing the capacitty of connections.

A 9: Material properties and partial safety factors for wooden structural elements

Table 9.1. Characteristic values for the strength, stiffness and bulk density parameters for softwood

Excerpt from EN 338 for softwood. The given values are based on the use of new or practically new timber.

Strength class		C 16	C 24	C 30	C 35
Strength parameters in N/mm²					
Bending	$f_{m,k}$	16	24	30	35
Tension along grain against the grain	$f_{t,0,k}$ $f_{t,90,k}$	10 0.4	14 0.4	18 0.4	21 0.4
Compression along grain against the grain	$f_{c,0,k}$ $f_{c,90,k}$	17 2.2	21 2.5	23 2.7	25 2.8
Shear	$f_{v,k}$ 1)	2.0	2.0	2.0	2.0
Stiffness parameters in kN/mm²					
Mean value of Young's modulus along the grain against the grain	$E_{0,mean}$ 2) $E_{90,mean}$	8 0.27	11 0.37	12 0.40	13 0.43
Shear modulus	G_{mean}	0.50	0.69	0.75	0.81
Bulk density in kg/m³					
Bulk density	ρ_k	310	350	380	400
Mean value of bulk density	ρ_{mean}	370	420	460	480

1) The characteristic values of the shear strength are uniformly adopted at 2.0 N/mm² in accordance with EN 1995-1-1/NA, NDP, 6.1.7(2).

2) Mean value; the following design value applies for the 5% quantile: $E_{0,05} = 2/3 \cdot E_{0,mean}$

Table 9.2. Partial safety factors

According to EN 1995-1-1/NA, Table NA.2, supplemented according to R 24

Action combination according to R 24	Design situation			
	DS-P	DS-T	DS-T/A	DS-A
γ_M for analysis of bearing capacity	(1.30)	1.30	1.30	1.00
γ_M for analysis of serviceability	(1.00)	1.00	1.00	1.00

A 10: Empirical values for skin friction and base resistance of sheet pile walls

a) For driven sheet pile walls in cohesionless soils the characteristic empirical values for the base resistance $q_{b,k}$ from Table 10.1 and for the skin friction $q_{s,k}$ from Table 10.2 may be selected for the ultimate limit state analysis in accordance with R 84 (Section 4.8). See Figure R 85-1 for the areas to be adopted.
Note: The values given in Tables 10.1 and 10.2 are similar to the upper table values given for piles in [165] relative to the mean value (around the 50% quantile). Their adoption assumes that a certain vertical deflection of the sheet pile retaining wall can be accepted, also see R 85 (Section 13.10), Paragraph 5.

Table 10.1. Empirical values for characteristic base resistance $q_{b,k}$ of sheet pile walls in cohesionless soils

Mean cone resistance q_c of CPT in MN/m^2	Base resistance $q_{b,k}$ in the ultimate limit state in MN/m^2
7.5	7.5
15	15
≥ 25	20

Table 10.2. Empirical values for characteristic skin friction $q_{s,k}$ of sheet pile walls in cohesionless soils

Mean cone resistance q_c of CPT in MN/m^2	Skin friction $q_{s,k}$ in the ultimate limit state in kN/m^2
7.5	20
15	40
≥ 25	50

Intermediate values may be linearly interpolated.

b) Adoption of the given empirical values assumes the sections are driven. Otherwise, the following shall be observed:

- If the sheet piling is vibrated in the given empirical values for skin friction and base resistance shall be reduced to 75%.
- If the sheet piles are installed to the target depth with the aid of loosening bores or flushing lances, the base resistance and skin friction may only be adopted if confirmed by the geotechnical designer or geotechnical expert.

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Terms and notation

Geometrical variables

H	Excavation depth
H'	Distance between ground level and the end of the earth pressure redistribution
a	Load distribution width
a	Distance between centres
a_1	Clear span between plate anchors
d	Thickness of a load distributing layer
h_A	Height of first row of struts above the excavation level
s	Settlement
t	Embedment depth from the excavation level to the lower edge of the wall
t_0	Numerically required embedment depth below the excavation level with free-earth support
t_1	Numerically required embedment depth below the excavation level with fixed-earth support after *Blum*
t'_1	Numerically required embedment depth below the excavation level with partial support after *Blum*
t_B	Embedment depth utilised by the subgrade
z'	Height of the resultant support force in the ground below the excavation level
z_e	Height of the resultant above the toe of the pressure diagram
Δt_1	Embedment depth surcharge for restraint after *Blum*

Subsoil and soil parameters

c'	Cohesion intercept in terms of effective stress
c_c	Capillary cohesion in cohesionless soil
c_u	Undrained shear strength
q_s	Shaft resistance
γ	Weight density

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γ'	Effective weight density
γ_r	Saturated weight density
φ'	Angle of shearing resistance in terms of effective stress
equiv. φ_s	Equivalent friction angle for soft soils
$\varphi'_{Equiv.}$	Equivalent for determination of the minimum earth pressure

Earth pressure and passive earth pressure

E	Earth pressure force
E_0	At-rest earth pressure force
E_a	Active earth pressure force
E_P	Passive earth pressure force
mob E_p	Mobilised passive earth pressure force in the serviceability limit state
E_V	Residual at-rest earth pressure force below the excavation base
K_0	At-rest earth pressure coefficient
K_a	Active earth pressure coefficient
K_p	Passive earth pressure coefficient
e	Earth pressure ordinate
e_0	At-rest earth pressure ordinate
e_a	Active earth pressure ordinate
e_p	Passive earth pressure ordinate
g	Index for soil self-weight
h	Index for horizontal component
v	Index for vertical component
δ_0	Angle of at-rest earth pressure
δ_a	Angle of active earth pressure
δ_p	Angle of passive earth pressure
ϑ_a	Angle of planar slip surface for active earth pressure
ϑ_p	Angle of planar slip surface for passive earth pressure
ϑ_z	Angle of an imposed planar slip plane

Further loads, forces and action effects

B	Resultant reaction force/ground reaction in the ground support
B_{Bh}	Resultant reaction force from the soil stresses in the ground support
C	Equivalent force after *Blum*
G	Dead load
H	Horizontal force
M	Bending moment
P	Load on an anchorage
Q	Variable action
Q	Resultant in the slip plane
V	Vertical force
p	Unbounded distributed load $\leq 10\ kN/m^2$
q	Component of unbounded distributed loads over and above $p = 10\ kN/m^2$
q′	Strip load
$\overline{q}$	Line load
δ_C	Angle of equivalent load after *Blum*
σ_{ph}	Horizontal component of the ground reaction stress (distribution of reaction force)
σ_B	Subgrade stresses in the ground support

Analyses using the partial safety factor approach

F	Action
E	Effect
G	Index for permanent action
Q	Index for unfavourable, changeable action
R	Resistance
d	Index for design values
k	Index for characteristic values
η_{Ep}	Calibration factor for passive earth pressure
μ	Utilisation factor
γ_E	Partial safety factor for an effect

γ_F Partial safety factor for an action

γ_m Partial safety factor for a soil parameter (material property)

γ_R Partial safety factor for a resistance

Miscellany

The term *pressure diagram* is used where reference is made to the earth pressure distribution on the retaining wall only; *load model*, in contrast, is used where retaining wall support through strut or anchor forces, and ground reactions is being described.

The notations of various terms, especially those widely recognised, are also adopted as indexes.

Recommendations in numerical order

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