Recommendations on Excavations EAB

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EAB

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Preface*

In response to a clearly overwhelming requirement, the *Deutsche Gesellschaft für Erd- und Grundbau e. V.* (German Society for Geotechnical and Foundation Engineering) called the Working Group for Tunnel Engineering into life in 1965 and transferred the chairmanship to the highly respected and now sadly missed Prof. *J. Schmidbauer*. The wide-ranging tasks of the Working Group were divided into three sub-groups "General", "Open Cut Methods" and "Trenchless Technology". The "Open Cut Methods" Working Group, under the chairmanship of the author, at first busied itself only with the urgent questions of analysis, design and construction of excavation enclosures. The German Society for Geotechnical and Foundation 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 analysis, design and construction of excavation enclosures, it was recognised that these matters were so comprehensive that the German Society for Geotechnical and Foundation Engineering 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 first publication of the new Working group appeared with the title "Recommendations of the Working Group for Excavations" in the journal "*Die Bautechnik*" (Construction Technology) in 1970. It was based on a thorough reworking, restructuring and enhancement of the proposals published in 1968 and consisted of 24 numbered Recommendations, which primarily dealt with the basic principles of the analysis of excavation enclosures, analysis of soldier pile walls, sheet pile and in-situ concrete walls for excavations, and with the impact of buildings beside excavations.

In the years following this, the Working Group for Excavations published new and reworked Recommendations in two-year periods. 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 one single volume.

In the 2nd (German) edition, published in 1988, the Recommendations were partly reworked and, in addition, supplemented by nine further Recommendations dealing with "Excavations in Water", which were 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 from endeavours

* The Preface refers to the 4th German Edition.

to make the Recommendations more easily understandable. The alterations and supplements are described in an article in the 1989 volume of *Bautechnik*.

In the 3rd (German) edition, published in 1994, a number of the Recommendations were reworked and three new Recommendations on “Excavations with Special Ground Plans” added. The modifications 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. Furthermore, an appendix was included, containing the principal construction supervision regulations, where they are relevant to stability analysis.

At the same time that the 3rd (German) 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 foundation engineering. On the one hand this was because several members of the Working Group for Excavations were also represented in the “Safety in Geotechnical and Foundation Engineering” Committee, which was compiling the 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 foundation engineering structures. In particular the specification in the new draft European regulations EN 1997-1, prescribing two analyses was unacceptable. This applied partial safety factors to the shear strength on one side and to the actions on the other. Compared to previously tried and tested practice it produced results that in part led to considerably greater dimensions but also to results that were too liberal. 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, and to the earth pressure and soil resistances. In the EAB-100, published in 1996 together with 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 the 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 the DIN 1054-100 was modified such that the originally envisaged superpositioning of earth pressure design values 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 internal forces 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. This 4th (German) edition of the EAB rests entirely upon these points, but also expands them by supplementary regulations, just as it has in the past. Moreover, all the Recommendations of the 3rd (German) edition have been subjected to thorough reworking. 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 this issue.

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

a) simplify analysis of excavation enclosures;
b) unify load approaches and analysis procedures;
c) guarantee the stability of the excavation structure and its individual components and;
d) guarantee the economic design of the 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. Weissenbach

Contents

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 importance. A duty to apply the Recommendations may result from legislative or administrative provisions, contractual obligations or from further legal provisions.
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 advanced or 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 on 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 his or her own risk.

1 General Recommendations

1.1 Engineering requirements 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 so that intimate contact with the ground is ensured. The infilling or lining can consist of wood, concrete, steel, hardened cement-bentonite suspension or stabilised soil. It shall be installed so that the contact to the soil is as uniform as possible. Soil excavation should not advance considerably faster than piling advance.
3. Sheet pile walls and trench sheet piles are installed so 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. See DIN 1538 for execution of diaphragm walls. For bored pile walls proceed according to DIN EN 1536. 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 so 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 adoption of the remaining Recommendations. However, the consequences of any deviations shall be investigated and taken into consideration.

1.2 Governing regulations (R 76)

1. In the long term a considerable proportion of current German standards relating to structural engineering will be replaced by European standards. They were initiated in the shape of Eurocodes by what was then the Commission of the European Community and further developed under the support of the European Committee for Standardisation (Comité Européen de Normalisation, CEN). Although these Eurocodes, represented by the EN 1997 “Draft, Geotechnical Design” for geotechnical and foundation engineering, have

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meanwhile achieved a considerable degree of maturity, their introduction as building regulations is not yet envisaged at the time of publication of the 4th German edition of the EAB.

2. The new generation of national standards based on the partial safety factor approach serves as a temporary solution for all fields of structural engineering until the introduction of the Eurocodes. All standards mentioned refer to the latest version using the partial safety factor approach. Year and month data are not provided here. The following codes in particular are the governing standards for excavation structures:

DIN 1055-100	"Basis of design"
DIN 1054	"Verification of the safety of earthworks and foundations"
DIN 18 800	"Steel structures" including the steel structure adaptation directive
DIN 1045	"Concrete, reinforced and prestressed concrete structures"
DIN 1052	"Timber structures"
DIN 1055-2	"Soil properties"

3. DIN 1054 only regulates fundamental questions of geotechnical and foundation 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 represent the governing standards for excavation structures:

DIN 4084	"Calculation of embankment failure and overall stability of retaining structures"
DIN 4085	"Calculation of earth-pressure"
DIN 4126	"Stability analysis of diaphragm walls"

4. The existing standards covering the exploration, investigation and description of ground are not affected by the adaptation to partial safety factors and therefore remain valid in their respective latest editions:

DIN 4020:	Geotechnical investigations for civil engineering purposes
DIN 4021:	Exploration by excavation and borings
DIN 4022:	Designation and description of soil and rock
DIN 4023:	Graphical presentation of logs and boreholes
DIN 4094:	Investigation by soundings (Part 3 replaced by EN ISO 22 476-2:2005)
DIN 18 121 to DIN 18 137:	Soil investigation and testing
DIN 18 196:	Soil classification for civil engineering purposes

5. DIN 1054 only replaces the analysis section of the previous standards DIN 4014 "Bored piles", DIN 4026 "Driven piles", DIN 4125 "Ground anchorages, temporary and permanent anchorages" and DIN 4128 "Injection

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:

DIN EN 1536:	Bored piles
DIN EN 1537:	Ground anchors
DIN EN 1538:	Diaphragm walls
DIN EN 12 063:	Sheet pile walls
DIN EN 12 699:	Displacement piles
DIN EN 12 715:	Grouting
DIN EN 12 716:	Jet grouting
DIN EN 12 794:	Precast concrete foundation piles
DIN EN 14 199:	Micropiles

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

DIN 4095:	Drainage for the protection of structures
DIN 4123:	Excavations, foundations and underpinnings in the range of existing buildings
DIN 4124:	Excavations and trenches

7. Until all relevant technical building regulations, standards and recommendations are adapted to the partial safety factor approach, the transitional regulations given in DIN 1054, Appendices F and G, apply.

1.3 New safety factor approach (R 77)

1. In contrast to the original probabilistic safety factor approach, the new 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 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; characterised by the index "k". Characteristic values are generally specified based on testing, measurements, analyses or empiricism.

3. If the bearing capacity in a given cross-section of the retaining wall or in an interface between the retaining wall and the ground 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 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' and c'; for unconsolidated soils or soils not drained for testing the shear parameters φ_u and $c_{u.}$ 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 for effects are multiplied by partial safety factors; the characteristic values for resistances are divided by partial safety 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". Three limit states are differentiated for stability analyses, according to R 78 (Section 1.4).

1.4 Limit states (R 78)

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

   a) In soil mechanics, the state in 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, embankment stability and overall stability of retaining structures.
   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 using the new 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. It is known as the ultimate limit state (ULS) in DIN 1054. Three 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. It is known as the serviceability limit state (SLS) in DIN 1054.

3. The EQU limit state describes the loss of static equilibrium. These include:

   – analysis of safety against failure by toppling;
   – analysis of safety against hydraulic failure by uplift (buoyancy);
   – analysis of safety against hydraulic failure by heave.

   The EQU limit state incorporates actions, but no resistances. The decisive 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 become as large as the stabilising action G_k, multiplied by the partial safety factor $\gamma_{stb} < 1$.

4. The STR limit state describes the failure of structures and structural elements or failure of the ground. These include:

   – analysis of the bearing capacity of structures and structural elements subject 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 soil is not exceeded is performed exactly as for any other construction material. The limit state condition is always the decisive 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 loads, may only become as large as the characteristic resistance R_k, divided by the partial safety factor γ_R.

5. The GEO limit state is peculiar to geotechnical engineering. It describes the loss of overall stability. These include:

   – analysis of safety against embankment failure;
   – analysis of overall stability of retaining structures.

   The limit state condition is always the decisive condition:

   $E_d \leq R_d$

   i.e. the load design value E_d may only become as large as the design value of the resistance 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 $c_{u,d} = c_{,k} / \gamma_c$

   i.e. the friction $\tan \varphi$ and the cohesion c are reduced at the beginning using the partial safety factors γ_φ and γ_c.

6. The serviceability limit state SLS 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 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) Ground anchors are tested according to EN 1537, Method 1, and are prestressed to at least 80% of the computed force required for the next construction stage.

c) A tight connection via frictional contact is established with displacement piles (previously "driven piles"), bored piles or micropiles (previously "grouted piles"), which verifiably display only 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 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 reach into 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 Using the EAB in conjunction with Eurocode 7-1 (R 105, draft)

1. This edition of the EAB is based on the specifications provided in DIN 1054 (2005). This publication in turn was closely harmonized with EN 1997-1 – Eurocode EC7-1. DIN 1054 is not identical in every detail with Eurocode EC7-1, but neither does it contradict it. As soon as Eurocode EC7-1 can be adopted, with the permission of the responsible authorities, DIN 1054 must at least be formally adapted to the specifications of Eurocode EC-7. The consequences associated with this for applying this edition of the Recommendations are related below as well as a preview will allow.

2. The following stipulations have been agreed upon in terms of the validity of the regulations:

   a) Once the DIN 1054 (2005) has been included in the model list of the Acknowledged Technical Rules for Works (*Technische Baubestimmungen*), it can be introduced by the responsible authorities of the federal states during 2005 and 2006. The end of the validity period of DIN 1054 (1976) is given as the end of 2007 in the model list.
   b) A two-year transition period began at the end of 2004; during this period a national annex to the Eurocode EC7-1 was to be compiled and published jointly with the Eurocode, and approved for use on the basis of European agreements.
   c) In addition, at the end of 2004 a five-year transition period began, at the end of which Eurocode EC7-1 was to be introduced by the responsible authorities and all contradictory national regulations be withdrawn.

d) The end of the validity period of DIN 1054 (2005) is fixed at the end of 2009 by the stipulations of Paragraph c).

The competent responsible authorities are:

– the higher building control authorities of the federal states for building measures subject to the respective state building code;
– the departments of the Federal Ministry of Transport, Building and Urban Affairs (*Bundesministerium für Verkehr, Bau- und Stadtentwicklung* (*BMVBS*)) responsible for inland waterways, for federal roads and road bridges, and the Federal Railway Authority (*Eisenbahn-Bundesamt*) responsible for rail traffic.

3. In terms of the STR limit state safety analyses according to R 78, Paragraph 4 (Section 1.4), Eurocode EC7-1 provides three options. DIN 1054 (2005) is based on analysis procedure 2 inasmuch as the partial safety factors are applied to the loads and to the resistances. To differentiate between this and the other scenario, in which the partial safety factors are not applied to the loads but to the actions, this procedure is designated as analysis method 2* in the Commentary to Eurocode EC7-1 [134]. DIN 1054 also utilises a number of gaps that are not specifically codified, e.g. using load cases according to R 79, Paragraph 1 (Section 2.4).

4. The National Annex represents a formal link between Eurocode EC7-1 and national standards. This National Annex states which of the possible analysis methods and partial safety factors are applicable in the respective national domains. Remarks, clarifications or supplements to Eurocode EC7-1 are not permitted. However, the applicable, complementary national codes may be given. The complementary national codes may not contradict Eurocode EC7-1. Moreover, the National Annex may not repeat information already given in Eurocode EC7-1.

5. The reworked DIN 1054 will be paramount in the complementary national code; it has the working title "DIN 1054 (2007)" and is the application rule to Eurocode EC7-1. It is likely that the following points will differ from the DIN 1054 (2005) edition:

   – where feasible it will be shortened to avoid the problem of repetitions;
   – from a formal point of view it will be more closely adapted to Eurocode EC7-1;
   – it will include supplements, improvements and modifications.

   The supplements, improvements and modifications shall be adhered to inasmuch as they affect the regulations of the EAB, if the respective excavation structure is designed to Eurocode EC7-1. However, they may also be accordingly utilised if the design is based on DIN 1054 (2005).

6. In the governing version, Eurocode 7-1 defines the following limit states in place of the GZ 1A, GZ 1B and GZ 1C limit states:

a) EQU: loss of equilibrium of the structure, regarded as rigid, without the influence of soil resistances. The designation is derived from “equilibrium”.
b) STR: inner failure or very large deformation of the structure or its components, whereby the strength of the materials is decisive for resistance. The designation is derived from “structure”.
c) GEO: failure or very large deformation of the ground, whereby the strength of the soil or rock is decisive for resistance. The designation is derived from “geotechnical”.
d) UPL: loss of equilibrium of the structure or ground due to uplift (buoyancy) or water pressure. The designation is derived from “uplift”.
e) HYD: hydraulic failure by heave, inner erosion or piping in the ground, caused by a flow gradient. The designation is derived from “hydraulic”.

7. In order to transfer the GZ 1B and GZ 1C limit states to the terminology used in EC7-1 the GEO limit state is divided into GEO B and GEO C:

   a) GEO B: failure or very large deformation of the ground in conjunction with identification of the action effects and dimensions; i.e. when utilising the shear strength for passive earth pressure, for sliding resistance and bearing capacity and when analysing stability in the low failure plane.
   b) GEO C: 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 embankment failure and overall stability of retaining structures, generally, when analysing the stability of engineered slope stabilisation measures.

8. The previous limit states are now 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 B 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 stability in the low failure plane.
   c) The previous limit state GZ 1C corresponds to the GEO C limit state in conjunction with analysis of overall stability, i.e. when utilising the shear strength for analysis of safety against embankment failure and overall stability of retaining structures.

   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 using the regulations of the GEO B limit state;
   – or in the sense of the previous limit state GZ 1C using the regulations of the GEO C limit state.

2 Analysis principles

2.1 Actions (R 24)

1. DIN 1055-100 and DIN 1054 differentiate between permanent and variable actions. In excavation structures the permanent actions include:

   – weight density of the excavation structure, if necessary taking provisional bridges and excavation covers into consideration;
   – earth pressure as a result of the weight density of the soil, if necessary taking cohesion into consideration;
   – earth pressure as a result of the permanent weight density of neighbouring 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.

   DIN 1054 also stipulates that, simplified, the earth pressure resulting from a variable, unbounded distributed load $p_k \leq 10$ kN/m^2 can be 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:

   – centrifugal, brake and nosing forces, e.g. for excavations beside or below railway or tram lines;
   – exceptional loads and improbable or rarely occurring combinations of loads or points of application of loads;

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- 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 flooded;
- the influence of temperature on struts, e.g. steel H-section struts without buckling protection devices or struts in narrow excavations with frost-sensitive ground.

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 due to scouring in front of the retaining wall.

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

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

2.2 Determination of soil properties (R 2)

1. In principle, the soil properties required for stability analyses are the immediate result of geotechnical investigations based on DIN 4020 "Geotechnical Investigations for Civil Engineering Purposes". To take the heterogeneity of the ground 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 the 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 STR and GEO 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 (buoyancy), safety against hydraulic heave and safety against lift-off, which are all incorporated in the EQU limit state, the lower characteristic values are the decisive values.

3. Characteristic values for shear strength should be selected as conservative estimates of the statistical mean value. Minor deviation from the mean value may be acceptable if the available samples are sufficiently representative of the soil in the region of the excavation structure to be analysed. A larger deviation shall be assumed for a small data pool and heterogeneous ground.
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 ground can be greatly reduced by hair cracks, slickensides 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 decisive proportion of montmorillonite, the residual shear strength may be the decisive 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 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 18 196 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 of packing. See Appendix A 1 for classification of soils in terms of relative density of packing.
b) The given empirical values apply to both natural ground and man-made, cohesionless soil-layers. 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 δ_a and δ_p 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 principally 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) Generally, the untreated surfaces of steel, concrete and wood can be regarded 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 if the standing time of the mud-supported trench is kept short according to the general stipulations for manufacture.
d) All rear wall faces should be classified as “smooth” if the soil 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 analysed 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	$\lvert\delta_k\rvert = \varphi'_k$	$\lvert\delta_k\rvert \leq {}^2/_3 \cdot \varphi'_k$
Rough wall	$30° \geq \lvert\delta_k\rvert \leq \varphi'_k - 2.5°$	$\lvert\delta_k\rvert \leq {}^2/_3 \cdot \varphi'_k$
Slightly rough wall	$\lvert\delta_k\rvert \leq {}^1/_2 \cdot \varphi'_k$	$\lvert\delta_k\rvert \leq {}^1/_2 \cdot \varphi'_k$
Smooth wall	$\lvert\delta_k\rvert = 0$	$\lvert\delta_k\rvert = 0$

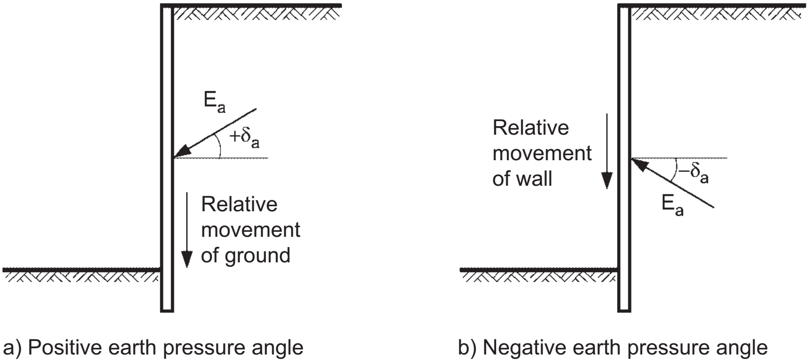

Figure R 89-1. Angle for active earth pressure

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 $\varphi'_k \leq 35°$.

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).
   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. See also Figure R 19-1 (Section 6.3).

2.4 Partial safety factors (R 79)

1. The magnitude of the partial safety factors depends in principle on the load cases, which are defined as a function of the combinations of actions, and the safety classes given in DIN 1054. Excavation structures are classified as Safety Class SC 2 and as Load Case LC 2 in conjunction with the loads in standard Action Combination AC 1, and as Load Case LC 3 in conjunction with the loads for accidental actions in Action Combination AC 3. Based on this the actions given in R 24 (Section 2.1) are allocated as follows:

a) The standard case according to R 24, Paragraph 3 corresponds to Load Case LC 2.
b) The special case according to R 24, Paragraph 4 corresponds to Load Case LC 2/3.
c) The exceptional case according to R 24, Paragraph 5 corresponds to Load Case LC 3.

2. The partial safety factors for actions for Load Cases LC 2 and LC 3 are based on DIN 1054. The partial safety factors for Load Case LC 2/3 are interpolated from these. 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 EQU limit state;
   – in Table 6.3 for resistances in the GEO limit state.

   The partial safety factors for Load Case LC 2/3 are interpolated between those of Load Cases LC 2 and LC 3, similar to those of the actions.

6. The numerical values for Load Case LC 1 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 stability of the lower failure plane according to R 44, Paragraph 10 (Section 7.3), in excavations adjacent to structures;
   – analysis of embankment stability and overall 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).

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.

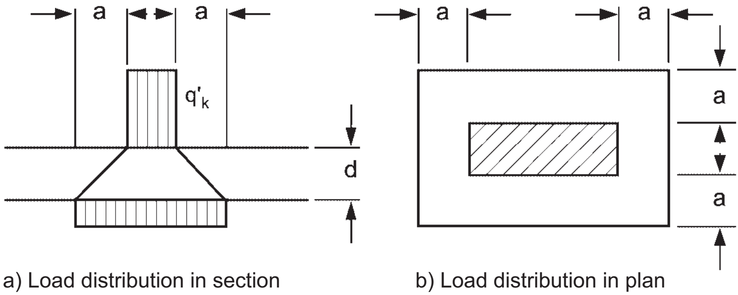

Figure R 3-1. Load distribution in the upper road layers

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. Load transfer in all directions within the upper road layers may be assumed as shown in Figure R 3-1 as follows, dependent 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 relevant technical regulations and guidelines for base courses in highway engineering for base course quality requirements.

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

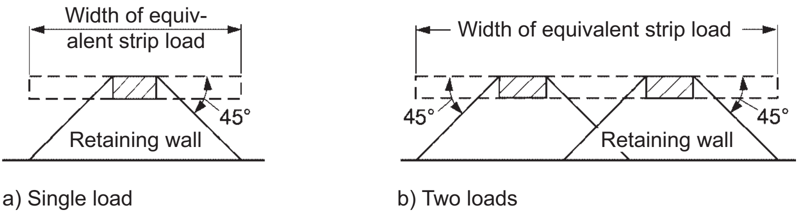

Figure R 3-2. Conversion of bounded surcharge loads to strip loads

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 3-2 b).

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. Reinforcement of the lagging of soldier pile walls on the opposite side is not necessary.

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

1. According to the German Road Transport Licensing Regulations (*StVZO*) of 07. February 2004, 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) = 10 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.

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.

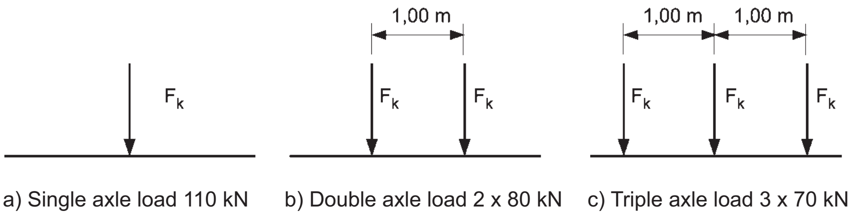

Figure R 55-1. Standard axle loads

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 direct 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 area of provisional bridges.

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

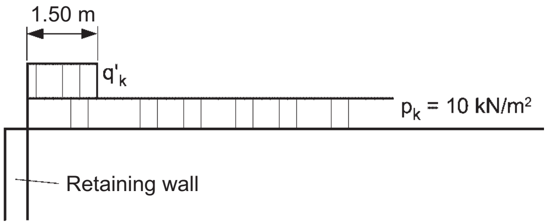

Figure R 55-2. Equivalent loads for road traffic at less than 1.00 m from the retaining wall

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 relevant regulations for bridges apply and those of the appropriate transport service provider for rail traffic, unless DIN Technical Report 101 is prescribed or agreed upon by contract. Where no other regulations are specified, it is sufficient to apply a bridge loading classification of 30/30 according to DIN 1072. On multiple-lane provisional bridges and excavation covers where vehicles which require a special permit due to their axle loads or total gross weight, and which are therefore not commonly licensed, it is generally sufficient to provide one specially designated lane for this purpose designed according to bridge loading classification of 60/30.
8. If analysis is based on DIN 1072 the loads given there are to be 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 according to DIN 1055-1 shall be carried out. The same applies to silo loads.
2. When applying equivalent loads for vehicles licensed for general public roads, such as trucks, 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 directly 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 Technical Report 101 applies 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 beside each other in neighbouring lanes, this shall be taken into consideration. Simplified loads may be adopted for vehicles that are commonly licensed, based on a bridge loading classification of 30/30.

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, besides weight density 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. See also 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 area of tracked equipment is taken from the manufacturer's specifications.
   b) The contact area of rubber-tyred equipment is 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 decisive excavator and lifting equipment distances from the retaining wall and all decisive 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 each 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 pointing perpendicular to the travel direction:
40% of the total load at each of 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 directly 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 p′, 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 Load Case 2 (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 contact area of at least 0.25 m^2 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 decisive distance.
c) If the road surface is metalled, load distribution at 45° from the rear edge of the equivalent load may be assumed.

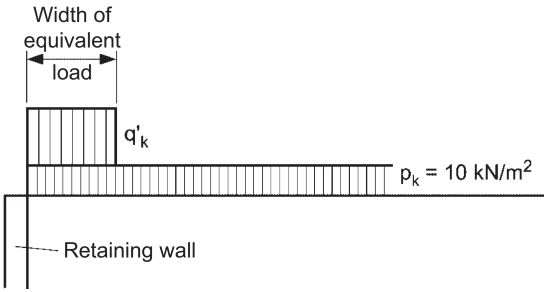

Figure R 57-1. Equivalent load for excavators and lifting equipment

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 decisive location and in the decisive 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 the current regulations for bridges.
   f) Where appropriate, consideration shall be given to loads from site traffic, which occur simultaneously to loads from excavators and lifting equipment, according to R 56, Paragraph 4 (Section 2.7).

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 is highly dependent on the degree of deflection and deformation of the retaining wall as a result of material excavation. The decisive 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 is generally smaller than the at-rest earth pressure 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 can 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 restrained in the ground, regardless of the types of soil present.

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

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

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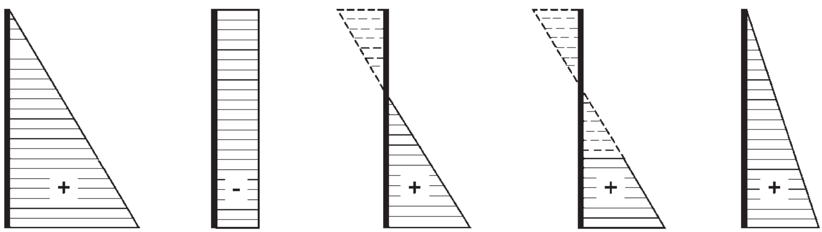
3.2 Magnitude of active earth pressure without surcharge loads (R 4)

1. The magnitude of the active earth pressure E_a from soil weight density 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 angle are adhered to. Otherwise, curved slip surfaces shall be used. This also applies to stratified soils.
2. The characteristic wall friction 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, wall friction 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 non-supported or yieldingly supported retaining walls, which rotate around the toe of the wall or a deeper point, the earth pressure 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).

   The decisive minimum earth pressure is the larger of the earth pressures.

   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 $\varphi'_{Equiv,k} = 45°$.



a) Earth pressure from soil self-weight
b) Earth pressure resulting from cohesion
c) Earth pressure for unsupported retaining walls
d) Earth pressure for supported retaining walls
e) Minimum earth pressure

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

4. The procedure is as follows for stratified soil:

   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 decisive values for determining the earth pressure of the strata 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 with the equivalent friction angle $\varphi'_{Equiv,k}$, as shown in Figure R 4-2 c).

   The decisive minimum earth pressure is the larger of the earth pressures of the respective strata. The total earth pressure is the result of addition of the decisive earth pressures of the individual strata.

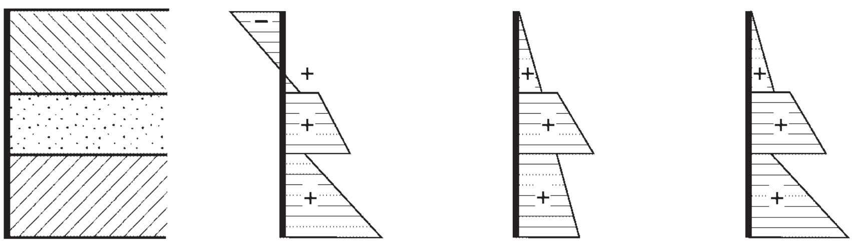
5. For slightly yieldingly supported retaining walls, where earth pressure redistribution is anticipated due to the prevailing conditions, the computed tensile stresses resulting from the characteristic shear strengths according to R 2 (Section 2.2) due to cohesion may be taken completely into consideration 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 shall be determined with the equivalent friction angle according to Paragraph 3 b). The larger value is the decisive minimum earth pressure.

   b) In stratified soil the earth pressure 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 decisive earth pressure is the larger



a) Stratified ground
b) Earth pressure with characteristic shear strengths
c) Earth pressure in the cohesive layers with equivalent friction angle
d) Minimum earth pressure

Figure R 4-2. Determination of active earth pressure load for partially cohesive soils of the earth pressures of the respective strata. The total pressure corresponds to the minimum earth pressure as shown in Figure R 4-2 d).

6. In cohesive soils and rocky ground, local experience should be examined 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, over and above that determined for the respective soil properties. In addition, where rocky ground is involved, it should be established whether bedding or joints predetermine certain slip surfaces, which influence the magnitude of the earth pressure. See also R 38 (Section 11.1).

7. 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 determined on the basis of Paragraphs 1 to 4.

8. By applying model tests and taking measurements on previously executed excavations (see [69, 73]) it has been demonstrated that under certain circumstances a portion of the earth pressure can be redistributed to below the excavation level when using flexible retaining walls, with the result that the earth pressure acting above the excavation level is smaller than the computed active earth pressure E_a. 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 set of struts of 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.

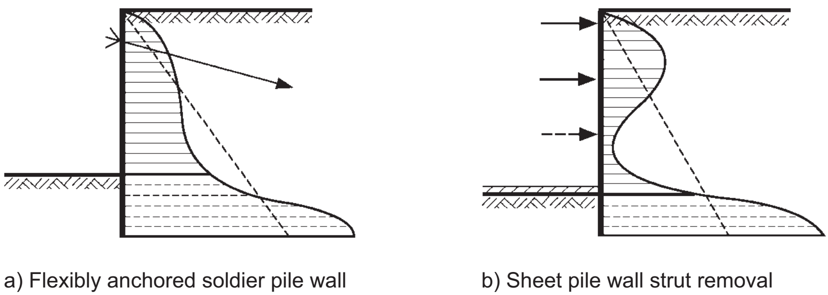


Figure R 4-3. Possible earth pressure redistribution in the region below the excavation level for flexible retaining walls

See R 68 (Section 3.8) for earth pressure reductions in retreating states.

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

3.3 Distribution of active earth pressure load 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 in the course of excavation progress, associated with parallel deflection and bending. 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 decisive 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′, in 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. It is at the height of the upper support in double-propped walls, 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 weight density 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 for sheet pile walls, diaphragm walls and pile walls is generally below half of the distance from ground level to the selected end of earth pressure redistribution.
f) This applies to medium-dense to dense soils. Loosely compacted 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 densely compacted, 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 more resemble either that of a medium-dense or that of a loosely compacted, cohesionless soil. The clay content and sensitivity are decisive in this respect.
c) In soft cohesive soils the earth pressure redistribution is at most equal to that of loosely compacted cohesionless soil, but is often either lower or does not occur at all. See Section 12.

The following apply with reservations:

– for soils whose behaviour can be impaired by hair cracks, slickensides, joints or intercalations of slightly cohesive or cohesionless soils;
– for 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 compels 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. The bending points and load increments of the selected pressure diagrams may be located at the support points to simplify analysis. If the preconditions given there are fulfilled, the 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 analyse using two pressure diagrams, which restricts the possible earth pressure distribution. The most unfavourable action effects are always decisive for the design of individual components.

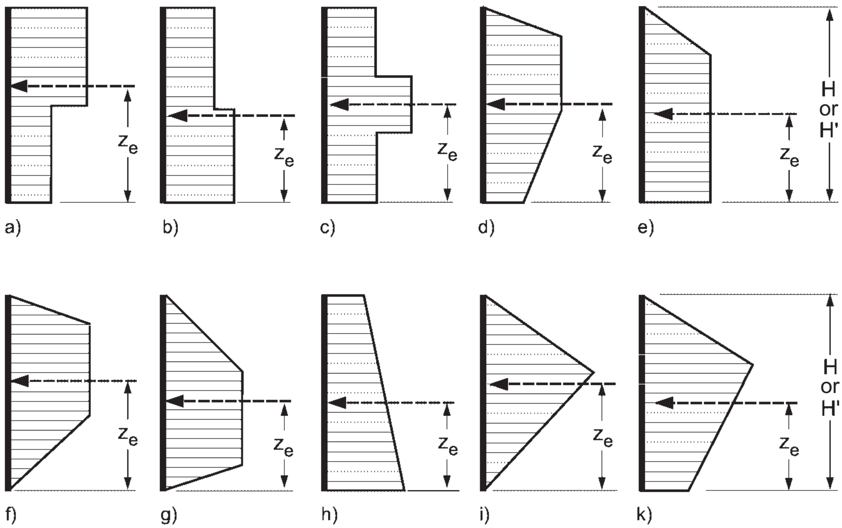


Figure R 5-1. Pressure diagrams for supported retaining walls (examples)

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

1. Determination of the earth pressure E_a from vertical, changeable loads may generally be based on the same wall friction angle $\delta_{a,k}$ as for determination of the earth pressure from soil weight density. See also R 4, Paragraph 2 (Section 3.2).
2. The magnitude of the earth pressure 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 from soil weight density.
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 an angle $\vartheta_{a,k}$, which is decisive for determination of the earth pressure from soil weight density, may be used to approximately determine the earth pressure 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. If the earth pressure from soil weight density and, if applicable, cohesion according to R 4, Paragraph 3 (Section 3.2) is determined for cohesive soil strata with the aid of an equivalent friction angle, it may also be used to determine the earth pressure from unbounded, vertical, distributed loads up to $p_k = 10$ kN/m^2. The earth pressure from line loads and strip loads, on the other hand, must always be determined according to Paragraph 3. Where

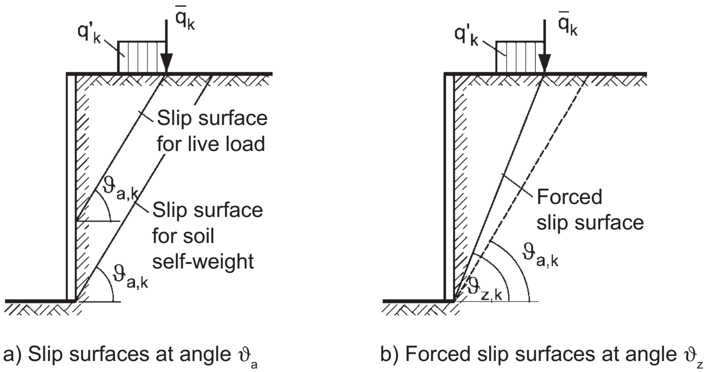


Figure R 6-1. Assumed slip surfaces for determination of the total active earth pressure from soil weight density and live loads

appropriate, this is also the case in principle for unbounded distributed loads q_k, inasmuch as this approach is selected in deviation to the information in Sections 2.6 to 2.8.

5. To determine the active earth pressure from line or strip loads for unsupported retaining walls which are merely restrained in the ground, or for yieldingly supported retaining walls, a forced slip surface shall also be investigated. This 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 weight density and live loads determined in this way is decisive for 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 weight density a, 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. The earth pressure E_{aHh} from horizontal line or strip loads H is adopted in particular for slightly yielding walls at:

   $E_{aHh} = H$

   For unsupported or yieldingly supported walls the earth pressure 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 $\vartheta_{z,k}$ are adopted for the slip surface angle ϑ_a.

7. For determination of the earth pressure with surcharges under various boundary conditions see R 71 (Section 3.6).

8. For determination of earth pressure from building loads see R 21 to R 23 and R 28 to R 29 (Sections 9.2 to 9.6).

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 distribution of earth pressure:

   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 yieldingly supported 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 $\overline{q}_k$ can 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 forced slip surfaces according to R 6, Paragraph 5 (Section 3.4), the pressure diagram ends at the intersection of the forced slip surface with the rear of the wall.

3. The shape of the pressure diagram can be specified as follows for unsupported or yieldingly 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 as shown in 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 as shown in Figure R 7-1 b).

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

4. For slightly yieldingly 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 of the 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.

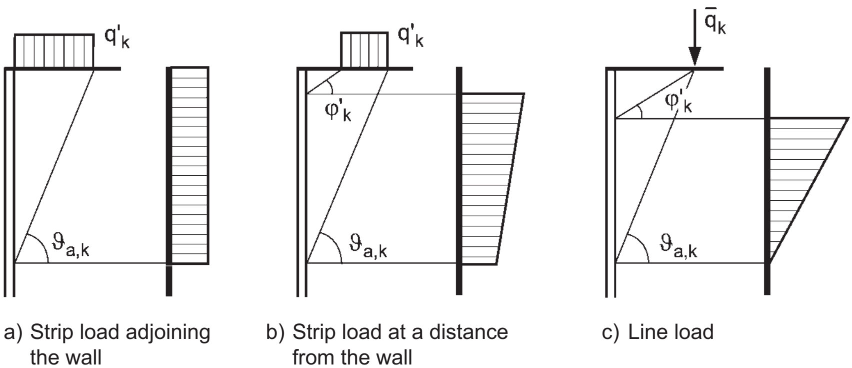
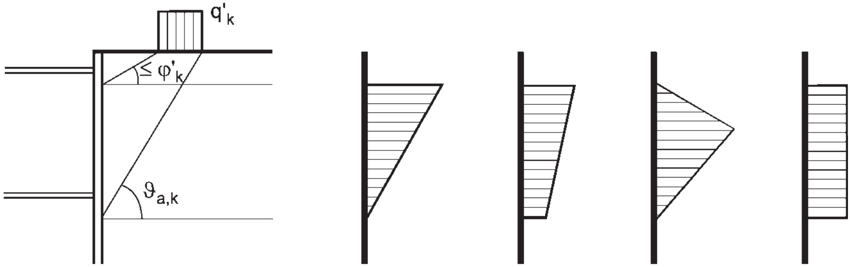


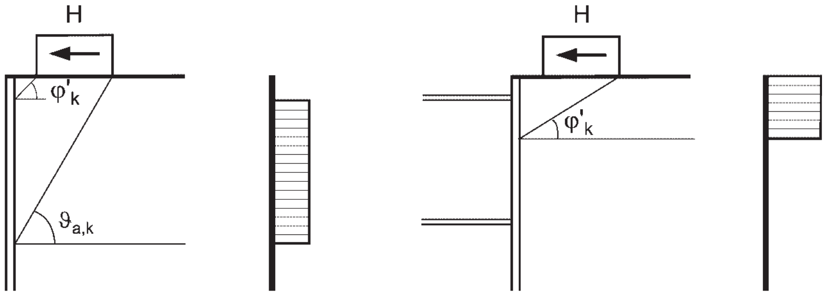
Figure R 7-1. Pressure diagrams for the earth pressure from vertical live loads for unsupported or yieldingly supported walls



a) Retaining wall, live load and load distribution

b) Possible simple pressure diagrams (examples)

Figure R 7-2. Pressure diagrams for the earth pressure from vertical live loads for slightly yielding walls



a) Unsupported or yieldingly supported wall

b) Slightly yielding wall

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

5. In principle, the distribution of the earth pressure from horizontal line and strip loads may be adopted in the same manner as the corresponding vertical load. This produces the pressure diagrams shown in Figure R 7-3 for a bounded strip load. The procedure for forced slip planes is analogous to vertical surcharges.
6. For determination of the earth pressure with surcharges under various boundary conditions see R 71 (Section 3.6).
7. For the distribution of earth pressure from building loads see R 28 and R 29 (Sections 9.3 and 9.4).

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 weight density, unbounded distributed load p_k and, where applicable, cohesion on the one hand and a locally acting strip load q_k or line load $\overline{q}_k$ on the other, may be determined separately and used to determine the action effects. In contrast, for unsupported retaining walls restrained in the ground, 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 forced 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 the angle $\vartheta_{a,k}$;
   b) the earth pressure components and pressure diagrams as shown in Figure R 71-2, adopting slip surfaces at the angle $\vartheta_{z,k}$.

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 $\vartheta_{a,k}$;
   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).

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.

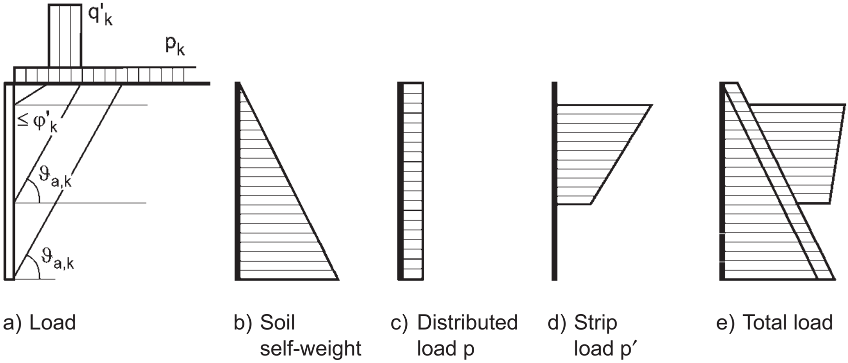


Figure R 71-1. Earth pressure distribution for an unsupported retaining wall, restrained in cohesionless soil, assuming slip surfaces at an angle $\vartheta_{a,k}$ (example)

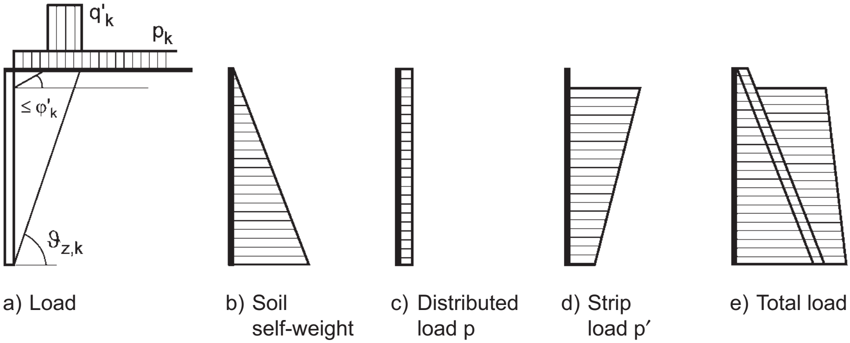


Figure R 71-2. Earth pressure distribution for an unsupported retaining wall, restrained in cohesionless soil, assuming slip surfaces at an angle $\vartheta_{z,k}$ (example)

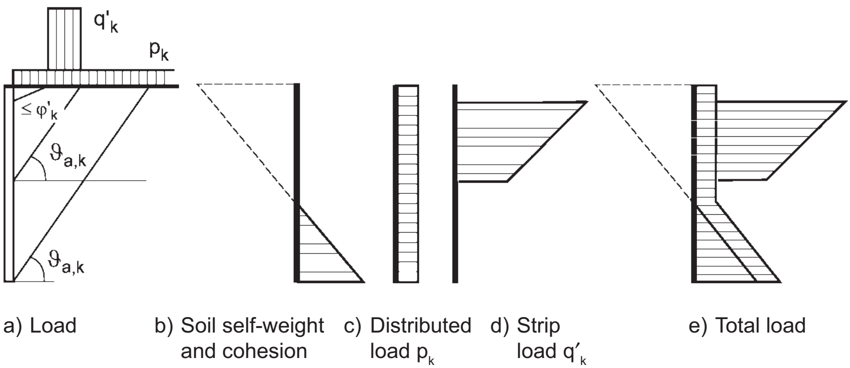


Figure R 71-3. Earth pressure distribution for an unsupported retaining wall, restrained in cohesive soil, assuming slip surfaces at an angle $\vartheta_{a,k}$ (example)

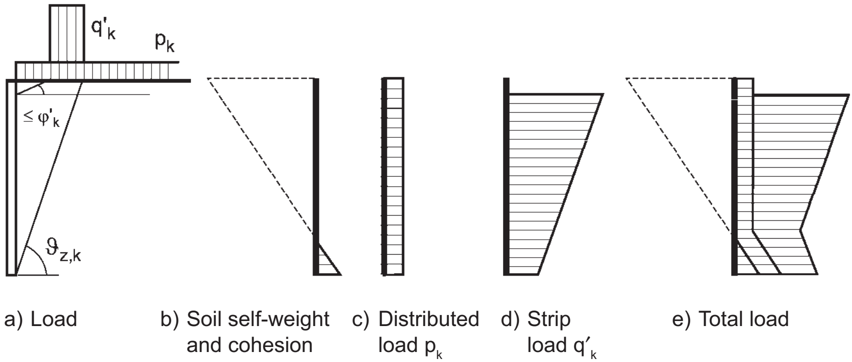


Figure R 71-4. Earth pressure distribution for an unsupported retaining wall, restrained in cohesive soil, assuming slip surfaces at an angle $\vartheta_{z,k}$ (example)

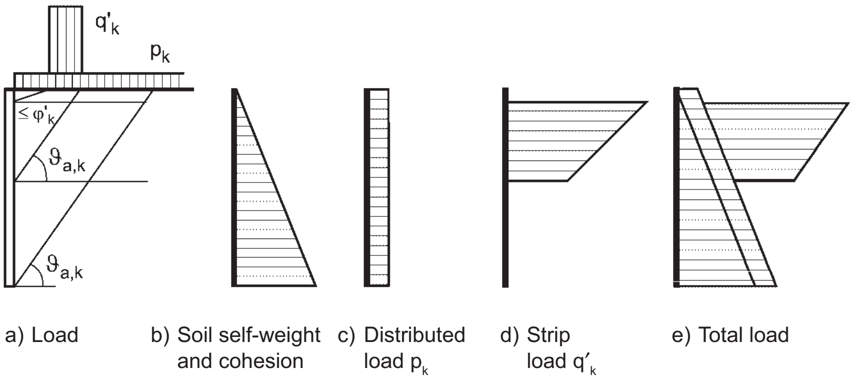


Figure R 71-5. Earth pressure distribution for an unsupported retaining wall, restrained in cohesive soil, assuming a minimum earth pressure (example)

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 for determination of 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 verifications are based on the characteristic at-rest earth pressure $E_{0,k}$. Here, the friction angle φ'_k represents merely a control variable. The following cases are differentiated below:

– at-rest earth pressure from soil weight density;
– 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 weight density 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) For a climbing ground surface 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 as an 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:

   $$f_o = \sqrt{\frac{\sigma_{vo}}{\sigma_v}}$$

   where:

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

   d) The approaches mentioned also apply as an 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 halfspace 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%, 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 no precise determination has been performed, e.g. according to [41] or [46].

6. In principle, the earth pressure $E_{0Bh,k}$ from building loads shall be divided into a permanent component $E_{0Bgh,k}$ from building weight density 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 104, 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 load together with the building weight density.

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 set of struts is removed or a set of anchors is detensioned.

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 densely compacted cohesionless soil or at least plastic cohesive soil, earth pressure redistribution over the remaining excavation depth shall be anticipated corresponding to the new supporting 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 internal design forces 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 selected with the same earth pressure.

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:

   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 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 $e_{hu,k} = ½ \cdot e_{hs,k}$ at the level of the top of the backfill.

   If only one set 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 set 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.

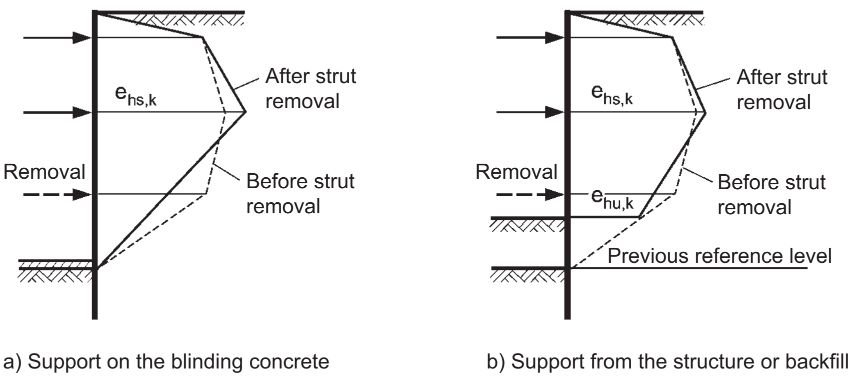


Figure R 68-1. Pressure diagrams for soldier pile walls in retreating states

4 General stipulations for analysis

4.1 Stability analysis (R 81)

For stability analyses in the STR limit state according to R 78, Paragraph 4 (Section 1.4), the following procedure is employed for linear-elastic systems according to DIN 1054:

1. The excavation structure is designed, the dimensions selected and the structural system defined.
2. The characteristic values of the actions are identified, e.g. the loads imposed by weight density, active earth pressure, increased active earth pressure, surcharge and, if applicable, the characteristic deformations. See R 63 (Section 10.6) for how to deal with water pressure.
3. The characteristic stresses E_k 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 are decisive to design.
4. The design values of the effects are determined for each decisive 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 $E_{Q,d} = \Sigma\, E_{Qi,k} \cdot \gamma_Q$

   by multiplying the characteristic action effects E_k 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 (king) 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.

   The resistance design values are obtained using:

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

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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 decisive combination of forces.

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

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

9. Details are given in further Recommendations.

4.2 General information on analysis methods (R 11)

1. All advancing and retreating states for excavating and backfilling shall be investigated. Advancing states refers 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 decisive, the following simplified approaches may be adopted to analyse embedment depth and to determine the effects:

   a) The structural system may be based on a beam on inflexible supports.
   b) The deformations in the various construction stages and the impact 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) 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, erroneous bending moments and incorrect deflections are necessarily obtained (see Figure R 11-1). They should be dealt with as follows:

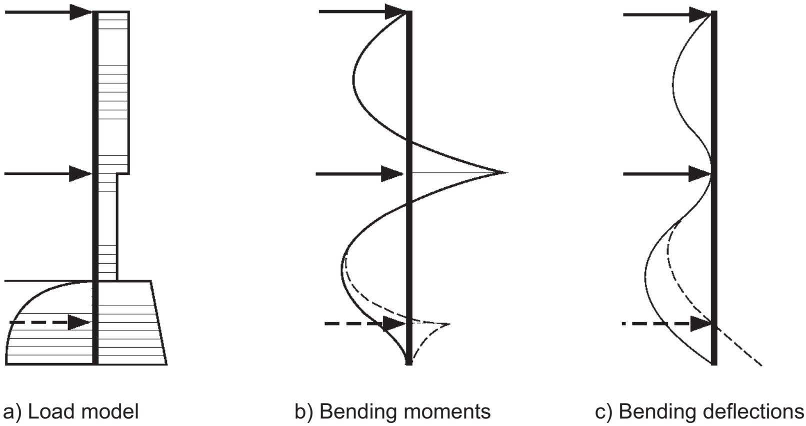


Figure R 11-1. Impact of replacing the ground reaction distributed over the embedment depth by a fixed support

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 backwards 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 from a substantial earth pressure from building loads or from positive water pressure, the resulting errors are generally no longer acceptable.

4. If analysis of serviceability also plays a role 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:

   a) The structural system shall be based on a beam on flexible 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 substituted by their resultant.
   d) 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 **selected** to determine the characteristic effects and to design the sections. For multiple-propped soldier pile walls, sheet pile

walls, girders and cable bridges the elastic-plastic and plastic-plastic analysis methods may also be adopted, beside the elastic-elastic 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) The limit load design method according to R 27 (Section 4.5) allows utilisation of the steel's plasticity reserves. In addition, simple determination of the effects using hand calculations is possible by reducing to individual fields.
c) Adoption of the modulus of subgrade reaction method according to R 102 (Section 4.6) and the finite-element method (FEM) according to R 103 (Section 4.7) allows identification of the soil-structure interaction in the embedment zone.
d) 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 4.7).

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

6. The following redistribution of bending moment is permissible for structurally indeterminate systems according to the elastic-elastic method:

   a) If numerical overloading of the soldier pile wall or the sheet pile wall 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 according to the elastic-plastic method described in DIN 18 800-1:1990-11, Paragraph 7.5.3, 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 neighbouring fields and at the neighbouring support points shall be proved; 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 18 800-1. Following moment redistribution the characteristic material parameters that were reduced by applying the appropriate partial safety factors may not be exceeded at any point, taking the design values of the normal forces into due consideration. In addition, the minimum thicknesses for the flange and webs shall be proved according to R 27, Paragraph 7 (Section 4.5).

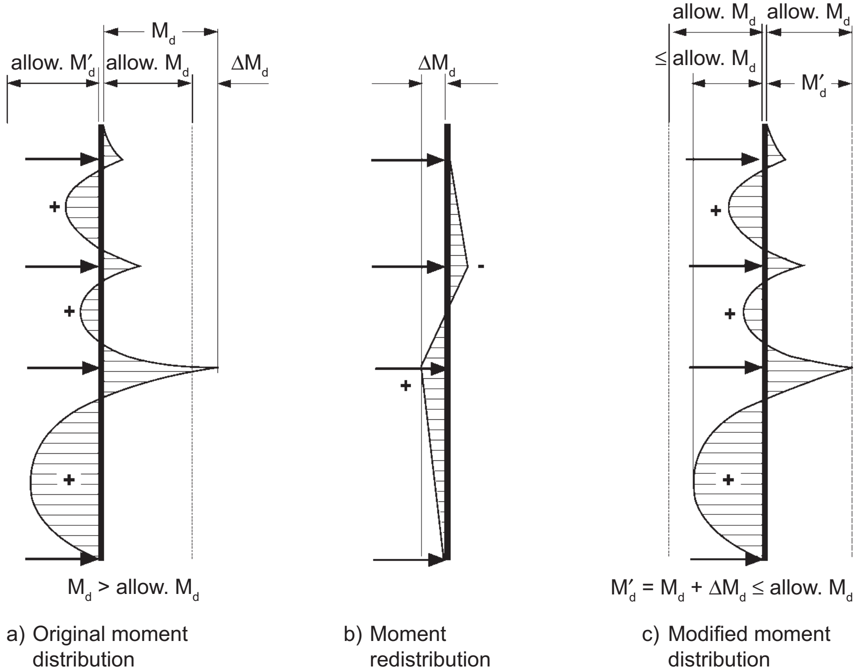


Figure R 11-2. Redistribution of bending moments

d) The moments may also be redistributed according to Paragraph 4 for in-situ concrete walls and bored pile walls, similar to DIN 1045-1:2001-07, Section 8.3. However, the support moment reduction may not be greater than given in DIN 1045, Section 8.3, 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 concrete wall is subsequently utilised as a load-bearing member in a permanent structure, it may prove expedient to forgo reduction of the support moment for the construction stage.

7. Application of the partial safety factor approach 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 named. If superimposing is expedient for programming purposes, the forces and the ground reactions shall be subsequently separated again.

4.3 Determination and analysis of embedment depth (R 80)

1. The STR limit state is decisive for 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.
2. The characteristic earth pressure value is obtained from the characteristic soil properties according to R 2 (Section 2.2):
   - from weight density and cohesion according to R 4 (Section 3.2);
   - from surcharges according to R 6 (Section 3.4).

   The earth pressure from an unbounded distributed surcharge load $p_k \leq 10$ kN/m^2 may be superimposed with the earth pressure from soil weight density 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 weight density 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. Beside the conservative triangular distribution a parabolic, bilinear, rectangular or trapezoid 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 is to be analysed, 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 decisive 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 for the embedment depth t_0.

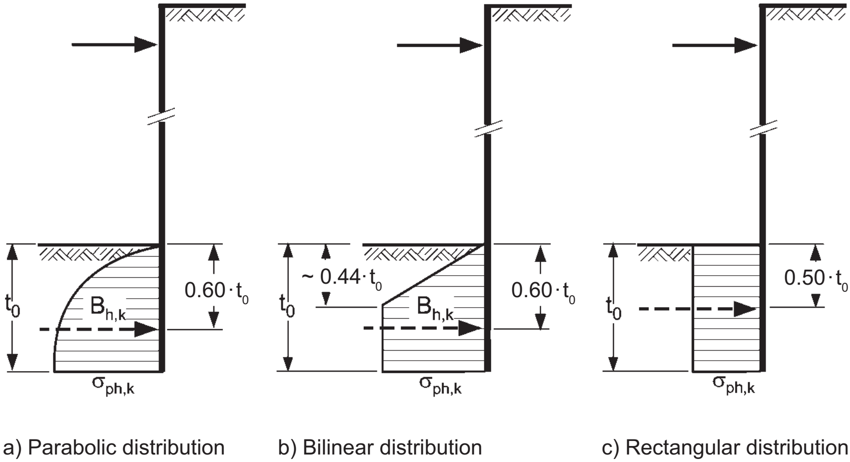


Figure R 80-1. Examples for adopting the ground reaction for free earth support

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 geotechnically restrained walls the ground reactions may be assumed according to *Blum's* [23] load approach. This assumes a linear increase in the ground reaction with depth as far as the theoretical toe, see Figures R 25-1 and R 25-2 (Section 5.4), and R 26-1 and R 26-2 (Section 6.4). The following apply:

   a) For a full geotechnical restraint of propped walls a vertical tangent to the deflection curve is required at the assumed theoretical toe. 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, propped 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 grater 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. For the divisor:
   - $(\gamma_{GQ} \cdot \gamma_{Ep}) \approx 1.20 \cdot 1.30 = 1.56 = \Rightarrow 1.60$ in Load Case 2;
   - $(\gamma_{GQ} \cdot \gamma_{Ep}) \approx 1.10 \cdot 1.25 = 1.37 = \Rightarrow 1.40$ in Load Case 2/3;
   - $(\gamma_{GQ} \cdot \gamma_{Ep}) = 1.00 \cdot 1.20 = 1.20 = \Rightarrow 1.20$ in Load Case 3;

   may be adopted.

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 decisive embedment depth is obtained using 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:

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

   The decisive 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 4.12).

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 verified 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 for 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 respectively most suitable procedure for each construction stage, e.g. according to R 27 (Section 4.5), R 102 (Section 4.6) or R 103 (Section 4.7).

   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.

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 excavators and lifting equipment moving 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, non-cohesive 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;

      better 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 system behaviour 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 system 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 not located at the same position, the respectively largest field moments max. $M_{G,k}$ and max. $M_{Q,k}$ may be regarded as decisive as a 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 $S_{Qi,k}$, 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 $S_{Qi,k}$ are obtained as a difference:

   max. $M_{Qi,k}$ = max. $M_{i,k}$ – max. $M_{G,k}$

   The determined characteristic action effects are converted to design values according to R 81, Paragraph 4 (Section 4.1). See R 105, Paragraph 5 (Section 4.12) 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 decisive action combinations within the respective Load Cases LC 2, LC 2/3 and LC 3:

   a) the characteristic action effects $E_{G,k}$ from permanent actions $S_{G,k}$ are determined separately;
   b) the action effects E_k are determined together with the permanent actions $S_{G,k}$ for every conceivable combination of variable actions $S_{Qi,k}$;
   c) the action effects for the respective combination of variable actions $S_{Qi,k}$ are obtained as a difference:

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

   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 moments from permanent actions and the largest field moments 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 is decisive for a very 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 base of a structure. This problem also occurs in excavations in water with an underwater concrete base. See also Figure R 63-3 (Section 10.6).

4.5 Limit load design method (R 27)

1. Instead of analysis employing the elastic-elastic or elastic-plastic methods, in which the action effects are determined according to elastic theory, analysis for the plastic-plastic method is performed by demonstrating that the stresses computed according to the plastic hinge or plastic zone methods do not lead to a violation of the limit action effects in the plastic condition, see DIN 18 800-1:1990-11, Section 7.5.4. The plastic section and system reserves are utilised. This method is also known as the limit load method. Below, the strict limit load design method, which fulfils all the requirements of the plastic-plastic method, and the simplified limit load design method are differentiated, which does not utilise all plastic load reserves in favour of simpler hand calculations.

2. The following requirements apply for analysis of soldier pile walls, sheet pile walls, girders and cable bridges using limit load methods:

   – the cross-section of individual sections shall be at least simple symmetrical;
   – the load only acts in planes of symmetry;
   – the section is safeguarded against buckling and overturning at the locations of possible plastic hinges; see R 48, Paragraph 3 (Section 13.3).

3. When analysing using the limit load method, earth pressure distribution shall be applied corresponding to wall deflection in a manner assumed to be compatible with the static and dynamic deformation states of the wall, taking plastic hinges into consideration. See [18] for example. This may be deviated from when applying simplified limit load methods if it is obvious or, if necessary, demonstrated, that by simplification no smaller cross sections are computed than with consideration of this demand. See also [19] and [27]. The use of a continuous rectangle is only permissible if it represents a realistic pressure diagram. See R 69 (Section 5.2) and R 70 (Section 6.2) as well as R 42 (Section 7.1).

4. If the decisive design bending moments are approximately determined with the help of compensation between span and support moments for rolled sections with uniform cross sections using the simplified limit load method described in [19] and [27], a bearing capacity analysis can be performed in place of the strict analysis according to DIN 18 800-1, as if the action effects had been determined with the help of elastic theory. In this case the characteristic material parameters for bending, shear and equivalent stresses, reduced by applying the appropriate partial safety factors, may not be exceeded.

5. When determining action effects according to the limit load method it must also be assumed, in analogy to determining action effects according to elastic theory, that the stresses in the serviceability state of the structure according to R 78, Paragraph 6 (Section 1.4) may not reach the yield point. However, the corresponding analysis can be dispensed with if:

   a) Load Case LC 1 is adopted against reaching the plastic limit load for design employing the strict limit load method according to DIN 18 800-1;
   b) the edge stress σ is not decisive for design using a simplified limit load method according to Paragraph 4, but rather the equivalent stress σ_v.

   If the corresponding analysis must be performed it is sufficient to investigate the decisive design construction stage. Here, non-yielding support points may be assumed according to R 11, Paragraph 2 (Section 4.2).

6. The continuity effect that occurs in the serviceability state shall be taken into consideration when determining shear and support forces. If necessary, the shear forces and support forces determined for the full plastification condition in the plastic hinges must therefore be increased by appropriate additions. See also [19] or [52].

7. Regardless of whether the determination of action effects and the design were performed according to the strict or the simplified limit load method, rolled sections for soldier piles, girders and cable bridges shall be investigated for adherence to the minimum thicknesses for flanges and webs in the regions of possible plastic hinges as given in DIN 18 800-1:1990-11, Section 7.5.4, Element 758. If the minimum thicknesses are not achieved at any one point, either a more precise stability investigation shall be performed or it shall be demonstrated that when applying elastic theory the characteristic material parameters, reduced by applying the appropriate partial safety factors, are not exceeded in the decisive construction stage.

4.6 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 in part also for analysis of serviceability. This allows the soil-wall interaction, the actual structural behaviour and the anticipated deflections and deformations to be

better identified than when assuming a predetermined distribution of ground reactions and deflection of the wall toe.

2. It may be approximately assumed that the original at-rest earth pressure on the excavation side of the wall remains generally unaffected even after soil removal is complete. 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 δ_p may be adopted for determination of 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 soil stress:

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

   as a function of the local displacement s_h, also see Figure R 102-1. 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.

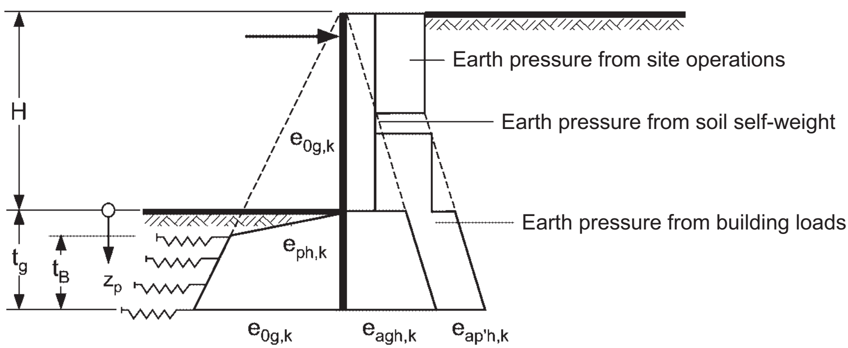


Figure R 102-1. Load model for elastic support in noncohesive soil

4. The most reliable values for the modulus of subgrade reaction $k_{s,h}$ are obtained on the basis of a resistance-deflection relationship for the passive earth pressure as shown in Figure R 102-2 using:

$$k_{sh,k} = \frac{B_{Bh,k}}{(s - s_v) \cdot t_B}$$

Here, the following points apply:

a) The resultant $B_{Bh,k}$ of the subgrade reaction must initially be estimated and then improved iteratively.
b) The value of the remaining at-rest earth pressure force $E_{V,k}$ is obtained from $e_{ph,k}$ and $e_{0g,k}$ as shown in Figure R 102-1 for the specified embedment depth t_g.
c) The variables s and s_V are obtained as shown in Figure R 102-2 and represent the deflections associated with $B_{Bh,k}$ and $E_{V,k}$.

The resistance-deflection relationship is described by the mobilisation curve for the passive earth pressure below the excavation level as shown in Figure R 102-2.

Where:

$E_{0,k}$ is the resultant of the theoretical at-rest earth pressure below the excavation level in the initial condition after excavation;
$E_{V,k}$ is the resultant of the remaining at-rest earth pressure in the excavated condition, taking the original preconsolidation into consideration;
$E_{pgh,k}$ is the characteristic passive earth pressure.

For non-preconsolidated ground the starting point of the mobilisation curve is given by $E_{0,k}$ and the deflection s = 0, or by $E_{V,k}$ and s_V for preconsolidated ground. In the STR limit state and for completely mobilised passive earth pressure $E_{pgh,k}$ the deflection $s = s_{Failure}$.

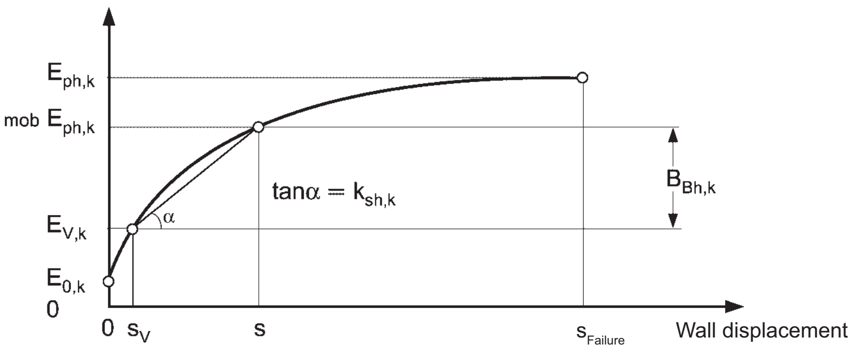


Figure R 102-2. Determination of the modulus of subgrade reaction

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

   a) As an approximation the following applies for in-situ concrete walls and sheet pile walls according to [96]:

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

   The embedment depth t_B utilised by the subgrade is decisive.

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

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

   The flange width b is decisive for driven soldier piles. For soldier piles installed in pre-drilled boreholes and concreted at the toe, the borehole diameter D replaces the flange width b. Otherwise, this approach assumes that a computed deflection $s = 0.03 \cdot b$ 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 metre for analysis purposes. This applies accordingly in general for the width b.

   c) The oedometric modulus $E_{S,h}$ is obtained from the anticipated stress range. If the oedometric modulus E_S is only known for the vertical stress, it shall be converted to horizontal stress, as an approximation, using the factor $0.5 \leq f \leq 1.0$.

6. Guide values for mean subgrade reaction moduli for continuous walls are given in Appendix A 5 as a function of the degree of utilisation of the passive earth pressure according to [94], [95] and [121]. They apply for wet soils. The given values shall be halved for buoyant soils. The values for noncohesive soils are determined as a function of the relative density. The data provided for cohesive soils is for stiff to very stiff consistencies.

7. If the stiffness conditions of the retaining wall and the soil allow a geotechnical restraint, the following apply below the point of rotation 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, a constant modulus of subgrade reaction may be assumed. However, in noncohesive soils the modulus of subgrade reaction in the region below the intersection of the at-rest earth pressure stresses $e_{0g,k}$ and the passive earth pressure stresses $e_{pgh,k}$, as shown in Figure R 102-1, shall be reduced enough that the resulting reaction stresses resulting from the initial stresses

$e_{0g,k}$, and the soil stresses $\sigma_{Bh,k}$ activated by the deflections, do not exceed the passive earth pressure stresses $e_{ph,k}$. At the start point of the ground resistance, $e_{0g,k} \approx e_{ph,k}$ as shown in Figure R 102-1, $\sigma_{Bh,k} = 0$. 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. 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 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 of the resultant, remaining at-rest earth pressure $E_{V,d}$ is obtained from the characteristic value $E_{V,k}$ by multiplying by the partial safety factor γ_G.

    When using the modulus of subgrade reaction guide values given in Appendix A 5 the determined characteristic values of the sum of $B_{Bh,k}$ and $E_{V,k}$ shall correspond to the associated degree of mobilisation of the characteristic passive earth pressure $E_{ph,k}$.

12. The passive earth pressure $E_{phP,k}$ for a free earth support may be adopted for parallel deflections when analysing according to Paragraph 10, as shown in Figure R 102-3 a). For a full or partial restraint effect the passive earth pressure $E_{phF,k}$ is decisive for rotation around the toe as shown in Figure R 102-3 b). As an approximation, the following relationship applies to continuous walls in noncohesive 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 to cohesive soils as an approximation.

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.

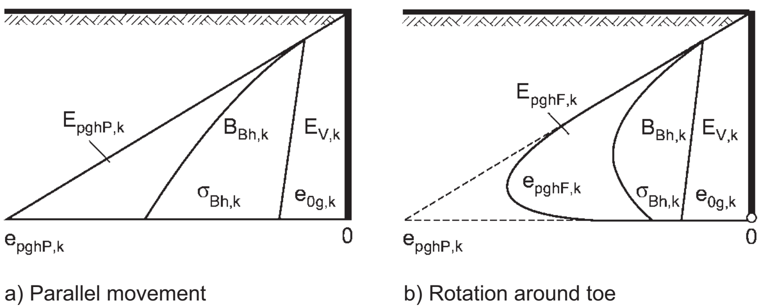


Figure R 102-3. Utilisation of passive earth pressure in noncohesive soil

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:

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

    is decisive.

15. 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 transmitted to the subsurface with sufficient safety, the vertical component of the ground reaction resultant may alternatively be replaced by the skin friction.

4.7 Finite-element method (R 103)

1. In principle the finite-element method (FEM) is suitable for:

   - determination of the characteristic stresses in decisive sections through the excavation structure and in the soil-structure interfaces;
   - estimates of settlements and deflections of the retaining wall, and the ground behind the retaining wall and below the excavation level;
   - analysis of safety against slope failure and general (global) failure.

   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. For example, this can involve the following cases:

   a) Retaining walls with support conditions that do not allow confident determination of the magnitude and distribution of earth pressure, with yielding anchors and a flexible wall.
   b) Excavations with complex geometrical dimensions, e.g. salient or re-entrant corners and staggered retaining walls with a berm width which does not allow confident determination of the magnitude and distribution of earth pressure using classical assumptions.
   c) Excavation structures in which a realistic assessment of the impacts from excavating and strut or anchor prestressing on the earth pressure redistribution and deflections of the retaining wall is required.

d) Excavation structures in which a realistic assessment of seepage and associated water pressures is required.
e) Excavations adjacent to buildings, pipelines, other structures or traffic areas.

The impact of negative porewater pressures, which can ensue in the course of excavation, cannot yet be reliably assessed by FEM.

Notes on the application of FEM for 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 application of FEM and definition of the constitutive equations used require particular care and experience, as well as specialised knowledge of soil mechanics, in particular for determining the necessary material parameters and variables. It is therefore generally assigned to Geotechnical Category GC 3 according to DIN 1054. The following points are recommended:

   a) A geotechnical expert trained in the required field and in possession of the appropriate experience as described in DIN 4020 or DIN 1054 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 expected of the geotechnical expert that a constitutive equation is recommended, which allows a realistic determination of the stress and displacement conditions, taking the problem proposition and the local ground conditions into consideration, e.g. consolidation conditions, granulometric properties and relative density.
   c) The Recommendations of the Working Group "Numerical Methods in Geotechnics" shall be observed when determining the material parameters and 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 unloading processes in particular, should be selected.
   b) The characteristic values of the parameters required for the selected constitutive equation shall be determined from laboratory and field tests, or by employing empirical values obtained in comparable ground conditions. In association with the Geotechnical Category GC 3 according to DIN 1054, triaxial tests should generally be carried out to determine the decisive stiffness parameters for the decisive soil layers. Consolidation tests may suffice for cohesive soils, depending on the constitutive equation adopted.
   c) Failing the relevant experience initial numerical calculations shall be performed to examine and optimise the range of the analysis and the mesh subdivisions, as well as the required modelling steps for the excavation condition being investigated.

d) If possible, initial numerical calculations shall be performed using measurement data from excavations with similar ground conditions in order to calibrate and check the selected parameters for the constitutive equation.

These investigations and initial calculations are necessary to achieve realistic results, taking the numerous possibilities for selecting and influencing parameters into consideration. In order to make the analysis assumptions transparent, the numerical analysis should always be preceded by appraisable documentation of points a) to d).

5. Generally speaking, realistic upper and lower limit values of the respective soil parameters should be included:

   a) For determination of the characteristic stresses for analysis of bearing capacity, only conservative values are initially required.
   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 both the upper and the lower characteristic values, e.g. if the impacts are in part favourable and in part unfavourable or if the possible result boundaries need 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 calculation by means of classical beam structural analysis on a suitable structural system is recommended in order to reduce the extent of iterative component optimisation.

7. It is recommended that suitable contact elements be adopted for considering the soil-structure interactions. 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 toe 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 resultant support force design value according to Paragraph 9;
   $E_{ph,d}$ the passive earth pressure design value 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.

Due to the common geometrical and physical non-linearity of FEM analyses the proposed procedure represents an approximation only; however, it is sufficient for practical purposes.

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.

10. 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 8, as shown in Figure R 102-3 a). For a full or partial restraint effect the passive earth pressure $E_{phF,k}$ is decisive for rotation around the toe as shown in Figure R 102-3 b). As an 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 as an approximation to cohesive soils.

11. Verification 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 already incorporated in the numerical analysis.

12. 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 transmitted to the subsurface with sufficient safety, the vertical component of the characteristic earth pressure is obtained by integration of the vertical stresses across the rear of the wall.

13. For homogeneous, cohesive soil and in cohesive soil layers, an additional analysis based on R 4, Paragraphs 3 to 5 (Section 3.2) shall be performed using the equivalent friction angle $\varphi_{Equiv,k}$, whereby all other parameters remain unchanged. The design of individual components utilises the most unfavourable action effects.

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

    a) Verification of safety against slope failure and general failure can only be performed with the aid of FEM if based on the Fellenius circular-arc method, and the shear strength in the soil and in the soil-structure contact area is reduced in stages until no computed equilibrium state is possible or a computed failure state occurs. The limits of this method and notes on the definition of 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 limit state according to DIN 1054 and cannot therefore be analysed using FEM.

15. The results of numerical analyses shall be transparent. 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 are recommended:

    a) a graph of the horizontal earth pressure components and the water pressure for the height of the retaining wall in the individual excavation stages;
    b) a graph of the action effects for the height of the retaining wall in the individual excavation stages;
    c) a graph of the action effects at the decisive sections as a function of the construction stage;
    d) a graph of the horizontal wall displacement at various points of the retaining wall as a function of the construction stage;
    e) a graph of the surface settlements at various points on the ground surface as a function of the construction stage.

    Furthermore, evaluations of plastic regions and vector displacements may be useful for result assessment.

4.8 Verification 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 downwardly 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 shall not be 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 weight density 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 inclination 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 and of the design condition may be equated to each other.

   c) Taking the assumptions in Point 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 decisive 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 $C_{h,k}$ 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 $\delta_C \leq 1/3 \cdot \varphi'_k$.

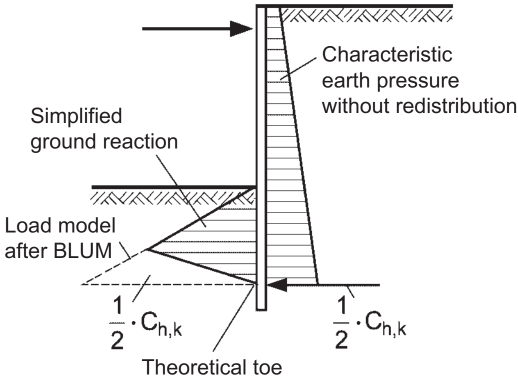


Figure R 9-1. Effective component of the ground reaction for *Blum's* earth restraint

4. The vertical forces from variable actions may not be taken into consideration for both the analysis according to Paragraph 1 and 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 $\alpha_A \geq 15°$ analysis 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 analysed, 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 altered 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 transmission 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 is decisive for 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 transmitted to the sheet pile wall or the soldier piles via bonding stress. See also [127].

4.9 Verification of the transmission of vertical forces into the subsurface (R 84)

1. It shall be verified that the downwardly directed vertical actions can be transmitted from the wall to the subsurface and that the wall will not sink. 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 downwardly 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 downwardly directed characteristic actions, e.g. the weight density of the wall, permanent forces acting immediately on the wall, the vertical component of the earth pressure determined using the positive angle of inclination and, where 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 are obtained:
      – the skin resistance of the developed surface A_S of the area and the skin friction $q_{s1,k}$ from
        $R_{v,k} = A_S \cdot q_{s1,k}$
      – the vertical component of the support force B_k from the horizontal support force $B_{h,k}$ and the friction coefficient tan $\delta_{B,k}$ from
        $R_{v,k} = B_{h,k} \cdot \tan \delta_{B,k}$

   See R 85 (Section 13.10) and Appendix A 10 for the skin friction $q_{s1,k}$.

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) The characteristic vertical component C_V of the upward-acting equivalent force C is obtained in contrast to the analysis according to R 9 (Section 4.8) 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 4.8) 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 4.8).

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 for continuous reinforcement of the structurally extended sections for diaphragm walls.

5. If transmission 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 transmission 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 altered 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) The partial safety factors γ_p for pile resistances may be adopted on the resistances side.

b) If the settlements of the retaining wall 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 weight density can be transmitted to the hardened cut-off wall via bonding stress, together with the vertical component A_V of an anchor force. See also [127].

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

1. It may be necessary to analyse safety against base heave for soils with a characteristic friction angle below the excavation level of less than $\varphi'_k = 25°$. See also [25], [26], [52] and [130]. This analysis forms part of the STR limit state. The following procedure is used:

   a) The decisive forces are those acting on a soil mass of width b_g. Actions include the weight G_k of the soil mass and, if applicable, surcharge load P_k. Resistances include the lateral vertical force $R_{v,k}$ and the bearing capacity $R_{Gr,k}$ of the load-carrying strip of width b_g (Figure R 10-1).

   b) The limit state condition using the design values:

   $$G_d + P_d \leq R_{v,d} + R_{Gr,d}$$

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

   $$\mu = \frac{G_d + P_d}{R_{v,d} + R_{Gr,d}}$$

   is obtained.

   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). The width does not need to be varied in the case of narrow excavations (Figure R 10-1 c), see [52].

   c) The limitation of the friction coefficient when determining the friction component of $R_{v,k}$ and the peculiarities for narrow excavations [52] shall be observed.

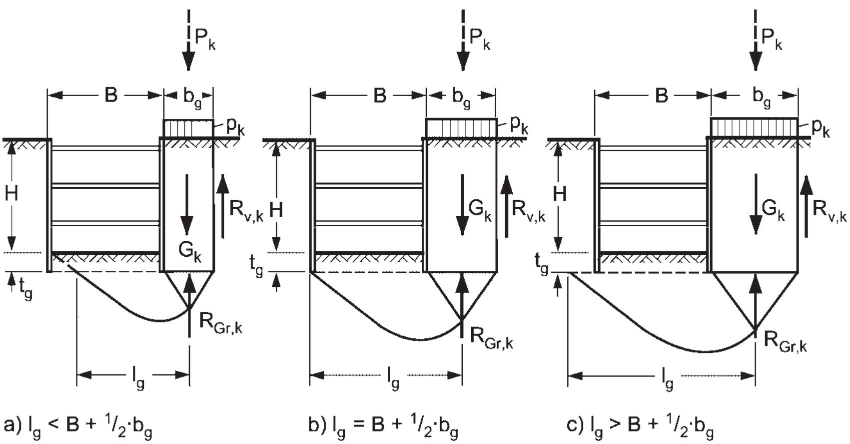


Figure R 10-1. Excavation base heave in homogeneous soil

d) The design values $R_{v,d}$ and $R_{Gr,d}$ are obtained from the characteristic values $R_{v,k}$ and $R_{Gr,k}$ by division by the partial safety factor γ_{Gr} for bearing capacity.

2. Bearing capacity safety shall be proved 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).

3. In exceptional cases it may be necessary to analyse general stability in the GEO 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 then when they unfavourably impact on stability due to their position above the centre 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]. See also R 63, Paragraph 5 (Section 10.6).

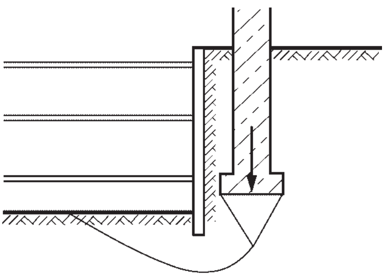


Figure R 10-2. Analysis of bearing capacity for a braced excavation

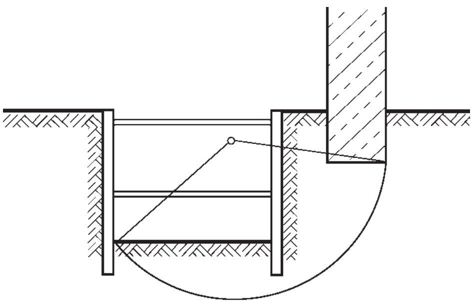


Figure R 10-3. Analysis of general stability for a braced excavation

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 analyse that the associated heave of provisional bridge supports or excavation coverings, or of intermediate supports for buckling protection devices, has no negative impact. See also [51] and [52].

4.11 Verification of serviceability (R 83)

1. The regulations of Sections 5 and 6 ensure that for at least medium-dense, noncohesive soil and at least stiff, cohesive soil, the displacements of the toe support of a multiple-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, where 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 hazard that the deformations and displacements of the retaining wall impair the stability or serviceability of adjacent structures despite adhering to the named measures, the serviceability limit state shall be analysed according to DIN 1054.
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 groundwater table (also see [96, 97]);
   - for excavations adjacent to structures founded in soft, cohesive soils;
   - for excavations adjacent to structures placing especially great 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 impacts 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 4.7).

4. Verification of serviceability is performed using 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 STR limit state. The earth pressure from an unbounded distributed surcharge 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 loads ensuing from traffic and site operations 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 applies for the earth support:

   a) Generally, the actual embedment depth and not the mathematically required embedment depth shall be assumed, if an embedment depth greater than that mathematically 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 for the STR limit state given 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 4.6).
   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 of flexible retaining walls, beside *Blum's* approach:

   a) As an approximation the distribution of the ground reactions may be assumed as shown in Figure R 9-1 (Section 4.8), 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 noncohesive 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 4.6).
   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 4.6.

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. As an approximation, the corresponding displacement is obtained:

– according to [20] or [46] for soldier pile walls in noncohesive soil and according to [93] in silty soils;
– according to [94] for continuous walls in noncohesive 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. Beside 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.:

– loosening of the ground prior to installing the piles of a soldier pile wall;
– soil removal when drilling, soil collapsing as a result of overcutting by the bit;
– unloading of the soil 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;
– compaction of the ground during driving or anchor casing;
– unloading of the ground caused by void formation when drawing sheet piling.

If these effects cannot be avoided by means of technical measures, the impacts on wall serviceability shall be approximately estimated.

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 arrangement of supports;
    - increasing the embedment depth;
    - installation of 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 STR limit state shall be performed.

13. Beside the deflections of the retaining walls and the deformations of the ground behind them, base heave and heave of the retaining wall may also play a role, even in braced excavations. See also [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. See also [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 EQU limit state, e.g. when applying the observational method.

4.12 Allowable simplifications in the STR limit state (R 104, draft)

1. The following major changes are coupled 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.

c) Superimposing earth pressure and reduced passive earth pressure are 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 strip loads caused by construction machinery, in a mutual pressure diagram with the earth pressure from soil weight density, 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) All dimensions shall be assumed 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 decisive, 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. See also Paragraphs 3 to 5.
   b) Transitional regulations are required for the period until new applications are available based strictly on the partial safety factor approach 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 weight density, 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 DIN 1054,** 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:

   – $f_q = \gamma_Q / \gamma_G = 1.30/1.20 = 1.08$ in Load Case LC 2
   – $f_q = \gamma_Q / \gamma_G = 1.15/1.10 = 1.05$ in Load Case LC 2/3
   – $f_q = \gamma_Q / \gamma_G = 1.00/1.00 = 1.00$ in Load Case LC 3

   and their effects in the shape of earth pressure from live loads be superimposed with the earth pressure from soil weight density, 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 the 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 $\eta_p = \gamma_{GQ} \cdot \gamma_{Ep} \geq 1.20 \cdot 1.30 = 1.56 \approx 1.60$ in Load Case LC 2
      – where $\eta_p = \gamma_{GQ} \cdot \gamma_{Ep} \geq 1.10 \cdot 1.25 = 1.38 \approx 1.40$ in Load Case LC 2/3
      – where $\eta_p = \gamma_{GQ} \cdot \gamma_{Ep} = 1.00 \cdot 1.20 = 1.20 = 1.20$ in Load Case LC 3

      if η_p can be selected.

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

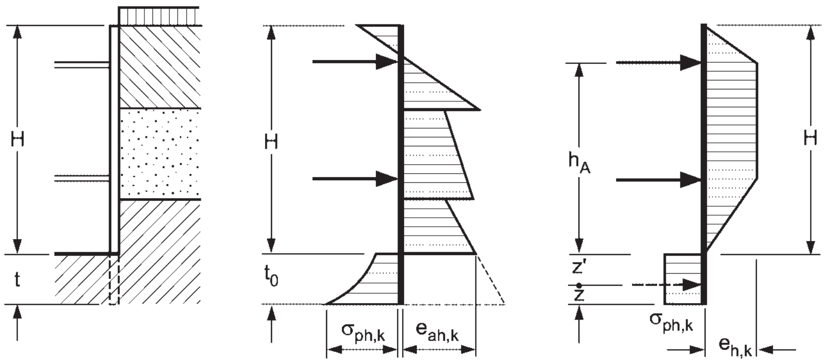
      – the characteristic earth pressure is increased using γ_G or γ_Q;
      – the characteristic passive earth pressure is reduced using γ_{Ep} and;
      – the increased earth pressure is superimposed with the reduced passive earth pressure.

      In order to cater for 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 decisive 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 fulfilled, 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 weight density, unbounded distributed load $p_k \leq 10$ kN/m^2 and, where 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.
2. For unsupported or yieldingly supported soldier pile walls restrained in the ground, and for flexibly supported soldier pile walls, the classical earth pressure distribution shall always be applied for verification 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.
3. For slightly yielding supported 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



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 69-3b

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)

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pressure diagram may generally approach $e_h = 0$ 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 according to R 15, Paragraph 6 c) (Section 5.5), for soldier pile walls with a free earth support, $e_h \geq 0$ shall be defined at the base of the soldier pile. R 15, Paragraph 7 c) (Section 5.5) also applies to soldier pile walls restrained in the ground. No differentiation need be made between a free earth support and restrained 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 supported 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 of these Recommendations [124] and [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, may not be superimposed on the earth pressure according to Paragraph 1. However, also see R 104, Paragraph 3 (Section 4.12).

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 according to R 14 (Section 5.3) or;
   b) as an earth restraint or partial earth restraint 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. Bored pile walls are treated as soldier pile walls. However, a small 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 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 densely compacted, cohesionless soil or at least stiff, cohesive soil is present;

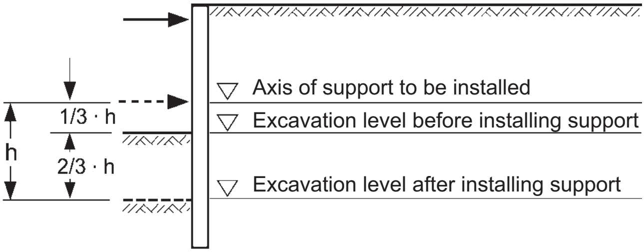


Figure R 69-1. Excavation limit before installing support

- 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 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 weight density, 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].

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

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

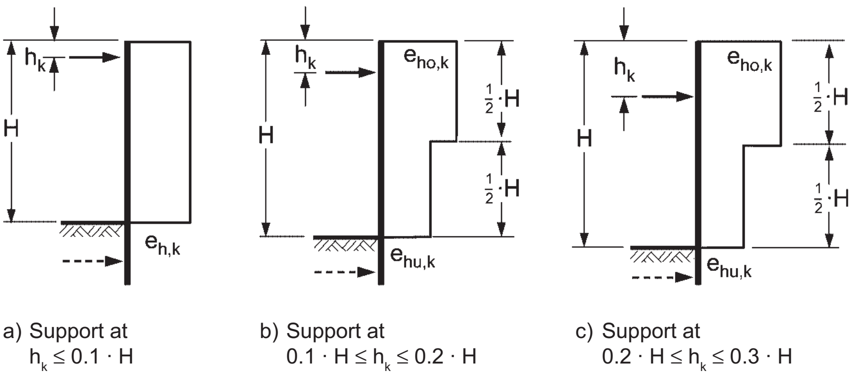


Figure R 69-2. Pressure diagrams for single-propped soldier pile walls

c) a stepped rectangle at half height 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 $h_k = 0.30 \cdot H$.

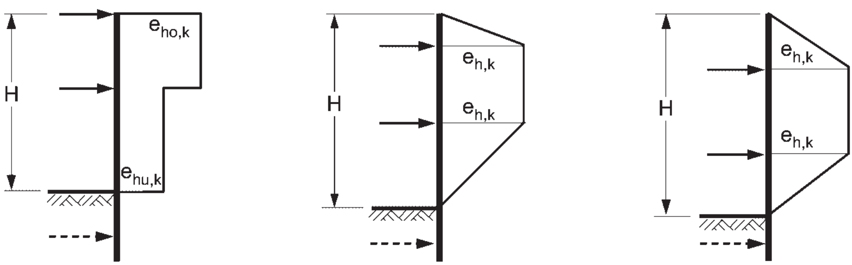
For $h_k > 0.30 \cdot H$ a triangle as shown in Figure R 5-1 i) (Section 3.3) is recommended as the realistic pressure diagram, with the largest ordinate at the height of the support.

3. The following pressure diagrams may be assumed 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 and an ordinate ratio $e_{h0,k} : e_{hu,k} = 2.00$ 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.

4. The trapezoid as shown in Figure R 69-4 may be assumed 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 $z_e = 0.55 \cdot H$.

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 principally 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.



a) High supports b) Central supports c) Low supports

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

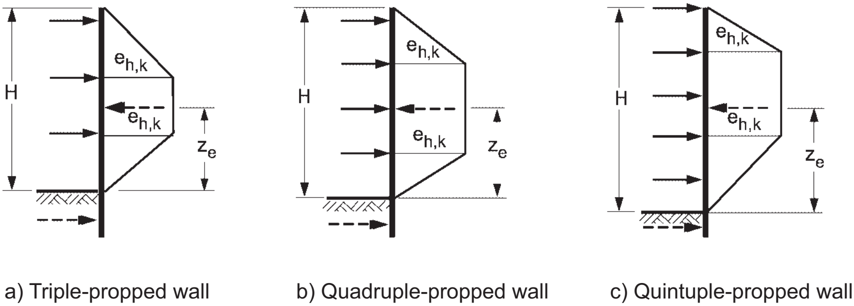


Figure R 69-4. Pressure diagrams for triple- or multiple-propped soldier pile walls

5.3 Passive earth pressure for soldier pile walls with free earth supports (R 14)

1. The characteristic passive earth pressure of soldier piles can be determined for cohesionless soils according to the proposal for analysis given in [20]. 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. The respectively smallest value is then decisive for analysis. See also [52]. If the analysis proposal derived for cohesionless soils is adopted for cohesive soils, the proportion of the passive earth pressure resulting from cohesion shall be reduced to half of the computed value. See also [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 γ_{Ep} according to 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 accepting the support force in the ground, considerable toe displacements must generally be anticipated. Only if the design passive earth pressure is reduced using the calibration factor $\eta_{Ep} = 0.80$ may it be assumed for cohesionless soils and at least stiff, cohesive soils that the displacements of the toe support 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 no 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. As an approximation for either cohesionless soil or at least stiff, cohesive soil, a parabolic or bilinear approach as shown in Figures R 80-1 a) and 80-1 b) (Section 4.3), with the centroid of the ground reaction at $z' = 0.60 \cdot t_0$, may be assumed. If equilibrium of the horizontal forces according to R 15, Paragraph 1 (Section 5.5) can be verified, a support at the centroid of the ground reaction may be assumed not only for a stability verification according to R 80, Paragraph 4 (Section 4.3), but also for a serviceability verification according to R 80, Paragraph 5 (Section 4.3). The cantilever moment described in R 11, Paragraph 3 a) (Section 4.2) does not occur in this case, because 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 selected, 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 earth restraint at depth t'_1 according to R 25, Paragraph 6 (Section 5.4).

   The following apply:

   a) The reduced embedment depth t_0 or the depth t'_1 may be determined or verified using the reduced partial safety factor $\gamma_{Ep,red} = 1.00$.
   b) Any calibration factor $\eta_{Ep} = 0.80$ 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 displacements of the toe may be estimated with the aid of the information given in [20], [93] and DIN 4085, or simple relationships between ground reaction and displacement be derived. The resultant $E_{V,k}$ in Figure R 102-2 (Section 4.6) is obtained from the remaining earth pressure stresses in the excavated condition as shown in Figure R 102-1 (Section 4.6), taking preconsolidation into consideration, whereby the stresses are only applied to the actual pile width. Iteration shall be performed where necessary, until the ground reaction and displacement are approximately congruent.

5.4 Toe restraint for soldier pile walls (R 25)

1. If the girders of a soldier pile wall embed deeply enough in the ground below the excavation level, a degree of restraint in the soil can be adopted for determination of action effects. This soldier pile restraint 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 earth restraint 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 restraint depends on the deformation behaviour of the soldier piles and the ground. In this case, a full geotechnical restraint necessitates 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 geotechnical restraint occurs in the ground 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, the backward rotation of the wall required for mobilisation of the equivalent force C below the theoretical toe may not occur or may only occur partially.

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 expedient 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 sheet pile walls. The result is the same passive earth pressure as in front of a sheet pile wall if the failure bodies in front of the individual soldier piles overlap and the wall friction angle is adopted at $\delta_p = 0$. 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 sheet pile wall [19, 20].

3. If the passive earth pressure failure bodies in front of the individual soldier piles in cohesionless soils 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, as an 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 restrained in the ground. When verifying the embedment depth

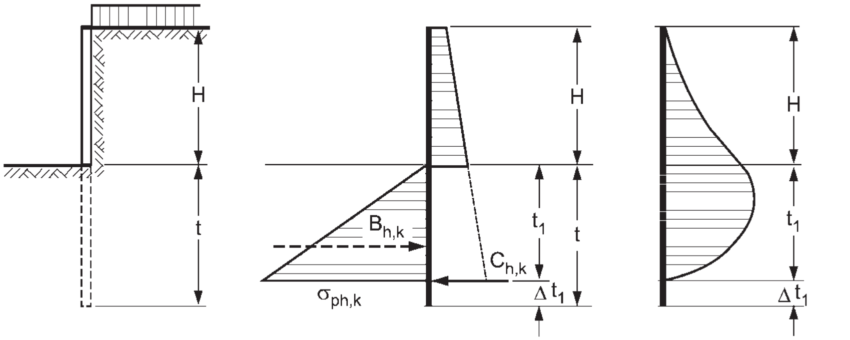


Figure R 25-1. System, load and moment distribution for an unsupported soldier pile wall restrained in the ground

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 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 selected, and utilisation of the ground reaction thus reduced and, if necessary, a stronger section than determined by calculations also be selected. This is particularly the case if loosely compacted, cohesionless soils or only roughly stiff, cohesive soils are present in the restraint area. For retaining walls close to foundation loads and for excavations in soft, cohesive soils an unsupported wall with only an earth restraint is generally not permissible due to the large anticipated deformations, see R 20 (Section 9.1) or R 101 (Section 12.12).

5. A load model as shown in Figure R 25-2 b) results for supported soldier pile walls. For medium-dense or densely compacted, cohesionless soil, or at least stiff, cohesive soil, it may generally be accepted that the deformation conditions associated with a full restraint 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 embedment depth. For loosely compacted soils and for stiff soldier pile sections 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 $\eta_{Ep} = 0.80$ according to R 14 (Section 5.3). A restraint effect may not generally be applied for soft, cohesive soils or soils with a high organic content, see R 96 (Section 12.7).

6. Between the limit cases of full restraint and free earth support, intermediate cases with partial restraint are possible and may be adopted for supported

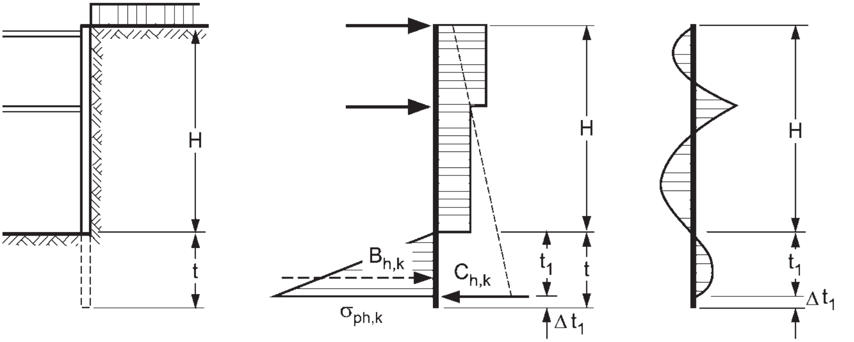


Figure R 25-2. System, load and moment distribution for a double-propped soldier pile wall restrained in the ground

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. See also R 80, Paragraph 5 b) (Section 4.3).

7. The embedment depth t_1, which is required for restraint 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 restraint can develop in the ground. As an approximation for partial restraint the surcharge Δt_1 may be linearly interpolated between the decisive full restraint 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 degree of restraint can also be based on elastic bedding using a deformation resistance. See also R 102 (Section 4.6).
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 restraint at the specified depth t'_1.

   The following apply:

   a) The reduced embedment depth t_1 or the depth t'_1 may be determined or verified using the reduced partial safety factor $\gamma_{Ep,red} = 1.00$.
   b) Any calibration factor $\eta_{Ep} = 0.80$ necessary according to Paragraph 3 remains unaffected.

10. Analysis of the vertical component of the support force B_k shall be performed according to R 9 (Section 4.8), that of the transfer of vertical forces to the subsurface according to R 84 (Section 4.9).

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

1. The earth pressure $\Delta E_{ah,d}$ 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 this earth pressure, together with the design support force $B_{h,d}$ from the soldier piles, is completely transmitted 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 multiplication with the partial safety factors γ_G and γ_Q, and by division by the partial safety factor γ_{Ep} according to R 79 (Section 2.4) respectively. If necessary the calibration factor $\eta_{Ep} = 0.60$, as specified in R 22, Paragraph 6 (Section 9.5), shall also 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 ensues 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 value is decisive. The theoretical support point for soldier piles restrained in the soil 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 restrained in the soil is equal to the numerically 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 reduced approximately by half of the computed equivalent force $C_{h,k}$, as shown in Figure R 9-1 (Section 4.8), considering the magnitude of the actual anticipated ground reaction required for soldier pile restraint. If applicable, R 25, Paragraph 8 (Section 5.4) shall be taken into consideration when adopting an elastic support.

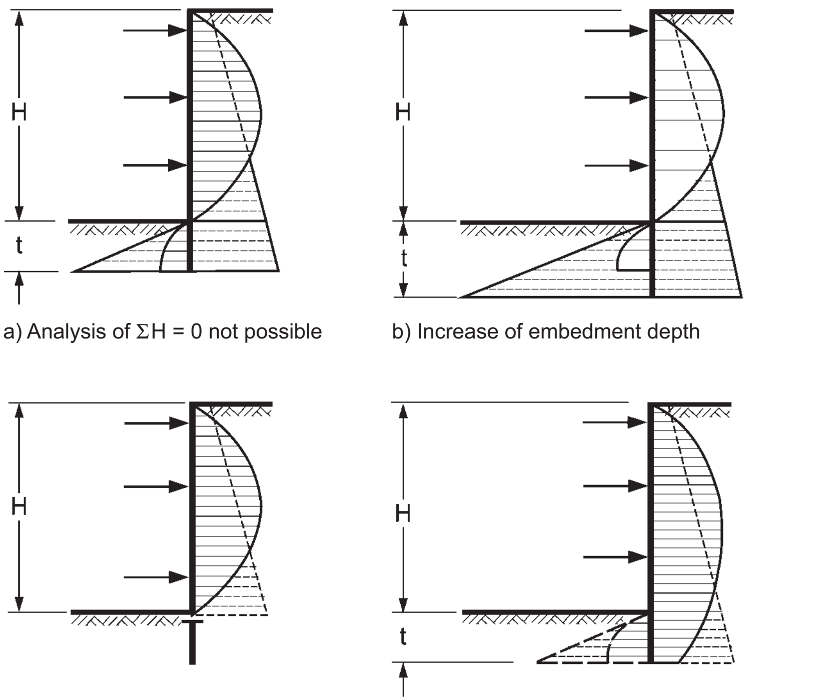
4. The characteristic passive earth pressure can be determined as for a closed wall using the wall friction angle $\delta_p = -\varphi'$, if based on curved or non-circular slip surfaces. See also R 19, Paragraph 1 (Section 6.3).

5. If analysis with the selected embedment depth according to Paragraph 1 is not possible for unsupported soldier pile walls restrained in the soil then either:

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

6. If analysis with the initially selected embedment depth and the earth pressure adopted (Figure R 15-1 a) according to Paragraph 1 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) analytical 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 taken up in the redistribution (Figure R 15-1 d).



a) Analysis of ΣH = 0 not possible

b) Increase of embedment depth

c) Analysis without embedment

d) Earth pressure redistribution to the wall toe

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

7. If analysis with the selected embedment depth according to Paragraph 1 is not possible for supported soldier pile walls restrained in the soil then either:

   a) the embedment depth shall be increased or;
   b) full restraint shall be dispensed with and analysis performed with a partial restraint or with a free earth support, or;
   c) the complete earth pressure from the surface to the theoretical toe shall be taken up in the redistribution, or;
   d) the soldier pile wall shall be treated as a sheet pile wall.

8. The following additional verifications are required for the solutions mentioned 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 must 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 must be demonstrated that the soldier piles and struts or anchors are capable of transmitting 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 accept the active earth pressure. This is the case for cohesionless soils in particular. However, also see R 10, Paragraph 1 (Section 4.10).
   c) The action effects shall be determined again for redistribution of earth pressure 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 applied. However, the component lying below the excavation level shall be taken into consideration for analysis according to Paragraph 1, which must be renewed for the altered conditions.
   d) If a greater embedment depth than that of the original analysis is found for a projected sheet pile wall during the additional investigation according to Section 5 b) or Section 6 b), the larger value is decisive. 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 of $\varphi'_k \geq 32.5°$ is present below the excavation level and possesses approximately the same weight density 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 centre distances 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 loosely compacted, 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

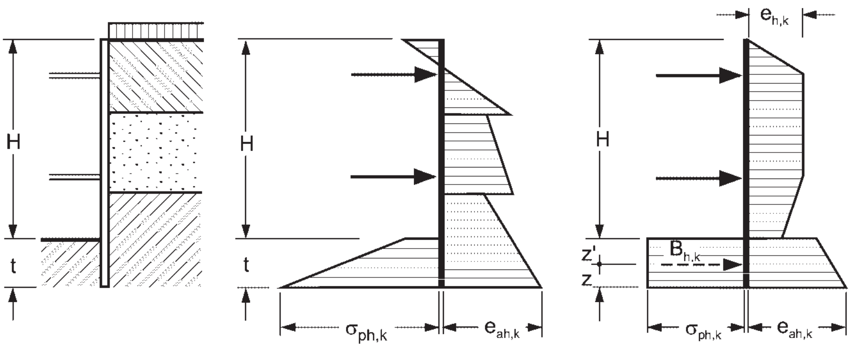
6.1 Determination of load models for sheet pile walls and in-situ concrete walls (R 16)

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 fulfilled, the earth pressure ordinates $e_{ah,k}$ according to R 4 (Section 3.2) and R 6 (Section 3.4) shall be determined according to classical earth pressure theory, using characteristic soil properties and taking into consideration soil weight density, 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 restrained in the ground.

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

2. For unsupported sheet pile walls and in-situ concrete walls restrained in the ground, and for yieldingly supported sheet pile 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 at the height of the base of the wall or the theoretical toe.



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 supported sheet pile walls and in-situ concrete walls with active earth pressure and free earth support

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3. For slightly yielding sheet pile walls and in-situ concrete walls, the 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 from the surface to the excavation level. If there are reasons for anticipating upward earth pressure redistribution from the region below the excavation level, 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 is restrained in the ground, 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 slightly yieldingly 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 in R 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, may not be superimposed on the earth pressure according to Paragraph 1. However, also see R 104, Paragraph 3 (Section 4.12).

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 according to R 19 (Section 6.3) or;
   b) as an earth restraint or partial earth restraint according to R 26 (Section 6.4).

   In the case of a free earth support in cohesive soil a load model according to 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;
   - 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 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 be adopted the pressure diagrams described below in the advancing and fully excavated states when adopting the active earth pressure from soil weight density, 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. See also [52].

   The pressure diagrams proposed below assume earth pressure redistribution from the ground surface to the excavation level. 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 assumed as realistic for single-supported sheet pile walls and in-situ concrete walls:

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

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

3. The following pressure diagrams may be assumed as realistic for double-supported 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;

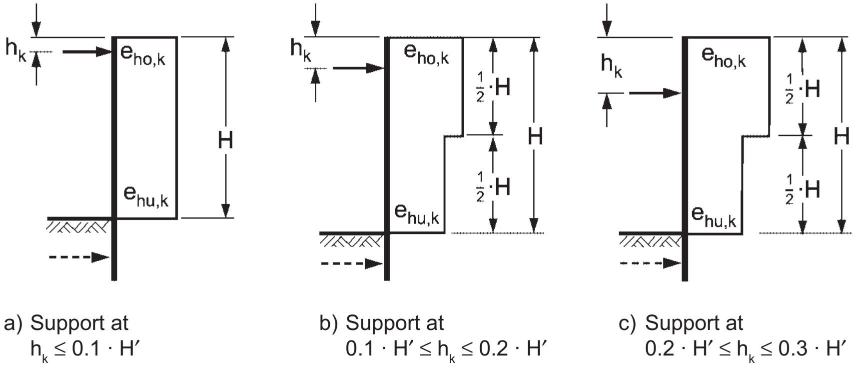


Figure R 70-1. Pressure diagrams for single-supported sheet pile walls and in-situ concrete walls

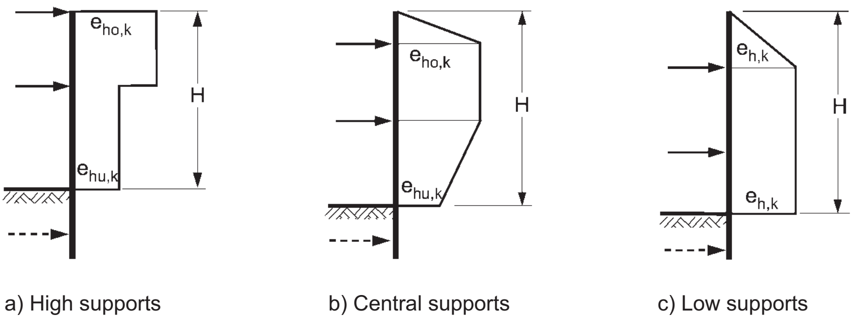


Figure R 70-2. Pressure diagrams for double-supported sheet pile walls and in-situ concrete walls

b) a pressure diagram according to *Lehmann* [9] with bending points at the support points and ordinate ratio $e_{ho,k} : e_{hu,k} = 2.00$ 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 chamfered rectangle as shown in Figure R 70-2 c), if both rows of struts or anchors are very low.

4. The pressure diagram according to *Lehmann* as shown in Figure R 70-3 may be assumed as realistic for triple- or multiple-supported 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 computed load resultant should be in the range $z_e = 0.40 \cdot H$ to $z_e = 0.50 \cdot H$.
5. The pressure diagrams recommended here do not take the previous construction stage into consideration. More precise definitions take the pressure

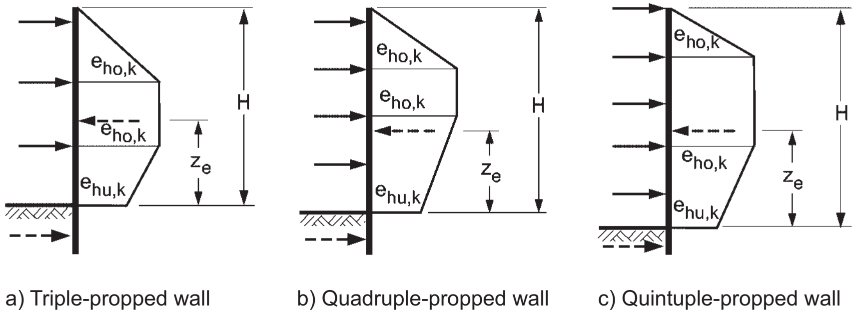


Figure R 70-3. Pressure diagrams for triple- and multiple-supported sheet pile walls and in-situ concrete walls

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 principally 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.

6. If earth pressure redistribution towards upwards 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 (R 19)

1. If the $\Sigma V = 0$ condition 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 according to *Caquot, Kerisel* and *Absi* [70] or according to DIN 4085, or fractured 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 used as the basis for analysis if the ground surface does not rise, the friction angle is not greater than $\varphi'_k = 35°$ and angle of inclination is reduced from $\delta_{p,k} = -\varphi'_k$ to $\delta_{p,k} = -{}^2/_3 \cdot \varphi'_k$.

   In the case of diaphragm walls a smaller inclination angle shall be selected 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.

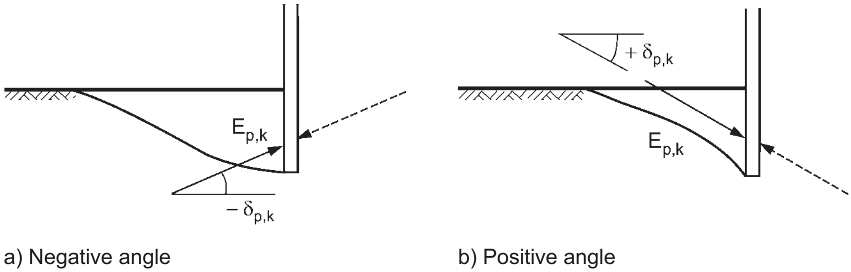


Figure R 19-1. Sign rule for the passive earth pressure inclination angle

2. The design passive earth pressure 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 γ_{Ep} according to 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 accepting the support force 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 displacements in soft, cohesive soils.
4. The point of acting of the resultant support force $B_{h,k}$ from the ground reaction $\sigma_{ph,k}$ for a wall with a free earth support may be assumed at $z' = 0.60\ t$ in the case of cohesionless soil and $z' = 0.50\ t$ 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 shown in Figure R 80-1 (Section 4.1), in the other case a rectangular distribution as shown in Figure R 16-1 c) (Section 6.1). 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 bending moments, shear forces and support forces may be based on:
   - either a reduced embedment depth t_0 or;
   - a partial earth restraint 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 analysed according to R 82, Paragraph 1 b) (Section 4.4) using the reduced partial safety factor $\gamma_{Ep,red} = 1.00$.
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 the information given in [94], [95], [126] and DIN 4085. The earth pressure stresses $e_{og,k}$ remaining from preconsolidation may be taken into consideration as shown in Figure R 102-2 (Section 4.6). Iteration shall be performed where necessary, until ground reaction and displacements fit together.

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 with alternating reinforced and unreinforced piles (secant pile wall), it shall be demonstrated that the wall can accept the loads and that the passive earth pressure can accept the support force.

6.4 Toe restraint for sheet pile walls and in-situ concrete walls (R 26)

1. If a sheet pile wall or an in-situ concrete wall embeds deeply enough below the excavation level, a geotechnical restraint can be applied under certain circumstances for determination of the action effects. This restraint may be taken into account with the aid of the *Blum* approach [23]. Supported and unsupported walls are to be differentiated:

   a) For unsupported walls in load-bearing ground the full geotechnical earth restraint 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 restraint depends on the deformation behaviour of the wall and the ground. In this case, a full geotechnical restraint 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 restraint in the ground is given 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 soil, a geotechnical restraint may only be adopted if the wall support reacts with great deformations.

2. A pressure diagram as shown in Figure R 26-1 b) is obtained for unsupported sheet pile walls and in-situ concrete walls restrained in the ground. The passive earth pressure design value is determined according to R 19 (Section 6.3). If the head deflections anticipated using this approach give cause for doubt,

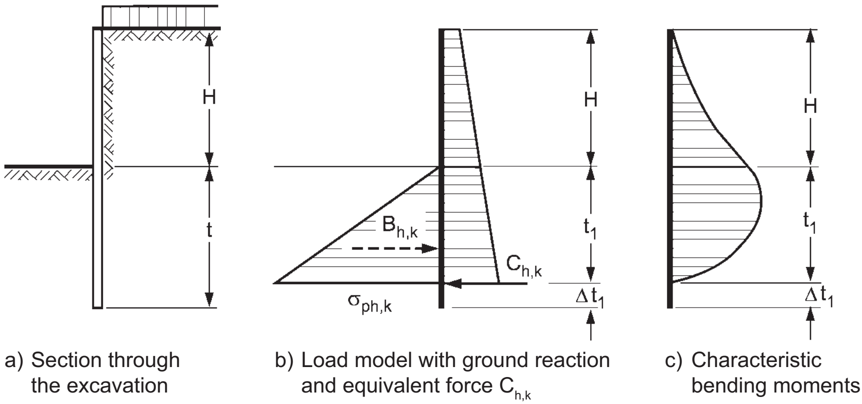
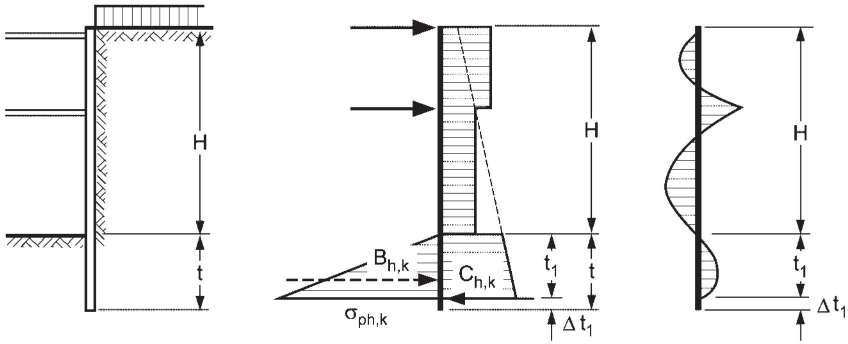


Figure R 26-1. System, load and moment distribution for an unsupported sheet pile wall or in-situ concrete wall restrained in the ground

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 selected, thus reducing utilisation of the ground reaction and, if necessary, a stronger section than determined by calculations also be selected. This is particularly the case if loosely compacted, cohesionless soil or only approximately stiff, cohesive soil is present in the restraint area. If necessary, serviceability shall be analysed again according to R 83 (Section 4.11) using the new dimensions. As a rule supported walls with only geotechnical restraint in the ground close to foundation loads and for excavations in soft, cohesive soils are not permissible due to the large anticipated deformations, see R 20 (Section 9.1) or R 101 (Section 12.12).

3. A pressure diagram as shown in Figure R 26-2 b) is obtained for supported sheet pile walls. It may generally be accepted for medium-dense or dense cohesionless soil, or at least stiff, cohesive soil, that the deformation conditions associated with a full restraint after *Blum* are approximately fulfilled. For loose cohesionless soil 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 restraint effect may not generally be applied for soft, cohesive soils or soils with a high organic content, see R 96 (Section 12.7).

4. Generally, it may be assumed that the fixed-end moment within the ground is not decisive for design of supported retaining walls in average soil conditions. The magnitude of the fixed-end moment need only be determined and used as the basis for design, if necessary, for sheet pile walls in very dense, cohesionless soils with high shear strength or for firm cohesive soils.



a) Section through the excavation

b) Load model with ground reaction and equivalent force $C_{h,k}$

c) Characteristic bending moments

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

5. Between the limit cases of full restraint and free earth support, intermediate cases with partial restraint are possible 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).

6. The embedment depth t_1, which is required for restraint of an unsupported sheet pile wall or in-situ concrete wall as shown in Figure R 26-1, 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 an augmentation of embedment depth of at least $\Delta t_1 = 0.10 \cdot t_1$ is required. The same applies to supported sheet pile walls as shown in Figure R 26-2, if the full restraint can develop in the ground. As an approximation for partial restraint the augmentation of embedment depth surcharge $\Delta t'_1$ may be linearly interpolated between the decisive full restraint value Δt_1 and the free earth support value $\Delta t_1 = 0$ as a function of the ratio $t'_1 : t_1$.

7. The more precise analysis stipulated in Paragraph 7 may be performed according to *Lackner* [2], [24]. Converted for the partial safety factor approach the following is obtained as shown in Figure R 26-3:

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

Where:

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

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

$$e_{phC,k} = (g_k + p_k) \cdot K_{pghC} + 2 \cdot c'_k \cdot \sqrt{K_{pghC}}$$

The following shall be observed:

a) The vertical stress g_k at the height of the theoretical toe is determined from the weight of the overlying strata, if necessary taking into account buoyancy if necessary.
b) The size and sign of the angle of inclination $\delta_{pC,k}$ is obtained from analysis of the vertical component of the mobilised passive earth pressure according to R 9 (Section 4.8), or from analysis of the transfer of vertical forces to the ground according to R 84 (Section 4.9). If both analyses are required, the more unfavourable case is decisive.

Note: The design value $C_{h,d}$ is determined here from the characteristic variables $C_{Gh,k}$ and $C_{Qh,k}$, which are obtained using the *Blum* load approach. In the EAU 2004, Recommendation R 56, Section 8.2.9 [2], half the value of these variables is assigned to these designations. See R 9, Paragraph 3 b) (Section 4.8) for an explanatory statement. Accordingly, the EAU 2004 lacks the factor 2 in the denominator of the equation for determining the embedment depth surcharge t_1. The result is the same as here.

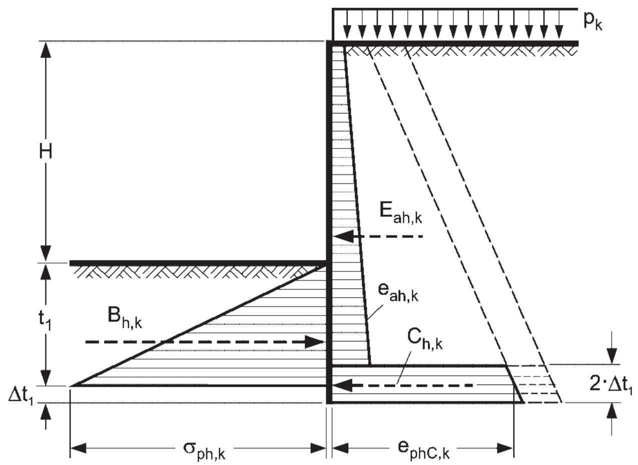


Figure R 26-3. Transfer of force $C_{h,k}$ at the toe of a wall restrained in the ground after *Lackner*

9. If necessary, determination of the degree of restraint can also be based on elastic bedding using a deformation resistance. See also R 102 (Section 4.6).

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

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

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

11. Analysis of the vertical component of the mobilised passive earth pressure shall be performed according to R 9 (Section 4.8), the transfer of vertical forces to the ground according to R 84 (Section 4.9).

12. See R 19, Paragraph 6 (Section 6.3) for staggering of the embedment length of 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 are principally a function of whether anchors are prestressed and, if so, with what force they are prestressed and fixed. An earth pressure distribution deviating from the classical earth pressure is generally only obtained, e.g. the pressure diagram as shown in Figure R 51 (Section 3.3), 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 step. 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 compelled, 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 analysis of the stability in a low failure plane according to R 44 (Section 7.3), general (global) stability analysis 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 such that a yieldingly supported wall will be generated and the classical earth pressure distribution is decisive, at least for relatively stiff walls. With regard to the adoption of cohesion and investigation of the live load influence, the same considerations apply as for unsupported retaining walls restrained in the ground and for yieldingly 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 braced excavations. The anchors shall be prestressed appropriately. If necessary, the variable flexibility of the support points shall be taken into consideration when determining action effects.

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5. It is generally permissible to prestress all anchors to 80% of the forces computed for the fully excavated state for active earth pressure design and to 100% for design with pressures greater than the active earth pressure, see R 22, Section 9.5. Only if these measures lead to overloading of the retaining 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 prestress the anchors in a first step corresponding to the characteristic anchor forces prevalent in the construction stage following anchor installation. For subsequent construction stages the anchors will be prestressed with higher loads accordingly.

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

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

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

   is fulfilled, i.e. if the sum $E_{d,i}$ of the action design values is as large as the resistance design value R_d, at its greatest.
3. The design value of the actions is composed of:

   a) the design value of the acting anchor force, obtained from 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 anchored wall or anchor plate.

   The resistance design value R_d is determined according to Paragraphs 4 to 6 below.
4. For determination of the characteristic passive earth pressure $E_{p,k}$ in front of continuous anchored 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 weight density of the wall. Otherwise, the influence of anchor inclination shall be taken into consideration, in particular for anchors that slope down to the anchor-wall. If the anchored wall is covered with soil, the approximate passive earth pressure may be determined similar to a wall beginning at ground level. The passive earth pressure design value is obtained from:

   $R_d = E_{p,d} = E_{p,k} / \gamma_{Ep}$
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, not more than the plan horizontal

passive earth pressure $E_{p,k}$ determined using $\delta_p = 0$ may be adopted for a small spacing *a* of the anchor plates:

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

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

$E^*_{p,d} = E^*_{p,k} / \gamma_{Ep}$

6. The characteristic pull-out resistance R_k of tension piles and ground anchors is obtained according to R 86 (Section 13.11).

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

   – $R_d = R_k / \gamma_{Pt}$ for tension piles or
   – $R_d = R_k / \gamma_A$ for grouted piles

7.3 Analysis of stability at low failure plane (R 44)

1. A stability analysis at a low failure plane is required for anchored retaining walls. It serves to determine the necessary anchor length. One must imagine 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, the anchor length is first selected and then the stability is analysed.

2. The analysis model described below is based on *Kranz's* method [99]. This was originally derived for single-anchored walls utilising a free earth support with anchored walls and flexible 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.

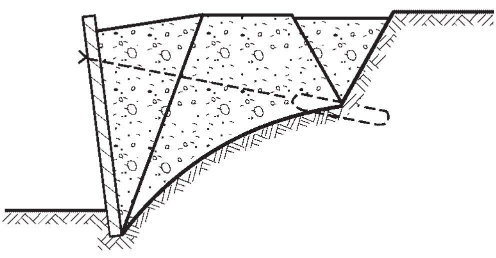
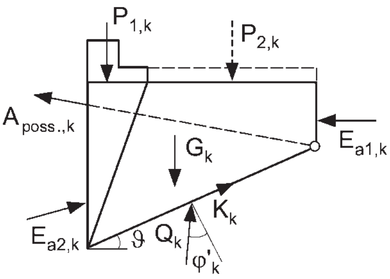
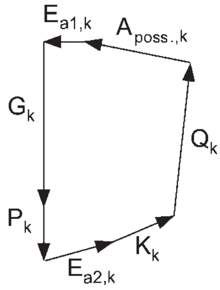


Figure R 44-1. Failure in a low plane after *Kranz* [99]



a) Forces acting on the slip body



b) Force polygon (not to scale)

Figure R 44-2. Determination of the resistance $A_{poss.,k}$ when analysing stability in a low failure plane

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 cut. The forces acting on the soil prism as shown in Figure R 44-2 a) are composed of the actions according to Paragraph 7 and the ground reactions in the low failure plane according to Paragraph 8. These divisions do 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 $A_{poss.,k}$. The details are described in the following paragraphs.

4. When analysing the stability of a low failure plane consideration should be given to whether all the soil between the anchors participates in the formation of a soil prism as mentioned in Paragraph 1:

   a) For anchored walls, anchor plates and tension piles with spacing *a* smaller than half the force transmission zone l_r, sufficient stability is given if the condition for the STR limit state:

      $A_{working,d} \leq A_{poss.,d}$

      is fulfilled.

      Where:

      $A_{working,d}$ the anchor load design value
      $A_{poss.,d}$ the resistance design value

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

      $A_{h,working,d} \leq A_{h,poss.,d}$

b) If the spacing *a* is greater than half of the force transmission zone l_r for ground anchors, the possible anchor force $A_{poss.,k}$ shall be reduced to:

$A_{poss.,red,k} = ½ \cdot A_{poss.,k} \cdot l_r / a$

according to the EAU, Recommendation R 66 [2]. The limit equilibrium condition is then decisive:

$A_{working,d} \leq A_{poss.,red,d}$

5. The anchor load design value is determined from:

$A_{working,d} = A_{working,G,k} \cdot \gamma_G + A_{working,Q,k} \cdot \gamma_Q$

the resistance design value from:

$A_{poss.,d} = A_{poss.,k} / \gamma_{Ep}$

The variables $A_{working,G,k}$ and $A_{working,Q,k}$ are obtained from determining the action effects on the retaining wall. If all changeable actions over and above the unbounded distributed load $p_k \leq 10$ kN/m² are increased by the factor f_q according to R 104, Paragraph 5 (Section 4.12), determination of the load design value is simplified.

6. The following apply to the rearward boundary of the sliding earth body:

a) for continuous anchored walls, a plane from the toe of the anchored wall, reaching vertically to the ground surface is decisive;
b) for individual anchor plates an equivalent anchored wall shall be assumed at a distance $½ \cdot a_1$ in front of the anchor plates, where a_1 represents the clear distance between the anchor plates;
c) for tension piles the equivalent anchored wall shall be assumed at the centre of the effective or computed force transmission zone, see EAU, Recommendation R 10 [2];
d) for ground anchors the equivalent anchored wall shall be assumed at the centre of the planned force transmission zone.

The low failure plane of anchored walls and anchor plates is taken from their lower edge, for tension piles and ground anchors from the centroid of the force transmission zone. The shear force point of zero stress is decisive for anchoring elements loaded at the head and restrained in the ground.

7. The toe of the low failure plane is assumed at the base of the wall or soldier pile for walls with a free earth support. Otherwise, the following 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 is necessary to accept the horizontal support force in order to accept vertical loads, or for any other reason, the base is then taken as the depth which would be sufficient without considering the vertical loads.

c) If retaining wall embedment is dispensed with, either in reality or merely for analysis according to R 15, Paragraph 6 b) (Section 5.5), and thus also with a support below the excavation level, the toe is assumed at the depth at which the design earth pressure acting below the excavation level can be accepted 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 heavily loaded by water pressure despite the wall being lengthened:

– for buoyancy safety;
– to limit seepage forces or;
– to seal the excavation;

the actual wall toe is adopted according to [96] as the start of the low failure plane. This does not apply if the walls are braced at the earliest opportunity at the height of the excavation level, e.g. by an underwater concrete base, or at the height of the wall toe, e.g. by a deeper jet grouted base.

e) For full or partial geotechnical restraint and elastic support of the wall in the ground the shear force point of zero stress is taken as the toe.

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

a) The earth pressure force $E_{a1,k}$ is obtained using the same characteristic soil parameters as are used to determined 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 ground anchors and tension piles $\delta_a = \beta$. Analysis may be performed with $\delta_a = {}^2/_3 \cdot \varphi'_k$ for anchored walls and anchor plates.

b) The characteristic load G_k from soil weight density is obtained from the geometrical dimensions of the slip body and the same values of bulk density adopted for determination of the earth pressure force $E_{a2,k}$.

c) The changeable action P_k is composed of two elements:

- The changeable action $P_{1,k}$ is the sum of the live loads adopted for determining the earth pressure $E_{a2,k}$ and the anchor force $A_{working,k}$. As shown in Figure R 44-2 a), this is the proportion of live loads acting on the active failure wedge. This is generally bounded by a slip surface with the angle $\vartheta_{a,k}$. A slip surface with the angle $\vartheta_{z,k}$ can be decisive:
  – for yieldingly supported walls according to R 6, Paragraph 5 (Section 3.4) and;
  – for excavations adjacent to structures according to R 28, Paragraph 12 b), (Section 9.3).

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

The action P_k as shown in Figure R 44-2 b) corresponds to the force $P_{1,k}$ when $\vartheta \leq \varphi'$ or the sum of von $P_{1,k}$ and $P_{2,k}$ when $\vartheta > \varphi'_k$.

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

a) If applicable, the characteristic cohesive force K_k is obtained from the current cohesion c'_k with the slip surface length L using:

$K_k = c'_k \cdot L$

b) The characteristic reaction force Q_k in the low 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 $A_{poss.,k}$ 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, corresponding to Figure R 44-3:

a) Each centre point of a force transmission zone shall be assumed to be the end point of a low failure plane once in every construction stage.
b) The actions $A_{working,k}$ of all anchors whose force transmission zones are within the slipping earth prism or within the active failure wedge resulting in the earth pressure force $E_{ag1,k}$ are adopted as the characteristic forces.
c) The forces of anchors whose force transmission zones are intercepted by the low failure plane may be divided into a component in front of the intersection and a component behind it, assuming uniform skin friction

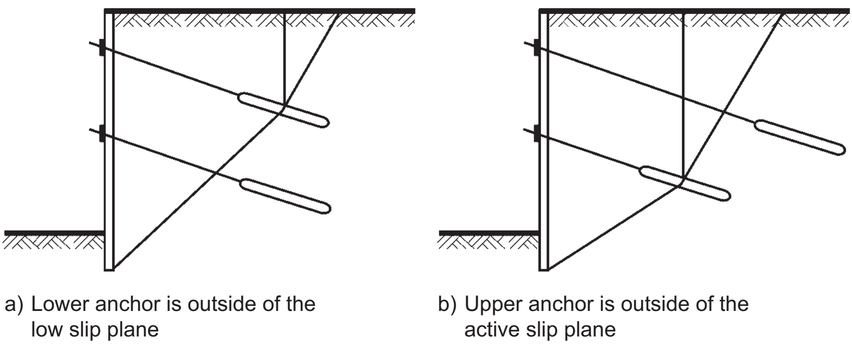


Figure R 44-3. Example of anchors whose forces are not taken into consideration as actions distribution in the force transmission zone. The proportion of the anchor force transferred within the slip body shall be treated as an action. The same applies to the forces of anchors intercepted by the active slip surface behind the equivalent anchored wall.

d) If, in exceptional cases, the earth pressure from the equivalent load $P_{1,k}$ according to Paragraph 7 c) as a result of continuity effect unloads the anchor adopted as the end point of the low failure plane, an investigation shall also 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 necessary, greater than the mean.

11. For anchored retaining walls designed for increased active earth pressure, or for reduced or full at-rest earth pressure, the stability of the low 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) In place of the active earth pressure $E_{a2,k}$ the force polygon as shown in Figure R 44-2 b) includes the earth pressure force $E_{2,k}$ determined according to the information given in R 22 (Section 9.5) and R 23 (Section 9.6).

b) In place of the active earth pressure $E_{a1,k}$ the force polygon as shown in Figure R 44-2 b) includes the earth pressure force $E_{1,k}$. 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 changeable loads and for resistances may be linearly interpolated according to R 22, Paragraph 3 (Section 9.5) between:

– the partial safety factors for Load Case LC 2 when adopting active earth pressure and;
– the partial safety factors for Load Case LC 1 when adopting at-rest earth pressure.

Because this interpolation only has a minor impact on the one hand, but on the other hand analysis of the stability of the low failure plane is highly sensitive to inaccuracies, it is generally recommended to analyse using the partial safety factors for Load Case LC 1, if the retaining wall was designed with increased active earth pressure.

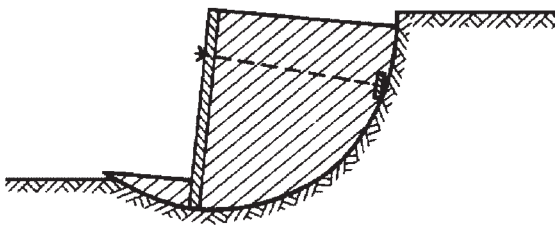
12. See EAU 2004, recommendation R 10 [2] for taking alternating soil layers and positive water pressure into consideration.

7.4 Analysis of global stability (R 45)

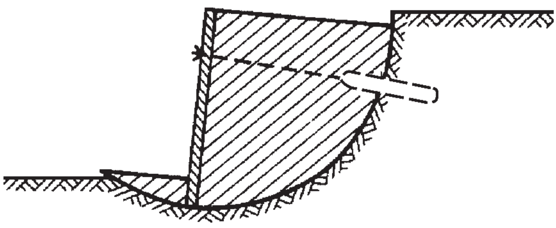
1. In principle, global stability shall also be analysed for anchored retaining walls. However, empiricism demonstrates that it is sufficient to limit this analysis to exceptional cases. Greater dimensions or greater anchor lengths may occur, for example:

   a) for large ground surcharges in the region of grouted sections;
   b) if a soil layer is present below the toe displaying a shear strength lower than that of the overlying layers;
   c) for retaining walls that do not reach as far as, or only slightly lower than, the excavation level;
   d) if the rear of the wall is heavily inclined towards the ground;
   e) if the ground behind the wall climbs;
   f) if the ground in front of the wall declines.

2. When analysing global stability one imagines that the anchors fix the retaining wall to the ground behind the wall to form a monolithic structure which slides on a curved slip surface (Figure R 45-1). Here, in contrast to analysis of the stability of the low failure plane, the wall toe moves further forwards than the top of the wall. This is associated with rotation of the monolithic body around a higher point. Analysis of global stability is one of the GEO limit states according to R 78, Paragraph 5 (Section 1.4). Sufficient global failure safety is given if the limit state condition:



a) Anchorage with a dead man



b) Anchorage using grouted anchors

Figure R 45-1. Global failure for a single-anchored wall

$M_{S,d} \leq M_{R,d}$

is adhered to, i.e. if the sum $M_{S,d}$ of the design values of the acting torques is no greater than the sum $M_{R,d}$ of the resisting torques.

3. In terms of the GEO limit state the design values of the decisive 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 acting loads with 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 γ_φ and γ_c as shown in Table 6.3 in Appendix A 6.

4. Analysis of global stability must generally be performed with circular slip surfaces. Only in well-substantiated cases, e.g. if development of a circular slip surface penetrating deep in the ground is hindered by differing shear strengths or the inclination of the soil layers, may it be necessary to assume slip surfaces formed by 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.

5. In principle, the decisive failure mechanism is influenced by two points:

   a) At the top, the end of the anchor construction is decisive. For anchored walls and anchor plates the decisive slip surface contacts the rear face of the anchor construction, see Figure R 45-1 a). For tension piles and ground anchors it is sufficiently precise and generally conservative enough to assume the centroid of the force transmission zone as the effective end point of the anchor as shown in Figure R 45-1 b) according to R 44, Paragraph 5 d) (Section 7.3).
   b) At the bottom, the decisive 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 decisive slip surface can also be deeper.

   For a more precise investigation of tension piles and ground anchors according to DIN 4084, those failure mechanisms completely enclosing the force transmission zone and those intersecting the force transmission zone 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 observed:

a) The torque 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 shall be taken into consideration.

b) If the anchor is intersected in the force transmission zone the effective axial force may be divided accordingly, assuming uniform skin friction distribution in the force transmission zone. Only that proportion of the force transmitted 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 transmitted 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 global failure in the intersected structural components may also be adopted. However, these shear forces may only be adopted:

   – 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 transmitted to the ground by the intersected component without 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 global stability, retaining walls designed for active earth pressure only require the partial safety factors given for Load Case LC 2. The following shall be observed for greater demands on serviceability:

   a) The partial safety factors stipulated for Load Case LC 1 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 Load Case LC 2 when adopting active earth pressure and the partial safety factors for Load Case LC 1 when adopting at-rest earth pressure, according to 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 Load Case LC 1, if the retaining wall was designed with increased active earth pressure.

7.5 Measures to counteract displacements in anchored retaining walls (R 46)

1. As is demonstrated by empirical data, displacements 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. Decisive in this respect are the displacements and deformations of the soil body which is enclosed by the retaining wall, similar to inside a cofferdam, and a plane that connects the points assumed to transmit the anchor forces to the ground according to R 44, Paragraph 7 (Section 7.3) (Figure R 46-1). According to R 83, Paragraphs 8 to 11 (Section 4.11), the displacements principally consist of:

   a) elastic deformation of the wall;
   b) tilting of the cofferdam-like soil body;
   c) shear deformation of the soil body 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 displacement of the ground surface are associated with the movement of the cofferdam-like soil body.

2. For excavations in water the toe of the wall can move more than usual if the effective vertical stress resulting from seepage pressure below a soft gel grout blanket as shown in Figure R 62-1 c) (Section 10.5) is reduced within the soft gel grout blanket. The ground reaction force $B_{h,k}$ is then transmitted by the soil layer above the soft gel grout blanket to the opposite side of the retaining wall and not to the deeper subsurface. The entire soil layer above the soft gel grout blanket is thus subject to compaction resulting from the ground reaction force $B_{h,k}$, which may initiate a correspondingly large displacement of the wall toe [96].

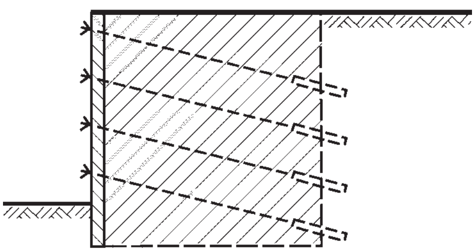


Figure R 46-1. Development of a cofferdam-like soil mass

3. If the serviceability analysis according to R 83 (Section 4.11) 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 datum points [29];
   d) sectionwise step by step manufacture of excavation and structure.

   Bracing shall be designed for substantially higher loads than would correspond to the proportion of the computed earth pressure, e.g. for the double load. If necessary, the observational method according to DIN 1054 shall be adopted.

4. Regardless of the measures implemented according to Paragraph 2 it is recommended to expand and stagger the anchors adjacent to other structures and to stagger the lengths in the force transmission zone, if the grouted sections are not located within competent rock. Expanding can also reduce the mutual influence of the anchors. By staggering the anchor lengths, the danger of sudden jumps in the settlement values behind the cofferdam-like monolith can be eliminated and a widened and shallower settlement trough can achieve.

   In the case of staggering, every second anchor shall be lengthened, retaining the original grouted section. If the staggering lies in an area where a sudden jump in settlement values needs to be avoided due to 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. The local conditions and requirements dictate to which rows of anchors the extensions are distributed across.

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 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 applying especially high prestressing. Such prestressing merely has the effect of generating internal stress conditions between the retaining wall and the grouted sections, which hinders formation of an active earth pressure failure wedge and decompaction of the ground. High prestressing, on the other hand, can lead to heavy lateral compaction of the soil mass, damage to cellar masonry and to 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 due to weight density and unbounded distributed loads is insignificantly different from 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 significantly less than 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 weight density and unbounded distributed loads depend 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 pile walls can be regarded as non-yielding systems if they form an unbroken circle and simultaneously serve as a ring beam. A precondition for this is that the ground cannot relieve whilst manufacturing the retaining wall.
   b) Retaining walls can be regarded as nearly inflexible if they possess a certain inherent flexibility, e.g. sheet pile walls and secant pile walls, but are supported by stiff ring beams.
   c) Generally, all retaining walls in which the ground face is open before the infill are installed, 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.

   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 annular load transmission.

3. The following apply for the determination of earth pressure:

   a) The at-rest earth pressure E_0 may be applicable as the upper limit value for inflexible systems according to Paragraph 2 a). An earth pressure of $E = ½ \cdot (E_0 + E_{aR})$ can be assumed as the lower limit. E_{aR} designates the

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three-dimensional active earth pressure according to the modified disk-element theory after *Walz* and *Hock* [81, 82].

b) The upper limit value for nearly inflexible systems according to Paragraph 2 b) can be taken as an earth pressure of $E = ½ \cdot (E_0 + E_{aR})$ 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 non-yielding systems according to Paragraph 2 c) can be taken as an earth pressure of 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 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 most 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 transmit 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 minimum earth pressure.

4. It can be assumed that the earth pressure distribution deviates only slightly from a linear depth increase 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 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.

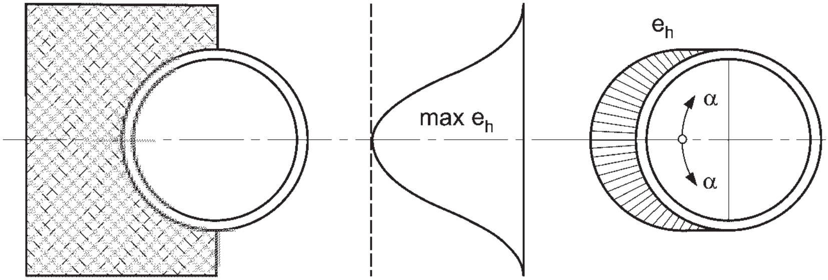


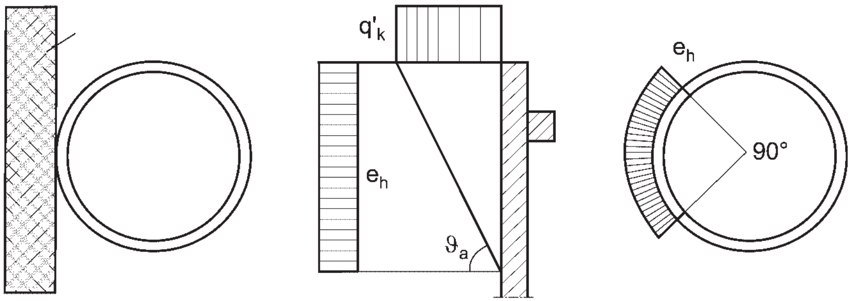
Figure R 73-1. Earth pressure from a bounded distributed load $p_k = 10$ kN/m^2

5. Unexpected 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. As an 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. $e_h = \max.\ e_{0ph} = p_k \cdot K_{0h}$ approach, in the case of active earth pressure from the max. $e_h = \max.\ e_{aph} = p_k \cdot K_{ah}$ 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.

   The recommended approach covers geometrical imperfections in structures without a ring beam with an oval plan and a maximum deviation of the A and B principle axis dimensions of A : B ≤ 1.03. Adherence to this condition shall be examined by on-site measurements. If the centres of secant pile walls or the longitudinal axes of individual diaphragm wall slices do not coincide with a possible pressure line, the imperfection shall be corrected or compensated for by design 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 decisive plane. As an approximation,



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

the earth pressure determined can be adopted for one quarter of the circumference of the excavation as a radially acting load e_h as shown in Figure R 73-2 c).

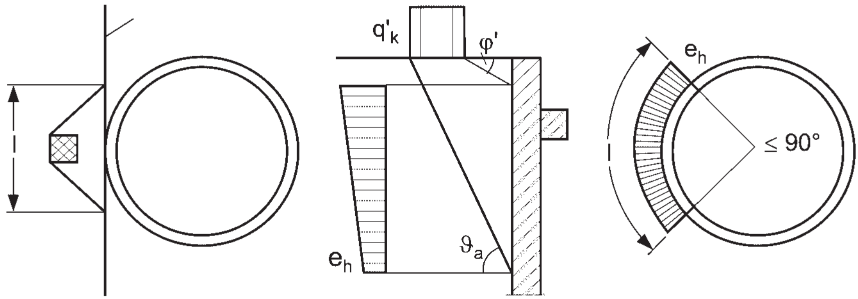
b) If the load is represented by point loads according to R 55 (Section 2.6) or R 57 (Section 2.8), as shown in Figure R 73-3 a), the earth pressure shall be determined according to R 6 (Section 3.4) and R 7 (Section 3.5), as if a fictitious plane at a tangent to the circular excavation structure is the decisive 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. As an approximation the earth pressure determined 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 circle circumference 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 weight density 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 theory of elastic half-space; if a value between at-rest earth pressure and active earth pressure is adopted for the earth pressure from soil weight density, this also applies for the earth pressure from live loads.

7. When determining the earth pressure from foundation loads for excavations adjacent to structures, the information given in Paragraph 6 applies accordingly:

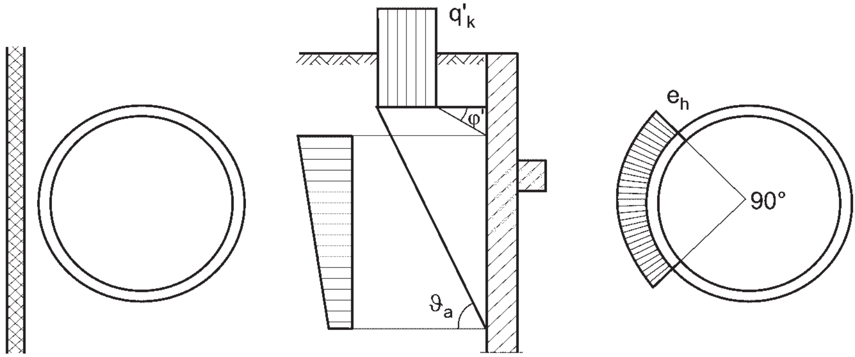
   a) The load distribution and load length at the circumference of the excavation shall be assumed as shown in Figure R 73-3 for footing foundations.
   b) The earth pressure determined from strip loads shall be applied to a quarter of the circumference as shown in Figure R 73-4 c).

   Otherwise, please observe Chapter 9.



a) Load in plan b) Load in section (example) c) Earth pressure in plan

Figure R 73-3. Earth pressure for a point load



a) Load in plan b) Load in section (example) c) Earth pressure in plan

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. As an 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 modulus of subgrade reaction method was employed and no precise investigations were carried out, the modulus of subgrade reaction may be approximately determined using $k_{s,k} = E_{S,k} / r$ from

the constrained modulus of the ground and the outer radius of the excavation structure. 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. As an 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 considering the 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 ratio A : B 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 the radius 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).

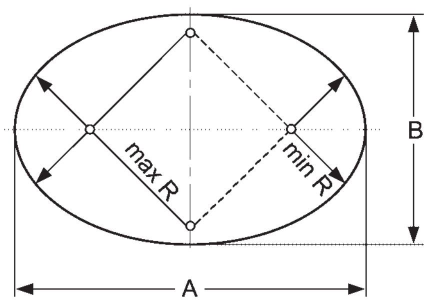


Figure R 74-1. Excavations with elliptically curved plan

2. The magnitude and distribution of earth pressure from soil weight density and unbounded distributed loads depend on the type of construction project, 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 pile walls can be regarded as nearly inflexible systems if they form an unbroken arc and simultaneously serve as a ring beam. A precondition for this is that the ground cannot relieve whilst manufacturing the retaining wall.
   b) Retaining walls can be regarded as non-yielding if they possess a certain inherent flexibility, e.g. sheet pile walls and secant 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 rings or other measures or not at all can be regarded as yielding systems, in particular soldier pile walls with timber infill walls.

3. The decisive 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. As an 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 10.

4. The following apply for the determination of earth pressure in the areas with the larger curve radius:

   a) The upper limit value for nearly inflexible systems according to Paragraph 2 a) can be taken as an earth pressure of $E = ½ \cdot (E_0 + E_{aR})$ and the lower limit as the three-dimensional earth pressure E_{aR} according to the modified disk-element theory after *Walz* and *Hock* [81, 82].
   b) The upper limit value for non-yielding systems according to Paragraph 2 b) can be taken as an earth pressure of 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, the 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 justify this.

g) For retaining systems that cannot transmit 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), applies with regard to 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 design of individual components of the excavation structure, an unbounded distributed load of at least $p_k = 10\ kN/m^2$ similar to Recommendations R 55 to R 57 (Sections 2.6 to 2.8) shall be adopted as traffic or operating loads. 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, whereas, 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.

7. If traffic or operating loads exceed the unbounded distributed load $p_k = 10\ kN/m^2$ according to Paragraph 6, only the actual load positions need be taken into consideration. Two cases may be considered:

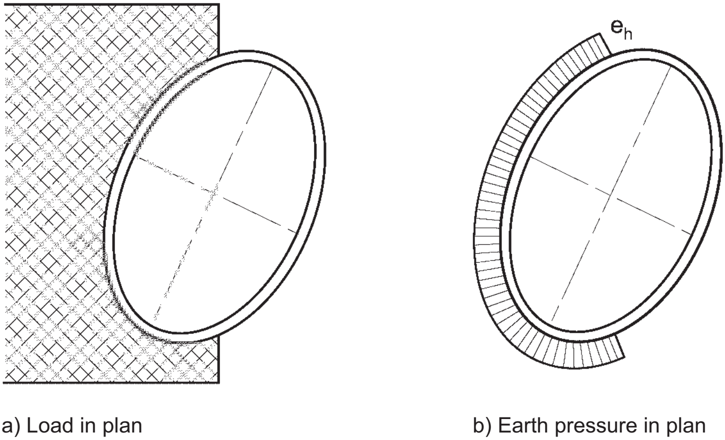
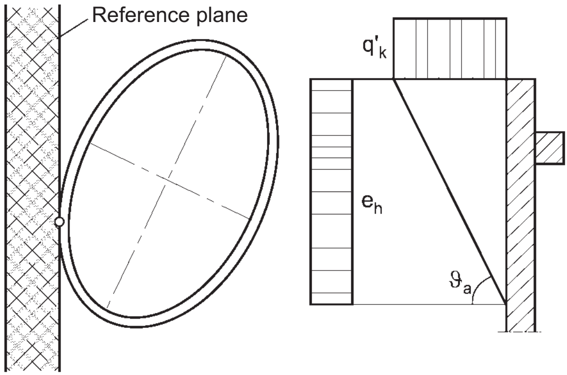


Figure R 74-2. Earth pressure from a bounded distributed load $p_k = 10\ kN/m^2$

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 a fictitious plane at a tangent to the circular excavation structure is the decisive plane. As an approximation, the determined earth pressure can be adopted as a radially acting load e_h as shown in Figure R 74-3 c), 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.

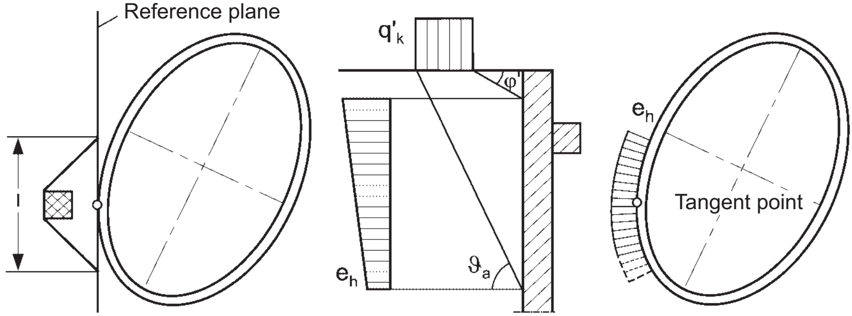
b) If the load is represented by point loads according to R 55 (Section 2.6) or R 57 (Section 2.8), as shown in Figure R 74-4 a), the earth pressure shall be determined according to R 6 (Section 3.4) and R 7 (Section 3.5), as if a fictitious plane at a tangent to the circular excavation structure is the decisive 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. As an 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), but for a maximum of one quarter of the circumference, if it acts unfavourably.

If the earth pressure from soil weight density 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 weight density, 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 74-3. Earth pressure from a strip load q'_k



a) Load in plan b) Load in section (Example) c) Earth pressure in plan

Figure R 74-4. Earth pressure from a point load

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 loads 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 Chapter 9.

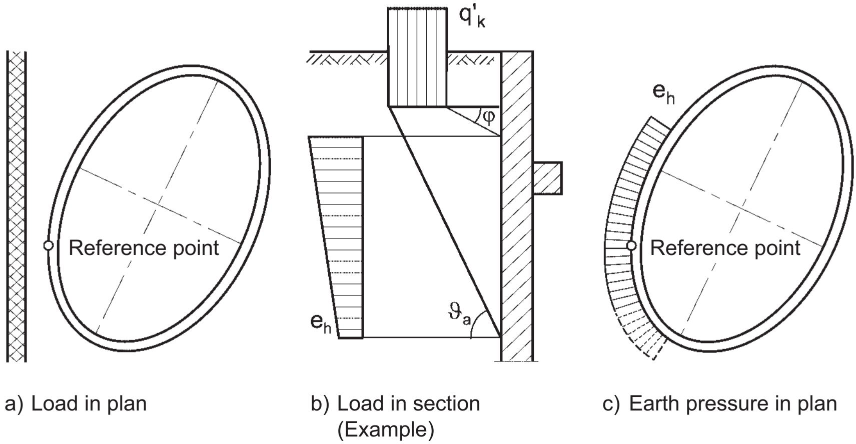


Figure R 74-5. Earth pressure from a strip footing

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 3. The same applies if the earth pressure from bounded surcharges according to Paragraphs 6 to 8 acts on one side in the region of a large radius curve. As an approximation in such cases, earth pressure of equal 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. 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 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 constrained modulus of the ground and the decisive 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 action effects for the decisive load combinations.
10. Ground reactions resulting from the subgrade may not be adopted at the edge of access openings in the retaining wall. As an 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 at the serviceability limit. 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 considering the normal force. A stability investigation may generally be dispensed with if the contact with the retaining wall prevents ring deflection.

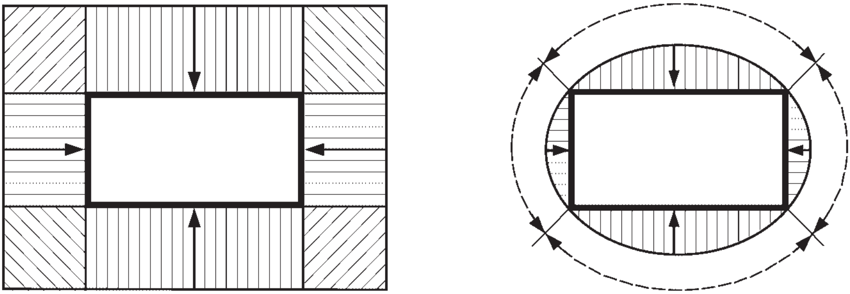
8.3 Excavations with rectangular plan (R 75) (11/05)

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, 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.

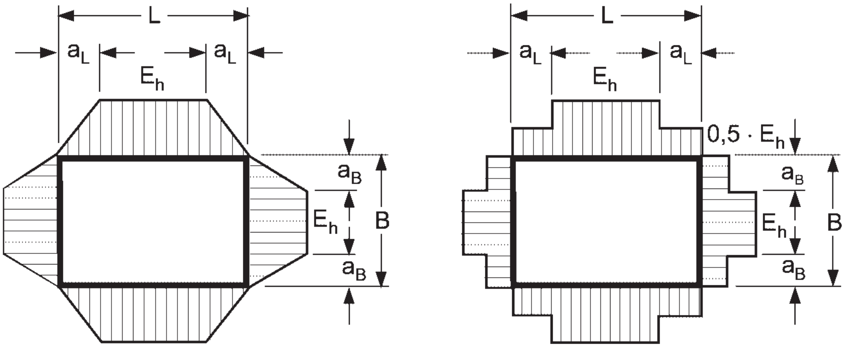
   The procedures suggested here assume an excavation structure similar to R 67 (Section 1.5), which is either not supported, yieldingly supported or slightly yieldingly supported but is sufficiently deformable, in order to facilitate reduction of the at-rest earth pressure to the active earth pressure. Where these displacements are hindered at the excavation corners for diaphragm walls and secant 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 and thus prevent slippage of the earth masses towards the excavation walls and reduce the total active earth pressure. Only procedures that do not overestimate the magnitudes



a) Lateral friction model b) Arching model

Figure R 75-1. Models for determination of the three-dimensional earth pressure for rectangular excavations



a) Earth pressure with chamfering b) Earth pressure with steps

Figure R 75-2. Simplified earth pressure for earth pressure reduction at excavation corners

of these forces may be selected. 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 of 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.

3. For those procedures that assume a three-dimensional failure body, the development of an arching effect as shown in Figure R 75-1 b) plays a decisive role for earth pressure reduction. Suitable procedures are those after *Karstedt* [53]

and *Piaskovski* and *Kovalevski* [87], or procedures according to DIN 4085. The procedures based on three-dimensional sliding bodies 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 mentioned above, can be implemented in the design of the individual components as shown in Figure R 75-3 a) in the form of chamfering, 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.

4. In Paragraphs 2 and 3, E_h designates the earth pressure on a continuous wall from soil weight density, 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 without further analysis as shown in Figures R 75-2 b) and R 75-3 b) to ½ · E_h. The wall lengths for which a reduction may be applied follow from *Walz* [88], as a function of the depth H as follows

   $a_L = (0.35 - 0.06 \cdot H : L)$ on those sides of length L;
   $a_B = (0.35 - 0.06 \cdot H : B)$ 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.

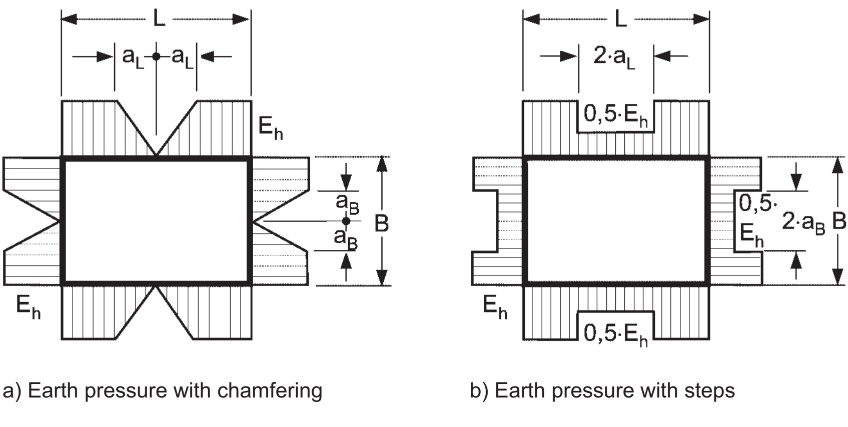


Figure R 75-3. Simplified earth pressure for earth pressure reduction on excavation sides

6. If the excavation length for which the total earth pressure E_h may be reduced results in $2\ a_L > L$ or $2\ a_B > B$ 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 on the sides of the excavation follows from Paragraphs 2 and 3 as one of the shapes represented in Figure R 75-4. When deciding on one of these shapes, the flexibility of the supports is decisive. The largest earth pressure should be anticipated where the displacement is smallest. The same applies accordingly for the longer sides of 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 weight density according to R 22 (Section 9.5).

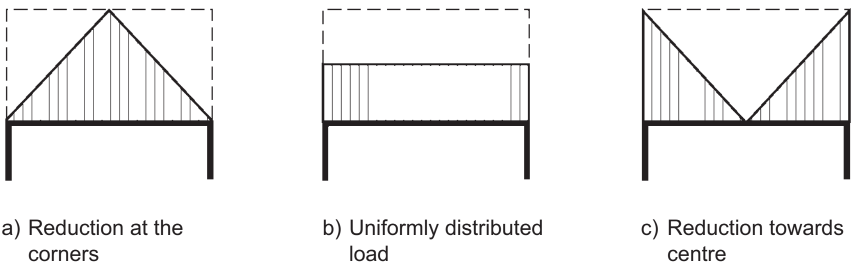


Figure R 75-4. Earth pressure on narrow excavation sides and shaft-like excavations

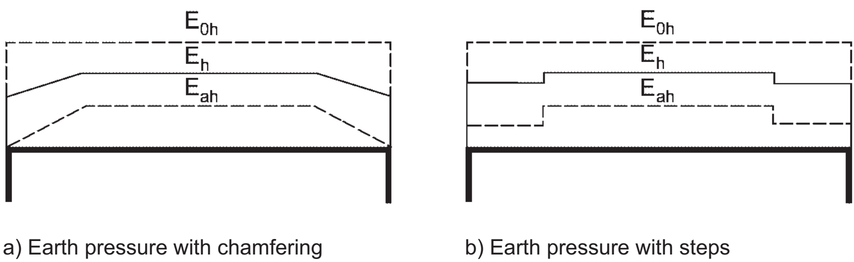


Figure R 75-5. Earth pressure in rectangular excavations with increased active earth pressure and earth pressure reduction at the excavation corners

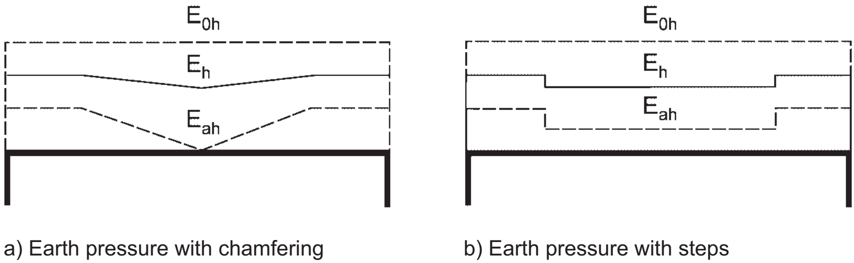


Figure R 75-6. Earth pressure in rectangular excavations with increased active earth pressure and earth pressure reduction at the excavation sides

Paragraphs 2 to 5 are decisive for applying the active earth pressure in the region of structures according to R 28 (Section 9.3) or according to R 29 (Section 9.4).

8. 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 reductions.
9. The earth pressure from point loads, line loads or strip loads from 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 33 (Section 9.5) and R 23 (Section 9.6), may not be reduced.
10. 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 a structure is 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 structure, and the ground conditions. Furthermore, for braced excavations, the elastic deformations, slippage and local deformations also play a role, in particular for long sets of struts consisting of a large amount of individual components. In particular, struts or anchors are preferentially positioned in the foundation load projection 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 area 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 under the following conditions:

   a) As a rule, it is generally necessary to eliminate as far as possible ground displacement caused by bending of the infilling or from the development of voids behind the infilling in the region below the foundation level by adopting appropriate measures. In suitable, temporarily stable ground, this can be achieved by excavating from soldier pile to soldier pile with the help of a template and by prestressing the individual components of the infilling to achieve a predefined curvature.
   b) These measures can be dispensed with if the infilling is manufactured using in-situ concrete for temporarily stable, cohesive soil.
   c) It may be expedient to install trench sheet piles as infilling in less stable ground and to wedge them against the waling to anticipate the computed deformations of the trench sheet piles and waling.
   d) For soldier piles installed in boreholes it is necessary to backfill the boreholes with a minimum of voids. The backfill material shall either be compacted or stabilised by means of a bonding agent.

3. If it is not possible or expedient to install a soldier pile wall, e.g.:

   – in uniformly grained and thus pronouncedly cohesionless soil;
   – in soft, cohesive soils and soils with a tendency to flow;
   – if dewatering is not desirable;
   – for small retaining wall-structure distances or;
   – for particularly sensitive structures;

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the installation of watertight and especially low-deformation retaining walls may be necessary, e.g. sheet pile walls, diaphragm walls or bored pile walls. It may be expedient in special cases to underpin the structure completely or in part, or to apply soil stabilisation measures.

4. When selecting the excavation lining it should be noted that not every system is equally suitable due to influences arising from the manufacturing 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 flow, settlement in the immediate area can ensue due to the soil being pushed into the void created by the drill bit projection. Also see R 92, Paragraph 3 (Section 12.3).
   c) In slurry-supported diaphragm wall trenches, the intersection of voids, e.g. pipelines, can lead to a loss of slurry; damage to adjacent structures may occur as a result of the lower trench support. As a safeguard against lowering of the slurry level it may be necessary to hold a sufficient quantity of slurry ready, as well as implementing counter-measures according to R 92, Paragraph 3 (Section 12.3). For self-hardening diaphragm walls the deformation with time resulting from the setting process shall be observed. This shall be taken into consideration when specifying the slice sequence. R 92, Paragraph 4 (Section 12.3) shall be taken into consideration for the manufacture of diaphragm walls.

   Each case shall be examined individually to determine the suitability of the construction method and manufacturing process.

5. In order to keep the anticipated wall displacement as small as possible it is expedient to:

   – select flexurally stiff sections or thick 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 greater than 80% of the characteristic stress computed for the subsequent construction stage;
   – if necessary, replace anchors with prestressed struts or other bracing structures.

   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 expedient 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 cannot negatively impact the structure. See also R 46, Paragraph 1 (Section 7.5) and [29], [39] and [72].
7. It may be expedient to carry out stabilising measures on the structure itself, regardless of any 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, as well as brickwork in openings and installing 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 to cases in which sensitive plant or installations may be endangered by manufacturing the excavation. For example, such installations can be:
   a) Railway installations, in particular badly positioned tracks or high train travelling speeds.
   b) Pipes without longitudinal tensional lock, in particular associated with brittle material.
   c) Water or gas pipes, in particular with large diameters and breaked routes.
   d) Masonry sewer pipes, in particular old or damaged pipes.
   e) Masts for illumination installations, signalling installations, electricity or overhead cables of rail vehicles, in particular if they are eccentrically loaded and restrained in the ground.

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 directly behind the retaining wall that are twice as large as the horizontal wall deflections and only fade at large distances from the excavation may be associated with this wall deflection. If a structure is located within this region, it shall be assumed that the resulting settlements will impact the foundations. As far as these settlements can be accepted, taking the condition and sensitivity of the structure into consideration, the excavation structure may be designed for active earth pressure.
2. Generally, for excavations adjacent to structures, 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:

   - the earth pressure $E_{ah,k}$ from soil weight density, unbounded 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;
   - the 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 4.12) 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 distances to structures, see R 28 (Section 9.3);
   b) small distances to structures, see R 29 (Section 9.4).

   The decisive factor for differentiation is whether a straight line touching the front edge of the foundation is 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 weight density and cohesion alone. The toes of these straight lines are regarded as:

   a) the real toe of the wall for soldier pile walls with free earth supports, sheet pile walls and in-situ concrete walls;

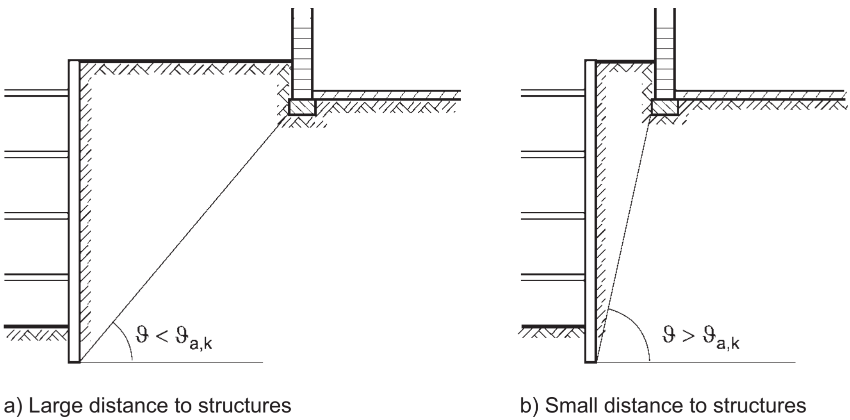


Figure R 21-1. Distance between retaining wall and structures

b) the theoretical toe of the wall for restrained soldier pile walls, sheet pile walls and in-situ concrete walls.

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 $\Sigma H = 0$ equilibrium condition 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. Application of passive earth pressure when analysing the embedment depth is:

   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.

   See R 81 (Section 4.1) and R 82 (Section 4.4) for determination of the action effects.

7. See R 9 (Section 4.8) for analysis of the equilibrium of vertical forces.

8. See R 83 (Section 4.11) for the serviceability analysis.

9. See R 30 (Section 9.7) for the design of retaining walls on opposite sides of braced excavations.

9.3 Active earth pressure for large distances to structures (R 28)

1. If the preconditions for adopting a large distance between the retaining wall and other structures given in R 21, Paragraph 4 (Section 9.2) are fulfilled, the magnitude of the earth pressure shall be determined in two ways:

   a) The earth pressure $E_{ah,k}$ is obtained for a slip surface intersecting the ground surface in front of the structure at an angle $\vartheta_{a,k}$. 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 is decisive for further analysis.

2. The magnitude of the earth pressure $E_{ah,k}$ from soil weight density, unbounded 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), is obtained in a similar manner to that for a retaining wall not loaded by the earth pressure from a structure. The general rules given in Chapters 3 to 6 apply for the earth pressure distribution.

3. The magnitude of the earth pressure Ezh,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 EzBh,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. As an 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 weight density, unbounded 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 Σ H = 0 according to R 15, Paragraph 6 c) or Paragraph 7 c) (Section 5.5).
   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 the building loads may be incorporated into this pressure diagram, taking the zone of influence according to Paragraph 2 into consideration, so that any sudden alteration in the earth pressure ordinate lies in 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. 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 $E_{aBqh,k}$ from building live loads. According to R 104, 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 load together with the building dead weight.

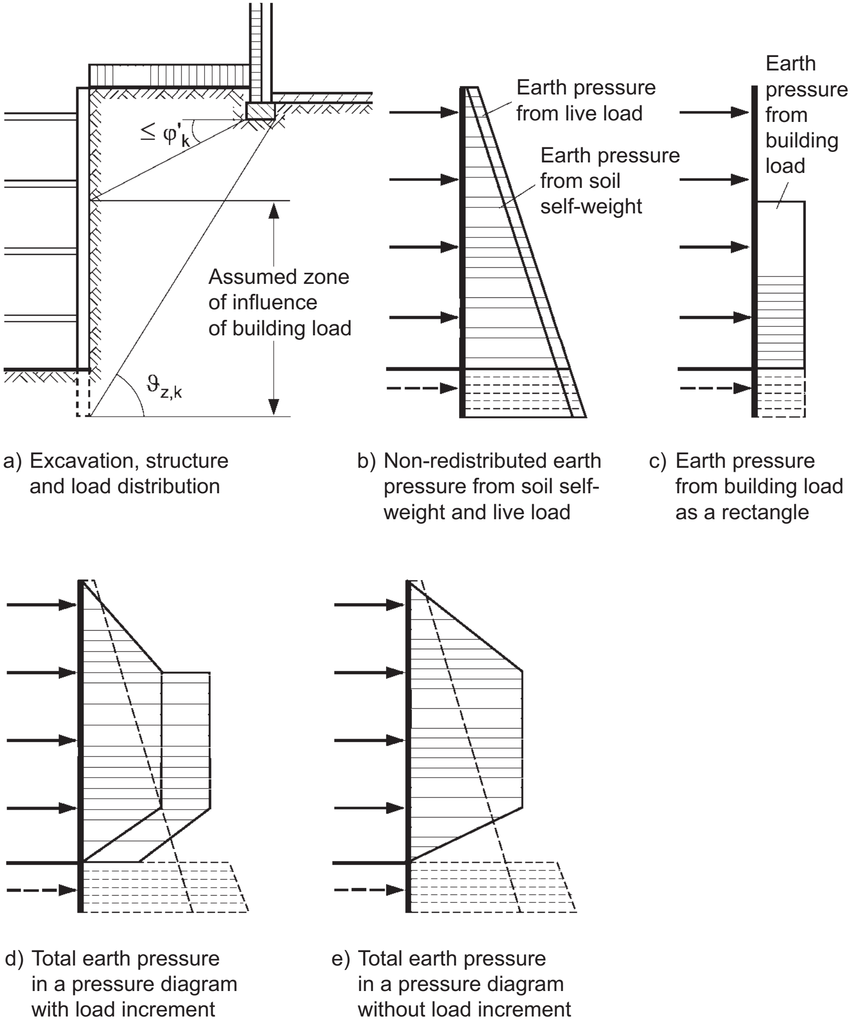


Figure R 28-1. Distribution of active earth pressure taking the influence of a building load with large distance between retaining wall and structure into consideration (example for a soldier pile wall with free-earth support)

9.4 Active earth pressure for small distances to structures (R 29)

1. If the preconditions for adopting a short distance between the retaining wall and other structures given in R 21, Paragraph 3 (Section 9.2) are fulfilled, it is convenient to determine the earth pressure $E_{ah,k}$ from soil weight density, unbounded 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 weight density 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 weight density of the soil below the foundation base, for the weight density of the soil above the foundation base within the structure and the cellar floor, and for a live load $p_k \leq 10$ kN/m^2 acting on the cellar floor.
2. The earth pressure from the weight density of the soil above the foundation base between the retaining wall and the structure, and the earth pressure from the effective live load in this region, are first determined down to the foundation base and then supplemented by the component resulting from an assumed 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 weight density 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 weight density 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 cellar floor and any live load $p_k \leq 10$ kN/m^2 in the cellar.
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 $\vartheta_{a,k}$. As an approximation, the zone of influence of the building load may be assumed as shown in Figure R 29-1 a) and the earth pressure distribution 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.

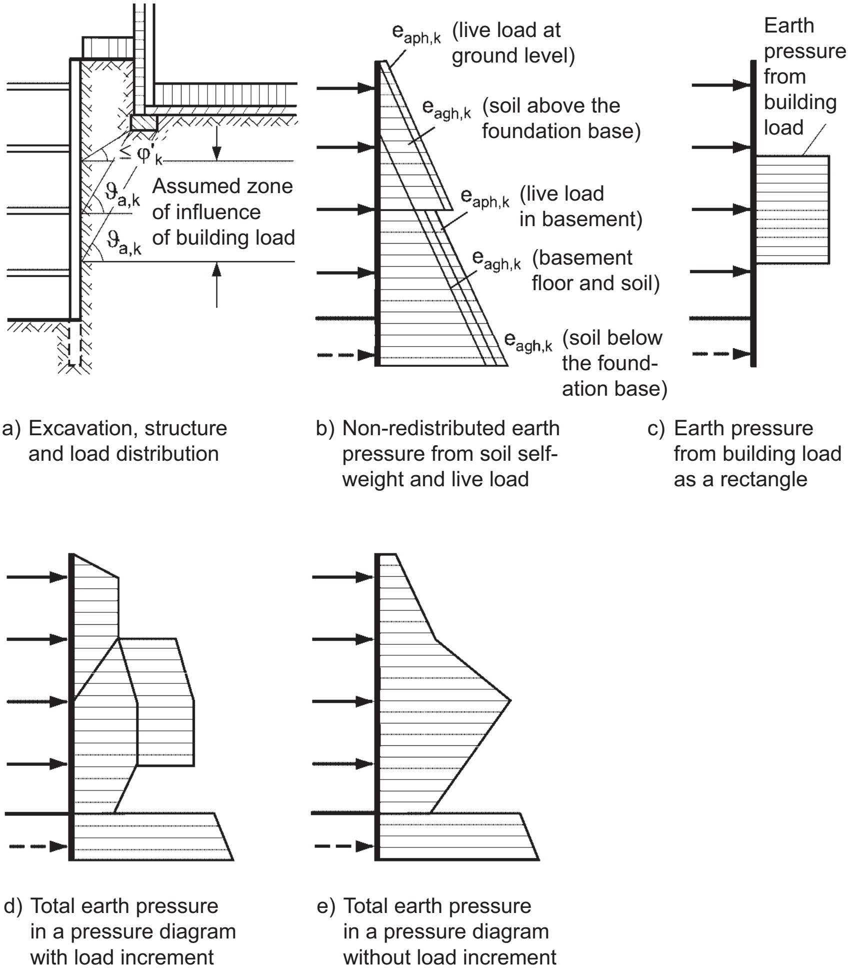


Figure R 29-1. Distribution of active earth pressure taking the influence of a building load with a small distance between the retaining wall and the structure into consideration (example for a sheet pile wall or in-situ concrete wall with free-earth support)

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 4.12), however, it is generally permissible to increase the building live load by the factor f_q and then to treat it as a permanent load together with the building dead weight.

6. The pressure diagrams determined according to Paragraphs 2 to 4 may be superimposed. The ensuing overall pressure diagram may be selected so that any sudden alteration in the earth pressure ordinate lies in the area of a support point (Figure R 29-l d), or so that no sudden alteration of the earth pressure ordinate occurs (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 according to Paragraph 3 into consideration.

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), taking existing structures into consideration using the measures stipulated in R 20, Paragraph 4 (Section 9.1), the excavation structure shall be designed for increased active earth pressure. Increased active earth pressure is defined as an earth pressure that is greater than active earth pressure but smaller than the at-rest earth pressure. The magnitude of this increased active earth pressure depends on the conditions in the excavation and on the structure. It should be noted here that the computed total at-rest earth pressure may be smaller than total active earth pressure for structures close to the excavation. Because the validity of the elastic half-space theory is in question in this case, large and small distances to the structures are differentiated when defining the increased active earth pressure.
2. For large distances to structures according to R 28 (Section 9.3) the mean value:

   $E_{h,k} = 0.50 \cdot (E_{0h,k} + E_{0Bh,k}) + 0.50 \cdot (E_{ah,k} + E_{zBh,k})$

   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. In simple cases the earth pressure:

   $E_{h,k} = 0.25 \cdot (E_{0h,k} + E_{0Bh,k}) + 0.75 \cdot (E_{ah,k} + E_{zBh,k})$ is sufficient

   in complex cases it may be necessary to adopt the magnitude of the earth pressure at:

   $E_{h,k} = 0.75 \cdot (E_{0h,k} + E_{0Bh,k}) + 0.25 \cdot (E_{ah,k} + E_{zBh,k})$

   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) $E_{h,k} = 0.25 \cdot E_{0h,k} + 0.75 \cdot E_{ah,k} + E_{aBh,k}$ in simple cases;
b) $E_{h,k} = 0.50 \cdot E_{0h,k} + 0.50 \cdot E_{ah,k} + E_{aBh,k}$ in normal cases;
c) $E_{h,k} = 0.75 \cdot E_{0h,k} + 0.25 \cdot E_{ah,k} + E_{aBh,k}$ in complex cases.

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 weight density, 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 weight density, 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 value between the active earth pressure and the at-rest earth pressure, it can be assumed that the earth pressure redistribution occurs in a similar manner to active earth pressure, but with a tendency to decrease, the greater the proportion 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 differentiation between structures at a large and those at a small distance 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). This is recommended in particular if neighbouring cellar walls, subsurface pipes or other structures are endangered by prestressing the struts in the upper region of the retaining wall.

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 the case of soldier pile walls the design passive earth pressure is multiplied by the calibration factor $\eta_{Ep} \leq 0.6$ or, in the case of sheet pile walls and in-situ concrete walls, by the calibration factor $\eta_{Ep} \leq 0.8$ in order to reduce the toe deflections in medium-dense or dense soils or at least stiff, cohesive soils. If the ground consists of soft, cohesive soil a design shall be selected that does not require an earth support.

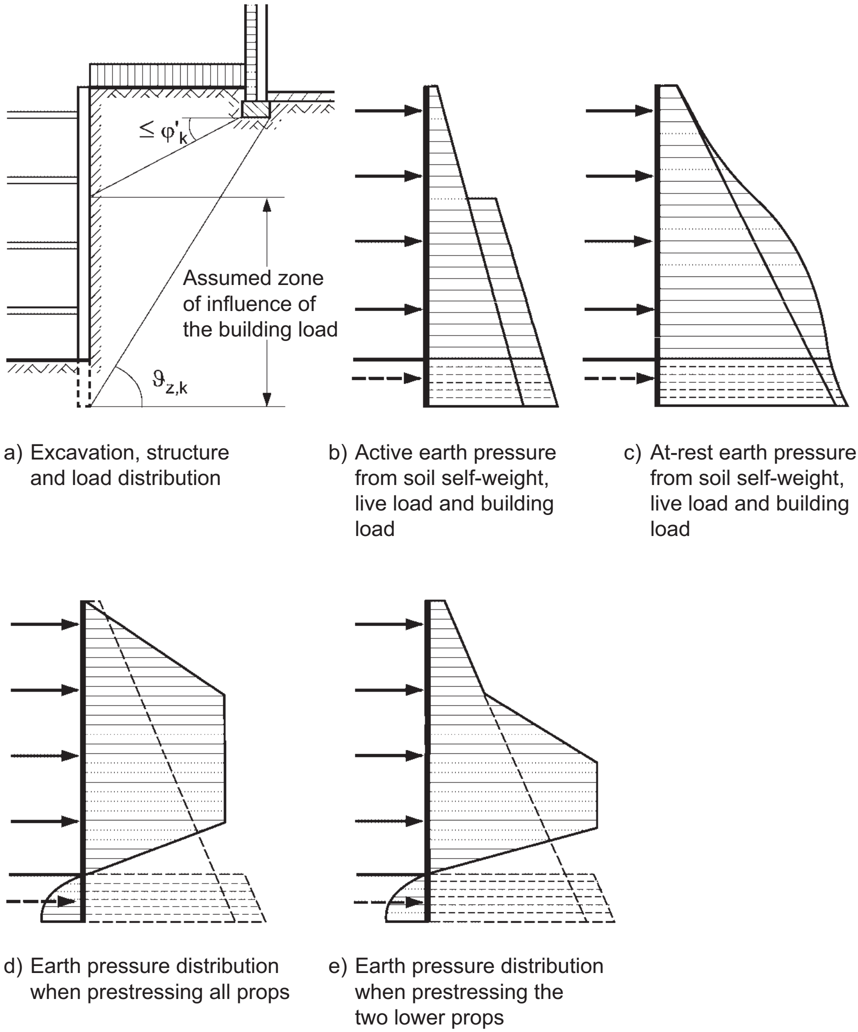


Figure R 22-1. Distribution of active earth pressure taking a building load with a large distance between the retaining wall and the structure into consideration (example for a soldier pile wall with free-earth support)

7. The information in Chapter 4 applies for analysis of the embedment depth and for determination of the design action effects. The decisive partial safety factor for permanent actions shall consist of the same ratios of the partial safety factors γ_{E0g} and γ_G as the decisive characteristic earth pressure $E_{h,k}$ according to Paragraph 2, for large spacing to buildings, or according to Paragraph 3, for small spacing to buildings. According to the prevalent situation, the partial safety factors for Load Case LC 2 or Load Case LC 2/3 given in Table 6.1 in Appendix A 6 are decisive. In the case of Paragraph 3, the partial safety factors for active earth pressure are decisive for 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 transmitted to the ground by the retaining wall according to R 9 (Section 4.8), and that the subsequent settlements have no detrimental impact on the structure. It may be necessary to forgo the adoption of an earth pressure angle when determining the active earth pressure. However, in this case it should be taken into consideration that additional vertical stresses and settlements can occur due to the circumvented distribution of the building load in the subsurface. Also see Figure R 22-2. If these settlements are deleterious to the structure suitable measures shall be implemented on the structure or in the subsurface.
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, according to R 15 (Section 5.5). The earth pressure acting below the excavation level shall be adopted with the same ratio as the active earth pressure and at-rest earth pressure acting above the excavation level. If the 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.

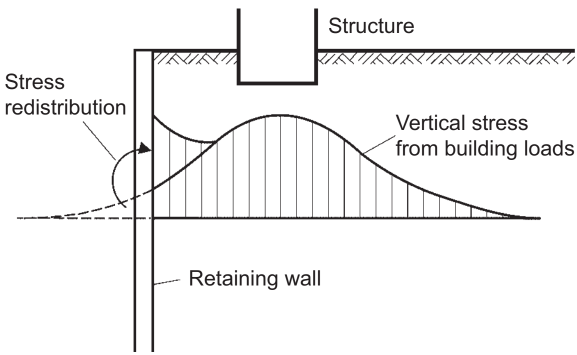


Figure R 22-2. Stress redistribution for restricted load distribution

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 normally 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, it is recommended to monitor the movements of the structure and the retaining wall by taking measurements where sensitive structures are involved, as well as monitoring the stresses on the struts or anchors, and providing for post-stressing measures where necessary.

11. See R 83 (Section 4.11) for serviceability analysis. The notes on the possible prevention of load distribution and associated settlements in Paragraph 8 should be observed. Use of the finite-element method according to R 103 (Section 4.7) is recommended for more precise investigations.

12. See R 30 (Section 9.7) for design of retaining walls on opposite sides of braced excavations.

9.6 Analysis of retaining walls with at-rest earth pressure (R 23)

1. In complex cases it is generally recommended to adopt the earth pressure at either $E_{h,k} = 0.75 \cdot (E_{0h,k} + E_{0Bh,k}) + 0.25 \cdot (E_{ah,k} + E_{zBh,k})$ or at $E_{h,k} = 0.75 \cdot E_{0h,k} + 0.25 \cdot E_{ah,k} + E_{aBh,k}$ according to R 22, Paragraphs 2 and 3 (Section 9.5). Only in those exceptional cases in which it can be demonstrated that the at-rest earth pressure of the undisturbed ground is maintained when installing the retaining wall and, in addition, ground unloading is avoided by the use of an inflexible support according to R 67, Paragraph 5 (Section 1.5), may it be expedient to apply the at-rest earth pressure to the retaining wall, e.g. for structures that are very high, poorly founded or in poor structural condition. However, this does not exclude the possibility of settlements affecting these structures.

2. The magnitude and distribution of the at-rest earth pressure are obtained according to R 18 (Section 3.7). The following apply for defining the pressure diagram:

   a) The at-rest earth pressure from soil weight density 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 constant from the lowest row of supports

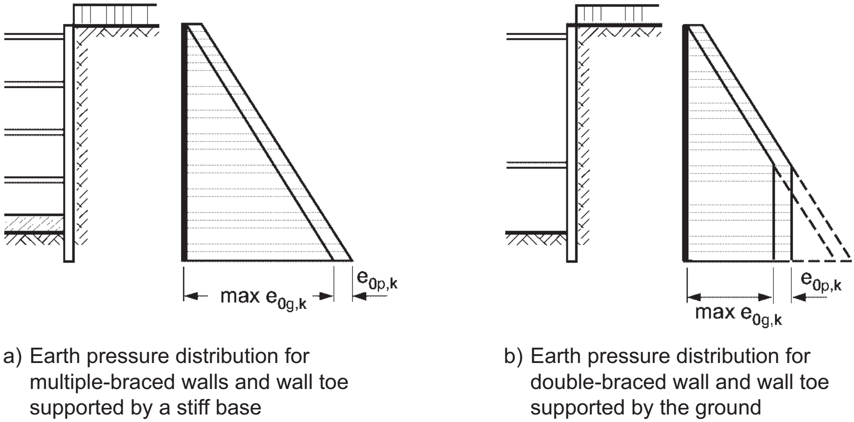


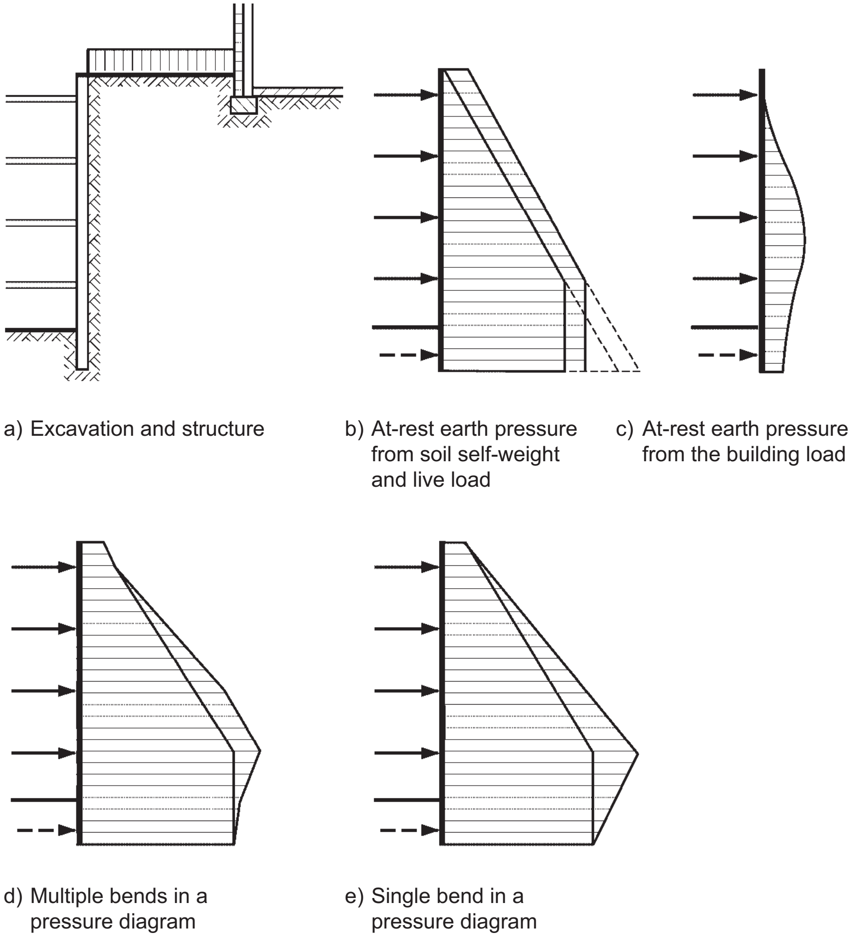
Figure R 23-1. Load model determination for in-situ concrete walls adopting the at-rest earth pressure

downwards for retaining walls with at least two rows of struts or anchors (Figure R 23-l b). For single-propped walls without timely bracing of the base, it cannot be assumed that the full at-rest earth pressure is maintained. 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) For the at-rest earth pressure from an unbounded distributed load $p_k \leq 10\ kN/m^2$ the pressure diagrams described in Paragraph a) 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 should begin approximately at the level of the base of the building and the resultant should 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 the individual at-rest earth pressure components may be simplified such that, for an unchanged total load magnitude, a pressure diagram ensues that displays no sudden changes (Figure R 23-2 d) and (Figure R 23-2 e), or for which a sudden change lies at a support point. This also applies to the changeable 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.



a) Excavation and structure

b) At-rest earth pressure from soil self-weight and live load

c) At-rest earth pressure from the building load

d) Multiple bends in a pressure diagram

e) Single bend in a pressure diagram

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 building load influence (example of an in-situ concrete wall with free earth support)

3. The passive earth pressure is generally 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-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, e.g. a previously stiffened base.

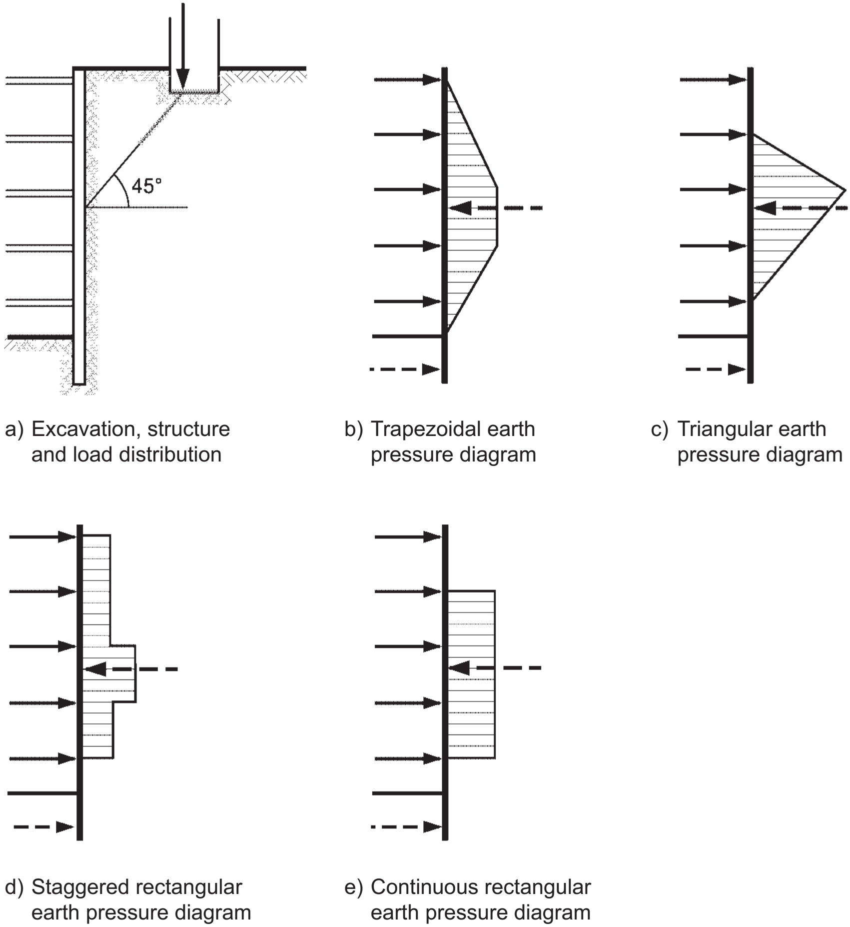


Figure R 23-3. At-rest earth pressure approximations for building loads and non-yielding retaining walls

4. The previously stiffened base solution discussed in Paragraph 3 is not only recommended for loosely compacted, cohesionless soil. If the wall is embedded sufficiently deep in the subsurface 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 determination of the embedment depth and the design action effects. 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, is decisive.

6. Because no construction measures are capable of guaranteeing that deformations or deflections of the retaining wall or the ground will not occur, the actual earth pressure distribution may deviate from the assumed at-rest earth pressure distribution as shown in Figure R 23-1. Therefore, it shall be ensured that the struts or anchors located in the upper part of the wall are, if necessary, capable of accepting the active earth pressure determined according to R 21 (Section 9.2). In the place of a precise analysis it is permissible to design the struts and anchors in the upper third of the wall for 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 weight density (for an inclined ground surface) and the at-rest earth pressure from the building load can be transmitted 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 transmitted by the wall to the subsurface without appreciable settlement according to R 9 (Section 4.8). 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. In this case it is necessary to either ensure the stability of the structure by further measures, e.g. by soil stabilisation, or to design the retaining wall for increased active earth pressure according to R 22 (Section 9.5).

8. In order to ensure that the retaining wall makes no undesired movements either towards or away from the structure, it is generally useful to continuously monitor its position by measurements during excavation work and, if necessary, to initiate counter-measures. Struts and anchors are prestressed to the full computed, characteristic load during installation and then post-stressed, if monitoring according to R 34, Paragraph 6 (Section 14.4) shows an appropriate result.

9. See R 83 (Section 4.11) for serviceability analysis.

10. See R 30 (Section 9.7) for design of retaining walls on opposite sides of braced excavations.

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, but is lined equally on both sides by soldier pile walls, sheet pile walls or in situ concrete walls, 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. A precondition for this approach, however, is that the earth pressure acting on the loaded side produces strut forces in all rows of struts that are greater than those from the earth pressure acting on the unloaded side. For example, if, for structures close to the retaining wall (Figure R 30-1), lower strut forces arise from the loaded side of the excavation, the same deliberations should be made as if there are structures on both sides. See also Paragraph 3 and Paragraph 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 further away from the structure can, as an approximation, be designed for 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 away from the structure. The characteristic support forces of the retaining wall subject to building loads shall be applied as loads to the retaining wall further away from the structure. The pressure diagram for this structure shall then be selected as appropriate for the loads, stiffness conditions and earth pressure theory.

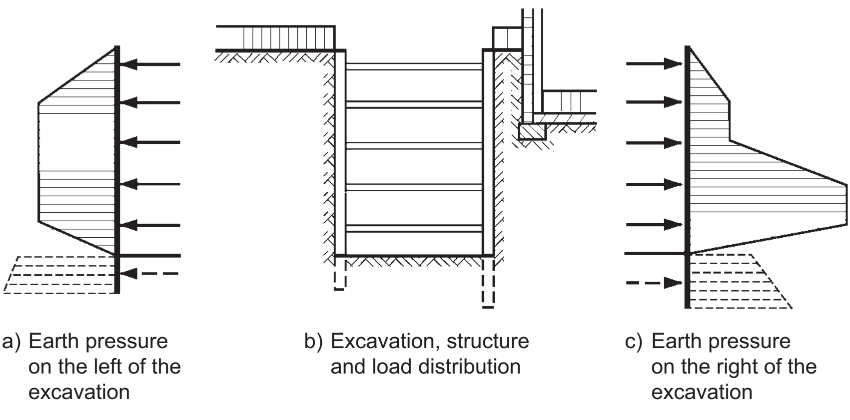
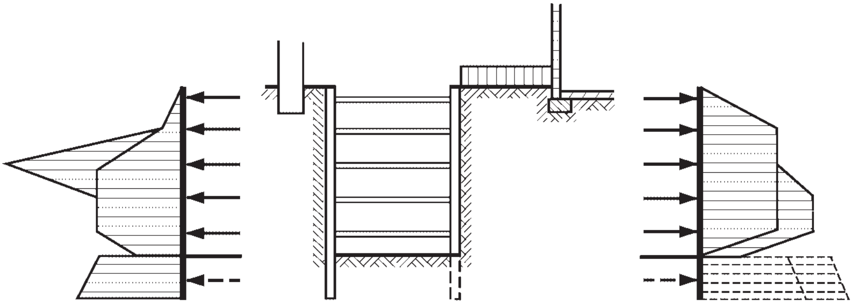


Figure R 30-1. Excavation with horizontal bracing and one-sided loading from a structure

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 investigated 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 are grossly dissimilar for the two retaining walls, the pressure diagrams shall be respectively developed so that roughly similar support forces result. If a soldier pile wall is installed on one side of the excavation and a sheet pile wall or in-situ concrete wall at the other, the earth pressure is adopted below the excavation level according to R 15 (Section 5.5) for soldier pile walls and R 16 (Section 6.1) for in-situ concrete walls. See Figures R 30-3 c) and R 30-4 c).

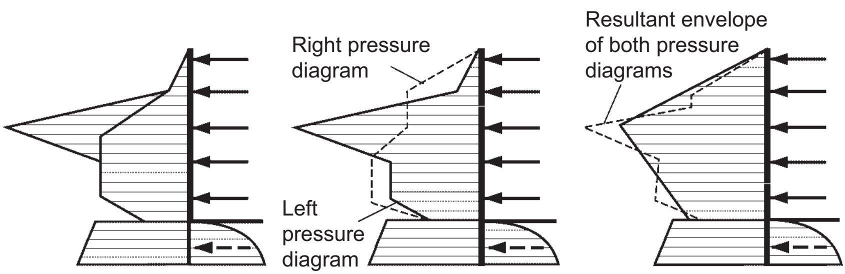


a) Earth pressure on the left of the excavation

b) Excavation, structure and load distribution

c) Earth pressure on the right of the excavation

Figure R 30-2. Excavation with horizontal bracing and bilateral loading from a structure



a) Original pressure diagram for the left side

b) Superimposed right pressure diagram

c) Resultant pressure diagram

Figure R 30-3. Superimposing pressure diagrams on the left of the excavation

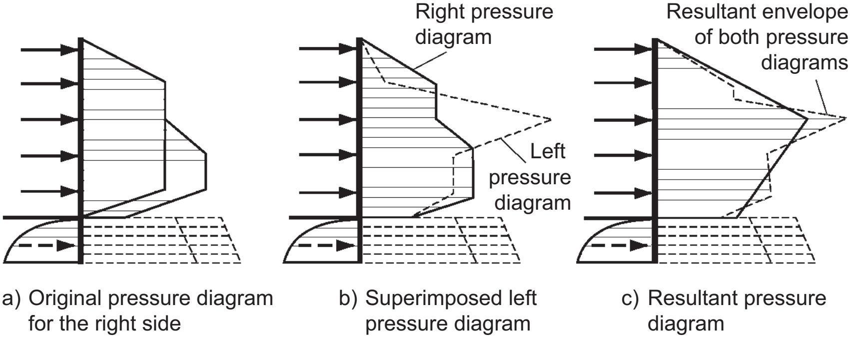


Figure R 30-4. Superimposing pressure diagrams on the right of the excavation

4. Because of the effects of mutual influence, a larger earth pressure shall be assumed for both sides according to Paragraph 3 than would be the case when determining the earth pressure for each side separately alone, if the earth pressure distribution is different for each side of the excavation, e.g. for one of the cases shown in Figures R 30-1 and R 30-2. If this needs to be avoided, e.g. because it would represent a hazard to the stability of the cellar wall for the case shown in Figure R 30-1, equal strut forces can be achieved with an appropriate configuration of the individual rows of struts, despite differing pressure diagrams on each side of the excavation, if the earth pressure magnitude is equal (Figure R 30-5). If this is not the case, and whenever the configuration of the rows of struts cannot be arranged according to these factors, the difference shall be adopted as a surcharge on the wall with the smallest computed support force. The additional earth pressure mobilised by doing this shall be selected according to the stiffness conditions of the wall. Otherwise, equilibrium of vertical forces according to R 9 (Section 4.8) shall be demonstrated at all times for inclined struts.

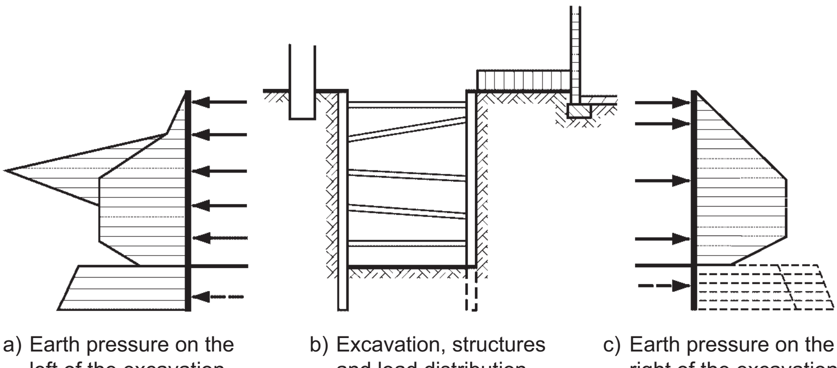


Figure R 30-5. Excavation with inclined bracing and bilateral loading from a structure

10 Excavations in water

10.1 General remarks for excavations in water (R 58)

1. With regard to the varying modes of water associated with excavations, the following cases may be differentiated in principle:

   – open water, e.g. lakes, rivers;
   – free (phreatic) groundwater;
   – confined groundwater.

2. With regard to the impact of the excavation structure and any water drawdown measures, the following cases may be differentiated in principle:

   a) If drawdown is performed as shown in Figure R 58-l a), both horizontal and downward directed seepage pressures occur in the soil mass pertinent to the excavation structure. In this context, R 59 (Section 10.2) shall be observed when determining the seepage pressure 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 seepage pressures also ensue. In this context, R 59 (Section 10.2) shall be observed when determining the seepage pressure, 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 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.

   Furthermore, in some cases, if the water is not pumped out of the excavation in situations b) and c), the water level inside and outside of the excavation may also be the same. In special cases it may even 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.

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 shear strength, the formation of a gap between the retaining wall and the ground is possible, in which hydrostatic pressure occurs.

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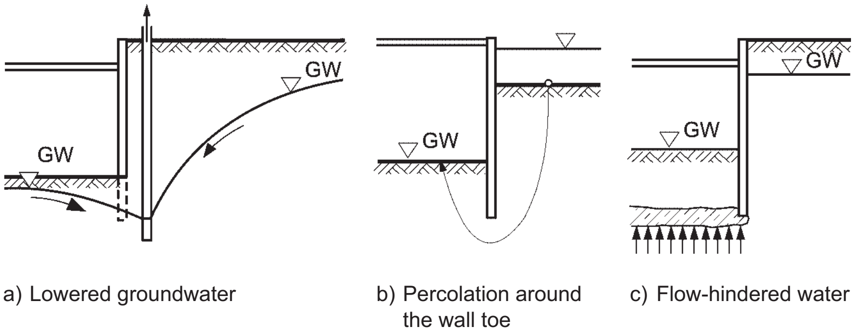


Figure R 58-1. Impacts of water on the excavation structure

4. In loosely compacted sand and silt in particular there is a danger of erosion 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 8 (Section 10.7) of this publication.
5. A decisive 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 low-permeability, 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 8 (Section 10.7) of this publication for possible structural counter-measures.
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 8 (Section 10.9).
7. According to R 65, Paragraph 4 (Section 10.8) the groundwater within the excavation should be lowered to approximately 0.50 m below the excavation level. As a simplification in the following figures, the computed groundwater table is shown at the excavation level.
8. When adopting the soil parameters for designing the excavation structure in water, it should be noted that:

– a local differential pressure can develop in saturated fine-sand and silt soils and that the ground may thereby assume flow characteristics, see DIN 1054:2005-01, 5.3.2 (5);
– porewater pressure may develop, see DIN 1054:2005-01, 5.3.2 (7);
– weathering or softening may occur, see DIN 1054:2005-01, 7.1 (3).

10.2 Seepage pressure (R 59) (11/05)

1. Seepage pressure develops if a potential difference is present as shown in Figure R 58-1 a) or Figure R 58-l b) (Section 10.1), which induces groundwater flow. The seepage pressure 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, the unit weight increases, if it is from bottom to top, the unit weight is reduced.
2. In principle, two methods are available for determining the seepage pressure:

   a) If the seepage pressure at any point in the subsurface is required a flow net is used. This is obtained as demonstrated in the EAU, Recommendation R 113 [2]:

      – by graphical methods based on the trial and error method or;
      – by numerical methods with the aid of potential theory.

      The excavation depth shall also be taken into consideration.

   b) If the seepage pressure at individual, specific points only is required, e.g. at the toe of a retaining wall, graphs and tables or simple numerical approaches may be utilised for uniformly permeable ground [26, 56, 57, 58]. The seepage pressure 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 seepage pressure. 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 seepage pressure in homogeneous soils is determined independent of the value of the coefficient of permeability. Not the amount of water flowing is decisive, but the potential energy of the water as a result of height differentials between the inner and outer water level.
4. The following apply with regard to the permeability of the ground:

   a) Because a pressure drop is always concentrated in the less permeable layers, alternating vertical permeability due to ground stratification shall always be taken into consideration when determining the seepage pressure. See also R 61, Paragraph 6 (Section 10.4). In particular, the possibility of horizontal water ingress through the more permeable layers should be examined.

b) The difference between the horizontal permeability k_h and the vertical permeability k_v resulting from natural anisotropy of the soil, at $k_h / k_v \approx 2$ to 3, is generally only taken into consideration if the horizontal component of the total length of the critical stream tube is longer than the vertical one.

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 net resulting 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 seepage pressure on the stability of the excavation structure shall be performed. If necessary, the seepage pressure shall be taken into consideration for stability analysis.

2. In permeable soils the water surface profile is generally so flat that the groundwater has no impact on earth pressure. For silt and fine-sand, 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 while the groundwater table falls to the equilibrium level. It is then assigned to Load Case LC 2/3.

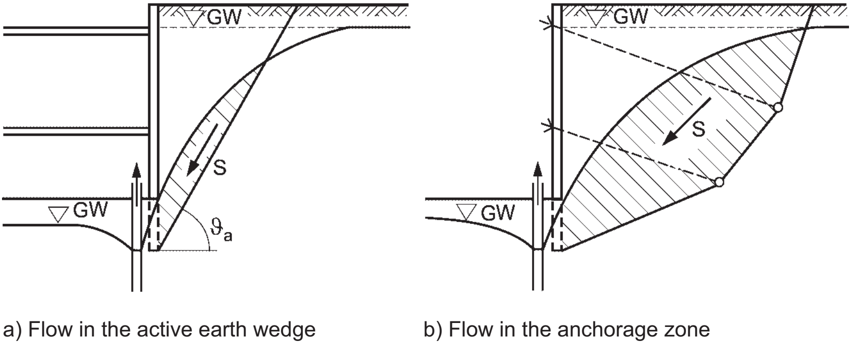


Figure R 60-1. Flow forces resulting from groundwater drawdown

3. Complete groundwater drawdown is often impossible in stratified soils. The following effects result from the remaining water compared to the target water level, as shown in Figure R 60-2 a):

   a) Additional water pressures develop in the region of the remaining water.
   b) The unit weight of the soil is lowered from γ to γ' in the region of the remaining water in the permeable layer.
   c) A gradient $i = (h + d) / d$ develops in the impermeable layer. The unit weight of the soil thus increases from $\gamma_r = \gamma' + \gamma_w$ to $\gamma_r = \gamma' + i \cdot \gamma_w$.

   The effects on water pressure are shown in Figure R 60-2 b), those on earth pressure corresponding to classical earth pressure theory 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 subject to pressure from below from confined groundwater despite water management measures, the unit weight reduction due to seepage pressure 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 decisive for stability, as shown in Figure R 60-1 b), the effect of seepage pressure shall be taken into consideration for both the stability analysis at the low failure plane according to R 44 (Section 7.3) and for the general stability analysis according to R 45 (Section 7.4).

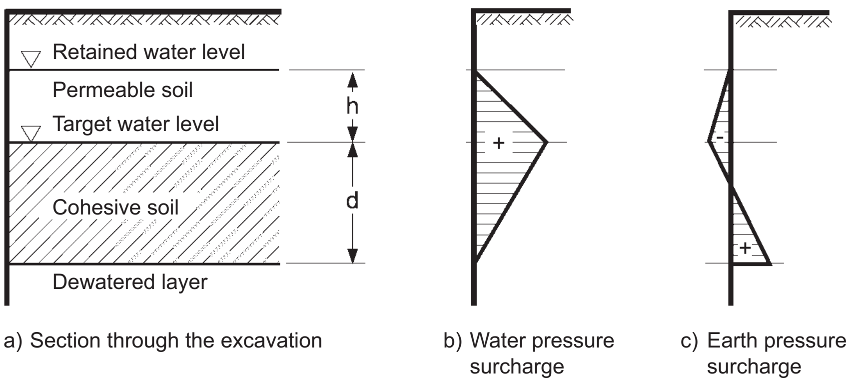


Figure R 60-2. Impact of retained water in stratified ground

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 the same effect as applying a load at ground level and may cause considerable settlement of soft, cohesive soils, which may also be detrimental to more distant buildings. If necessary, dewatering measures shall be dispensed with and different construction methods applied.

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 base failure occurs when cohesionless soils in front of the toe of a retaining wall become weightless as shown in Figure R 61-1 due to upward directed seepage pressure, or when the upward directed seepage pressure is equal to the sum of the soil weight density and additional restraining forces. Also see Paragraph 5.
2. The seepage pressures acting in the area of the investigated failure mass shall be determined according to R 59, Paragraph 2 (Section 10.2). An increase in the upward directed seepage pressure shall be anticipated if the preconditions for three-dimensional effects are given, e.g. in narrow, round or rectangular excavations [56, 57, 59]. If the hydraulic heave safety needs to be equal at all points in a rectangular excavation, the retaining walls shall be embedded deeper at the corners and possibly at the ends than at the centres of the longer sides. See also [117].
3. If necessary, the possibility of seepage path shortening, e.g. by fissure formation according to R 58, Paragraph 3 (Section 10.1) shall be taken into consideration. For staggered wall toes, the decisive depth for analysis of the hydraulic heave safety is always the lesser embedment depth.

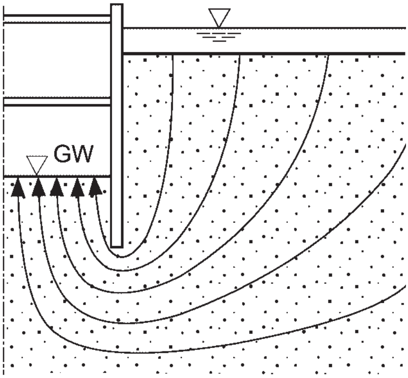


Figure R 61-1. Restriction of flow cross-section in the region of an upward directed flow in narrow excavations

4. No ground resistances are involved in hydraulic heave, only actions: the seepage force as an unfavourable permanent action; the soil weight density as favourable permanent action. Hydraulic heave failure is therefore classified as a failure resulting from the loss of equilibrium and thus assigned to the EQU limit state. In order to achieve sufficient safety against hydraulic heave failure it shall be demonstrated for homogeneous ground that the condition:

   $S'_k \cdot \gamma_H \leq G'_k \cdot \gamma_{G,stb}$

   is fulfilled.

   Where:

   S'_k the characteristic seepage force on the percolated soil mass;
   γ_H the partial safety factor for the seepage force in favourable or unfavourable ground in the EQU limit state taken from Table 6.1, Appendix A 6;
   G'_k the characteristic weight density of the buoyant, percolated soil mass;
   $\gamma_{G,stb}$ the partial safety factor for favourable permanent actions in the EQU 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. The simpler and more conservative stability analysis is performed using an infinitely narrow strip [61]. Friction forces between the failure body and the retaining wall may only be taken into consideration following special investigations. Expertise and experience in the geotechnical field are required.

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 the tensile strength may be locally lost 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

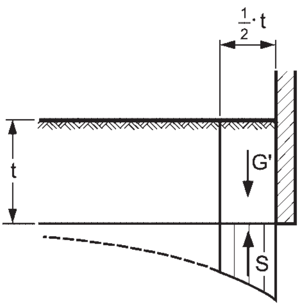
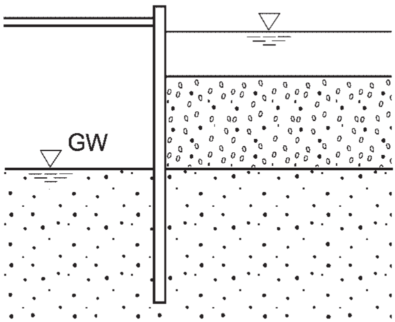


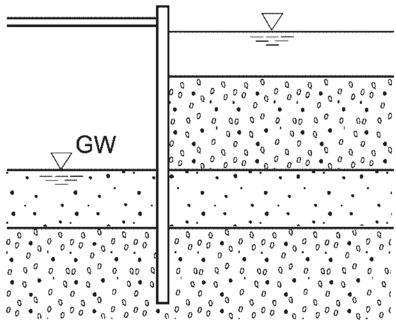
Figure R 61-2. Analysis of hydraulic heave safety after *Terzaghi* and *Peck* cannot be demonstrated, relief wells or spill wells according to Paragraph 10 a) shall be installed in order to decrease the seepage pressure on the inside of the excavation to the hydrostatic pressure.

6. If the ground is subject to variable permeability, the pressure drop is concentrated in the less permeable layers. In principle, two cases shall be differentiated here:

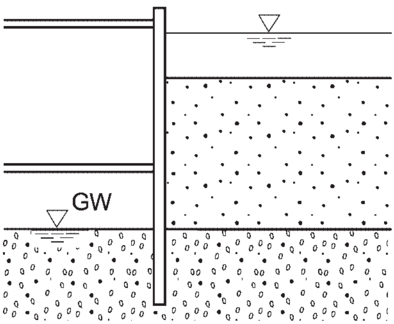
   a) With regard to the safety against hydraulic heave failure, the presence of a less permeable layer below the excavation base as shown in Figure R 61-3 a) acts unfavourably. In this case, only the seepage path through the less permeable layer may be adopted in the analysis.
   b) It is particularly 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.



a) More permeable layer above



b) Slightly permeable intermediate layer



c) More permeable layer below

Figure R 61-3. Influence of ground stratification

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 even slight disturbances in the subsurface structure can adversely effect the hydraulic heave safety at individual locations. 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 DIN 1054.

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 is decisive for hydraulic heave safety analysis. Generally, for low-permeability soils, in particular for silt and fine sand, the non-lowered groundwater table is taken as the basis for analysis.

8. The partial safety factors γ_H and $\gamma_{G,stb}$ required for analysis of the hydraulic heave safety in the EQU limit state can be taken from Table 6.1 of Appendix A 6. The following apply with regard to the partial safety factor γ_H for the seepage force in 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) Loosely compacted 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 or, if necessary, is installed in strips incorporating a geotextile.

   In unfavourable ground the hazard of piping failure shall be investigated. Countermeasures shall be provided for where necessary. See the EAU, Recommendation R 116 [2].

9. Analysis of the hydraulic heave safety can be dispensed with for excavations and trenches up to 5 m deep in homogeneous, groundwater-bearing soil, if the following conditions are adhered to with the designations given in Figure R 61-4:

   a) where $B \geq 2 \cdot h$: $t \geq 0.4 \cdot h$
   b) where $B \geq h$: $t \geq 0.5 \cdot h$
   c) where $B \geq 0.5 \cdot h$: $t \geq 0.7 \cdot h$

   Intermediate values may be linearly interpolated.

10. If investigations do not demonstrate sufficient hydraulic heave safety, the following measures may be taken in addition to enlarging the embedment depth:

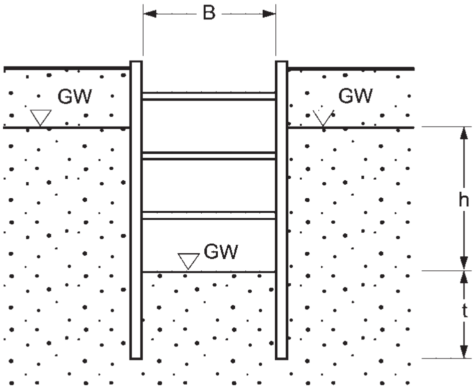


Figure R 61-4. Simplified analysis of hydraulic heave safety

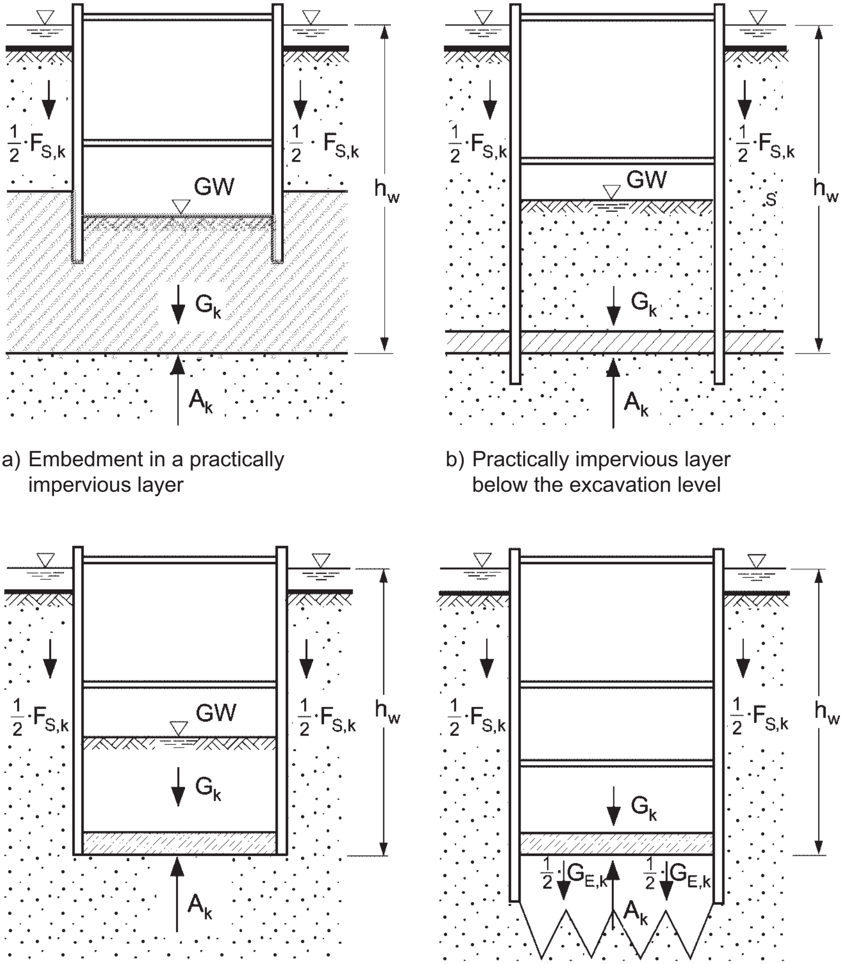
a) installation of spill wells (relief wells) within the excavation, see Section 10.8, Paragraph 6;
b) installation of gravity or vacuum wells within the excavation;
c) partial or complete dewatering or groundwater relief;
d) installation of a surcharge filter.

In addition, the installation of a base seal or an airtight cover on the excavation with subsequent use of compressed air are possible options.

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 are required within the excavation according to R 65, Paragraphs 5 and 6 (Section 10.8), if heterogeneities in the practically impermeable layer cannot be ruled out (see Paragraph 11).
   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 created at the level of the toe of the retaining walls, 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 or an anchored, jet grouted base (Figure R 62-1 d).



a) Embedment in a practically impervious layer

b) Practically impervious layer below the excavation level

c) Sealing layer

d) Raised, anchored concrete base

Figure R 62-1. Forces adopted for analysis of buoyancy safety

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 a tensile resistance does not act from a base anchored by tension piles or ground anchors, it shall be demonstrated that in the EQU limit state the condition:

$$A_k \cdot \gamma_{G,dst} \leq G_{k,stb} \cdot \gamma_{G,stb} + F_{S,k} \cdot \gamma_{G,stb} + F_{A,k} \cdot \gamma_{G,stb}$$

is fulfilled.

Where:

A_k — the vertical component of the characteristic hydrostatic buoyant force 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 EQU limit state taken from Table 6.1, Appendix A 6;

$G_{k,stb}$ — the lower characteristic value of the downward directed permanent actions from the weight density of the practically impermeable soil as shown in Figure R 62-1 a), or the practically impermeable soil layer including the overlying permeable soil as shown in Figure R 62-1 b), or the sealing layer including the overlying soil as shown in Figure R 62-1 c), and from the dead-weight of the retaining walls;

$\gamma_{G,stb}$ — the partial safety factor for favourable permanent actions in the EQU limit state taken from Table 6.1, Appendix A 6;

$F_{S,k}$ — the characteristic value of the vertical component of the earth pressure acting on the retaining wall as a permanent, downward directed action as shown in Figure R 62-1;

$F_{A,k}$ — the characteristic value of the vertical component of the anchor force from anchors supporting the retaining wall, as a permanent, downward directed action.

The forces $F_{S,k}$ and $F_{A,k}$ 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 5 to 8 for restrictions when adopting the downward directed actions. In addition, it may be necessary to demonstrate safety against hydraulic heave according to Paragraph 10.

3. If a tensile resistance from a base anchored by tension piles or ground anchors acts when analysing buoyancy safety, two limit cases shall always be investigated: the bearing capacity of the individual tension elements according to Paragraph a) on the one hand, and the bearing capacity of the tension elements 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 STR limit state. The tensile stress design value $E_{1Z,d}$ required for this analysis is determined from:

   $$E_{1Z,d} = A_{1GZ,k} \cdot \gamma_G - E_{1GD,k} \cdot \gamma_{G,inf}$$

Where:

$A_{1GZ,k}$ the characteristic value of the tensile stress resulting from the hydrostatic buoyant force acting on the underside of a concrete or jet grouted base as shown in Figure R 62-1 d);

γ_G the partial safety factor for permanent loads in the STR limit state taken from Table 6.1, Appendix A 6;

$E_{1GD,k}$ the characteristic value of a simultaneously acting compressive load resulting from permanent actions, e.g. the lower characteristic value of the dead-weight of a concrete or jet grouted base as shown in Figure R 62-1 d), the dead-weight of the retaining wall, the vertical component of the earth pressure acting on the retaining wall and the vertical component of the anchor load from anchors supporting the retaining wall;

$\gamma_{G,inf}$ the partial safety factor $\gamma_{G,inf} = 1.00$ for favourable compressive loads in the STR limit state.

Sufficient buoyancy safety is given if, for piles, the condition:

$E_{1Z,d} \leq R_{1,d}$

and for ground anchors the condition:

$E_{1Z,d} \leq R_d$

is fulfilled.

Where:

$R_{1,d}$ the design value of the tension pile resistance according to R 86 (Section 13.11);

R_d the design value $R_{a,d}$ of the pull-out resistance of the grouted section or the design value $R_{i,d}$ of the resistance of the steel tendon according to R 87 (Section 13.11).

See Paragraphs 5 to 8 for restrictions when adopting the downward directed action.

b) Assuming that the tension elements form a uniform soil monolith together with the ground within their zone of influence due to the group effect, sufficient buoyancy safety shall be demonstrated for the EQU limit state. This is demonstrated if the condition:

$A_k \cdot \gamma_{G,dst} \leq G_{k,stb} \cdot \gamma_{G,stb} + G_{E,k} \cdot \gamma_{G,stb} + F_{S,k} \cdot \gamma_{G,stb} + F_{A,k} \cdot \gamma_{G,stb}$

is fulfilled. Where, beside the variables declared in Paragraph 2:

$G_{E,k}$ the characteristic value of the downward directed permanent action from the weight density of the soil encompassed by the tension piles or the ground anchors.

See Paragraphs 5 to 8 for restrictions when adopting the downward directed action.

4. The full hydrostatic pressure $\gamma_w \cdot h_w$ on the base shall be adopted for determination of the characteristic buoyant force A_k, which is obtained from the design water level according to R 24, Paragraph 1, Paragraph 4 and Paragraph 5 (Section 2.1). The decisive base surface is the underside of the practically impermeable soil layer, the underside of the sealing layer or the underside of the anchored concrete base. For the purpose of calculations the underside shall be adopted high enough that all possible irregularities are taken into consideration conservatively. If force transmission from the retaining wall to the sealing layer is assumed according to Paragraph 8, the base water pressure acting on the underside of the retaining walls shall also be taken into consideration when determining the characteristic value of the buoyant force A_k. If applicable, the different magnitude of the base water pressure resulting from different height levels shall be observed.

5. The following apply for determination of the characteristic value of the weight density G_k as shown in Figure R 62-1:

   a) If the characteristic values of the unit weight are not verified by investigation or by soil sampling, no lower characteristic values may be adopted for natural soil and sealing layers than given in Table 3.1 in Appendix A 3 or in Table 4.1 in Appendix A 4.

   b) If the characteristic value of the unit weight is not determined by sampling, the unit weight of the concrete may be assumed at a maximum of 23 kN/m^3 and the characteristic value of the unit weight of reinforced concrete at a maximum of 24 kN/m^3.

   c) The characteristic value of the weight density of the soil within the excavation shall be determined using the wet unit weight of the soil above the water table and using the saturated unit weight below it. For this purpose the water level within the excavation shall be estimated conservatively at its lowest level, taking into consideration any drawdown due to water management measures.

   d) The characteristic value of the dead-weight of the retaining wall is determined as follows:

      – for a sheet pile wall, from the weight of steel in the wall without adhering soil;
      – for a secant pile wall with a footprint area of 0.9 × pile diameter;
      – for a diaphragm wall from the nominal thickness of the wall.

6. The force $F_{S,k}$ is obtained from:

   $F_{S,k} = \eta \cdot E_{ah,k} \cdot \tan \delta_{a,k}$

   where the calibration factor $\eta = 0.80$.

   The earth pressure on the retaining wall may only be adopted at its lower characteristic value. According to DIN 1054 this is generally half of the value used for designing the excavation structure in cohesionless soils and $E_a = 0$ for cohesive soils. More favourable approaches are only permissible after more precise investigations. Expertise and experience in the geotechnical field are required.

7. Only the lock-off force P_d may be adopted when determining the vertical component $F_{A,k}$ of the tensile force of prestressed anchors supporting the retaining wall.

8. If the dead-weight of the retaining wall according to Paragraph 5 d), the vertical component of the earth pressure according to Paragraph 6 or the vertical component of the anchor forces according to Paragraph 7 need to be made to bear upon a sealing layer, the following shall be observed:

   a) A corresponding force transmission to the sealing layer, or to the concrete or jet grouted base shall be ensured.
   b) Uniform distribution of the force transferred by the retaining wall across the entire width of the excavation due to vault development shall be ensured. However, this is only possible to a limited degree depending on the soil type and the ratio of the excavation width to the thickness of the soil mass. Assessment of this case requires geotechnical expertise and experience in the field.
   c) Uniform distribution of the force transferred on the concrete base across the entire width of the excavation via the retaining wall, e.g. by vault development or by reinforcement, shall be demonstrated. Assessment of this case requires expertise and experience in the field of concrete engineering.
   d) The dead-weight of the retaining wall determined according to Paragraph 5 d) shall be transferred into the sealing layer, minus the base water pressure acting on the underside of the wall.

9. The characteristic weight density $G_{E,k}$ of the soil held by the tension piles or the ground anchors may be determined using the geometric relationships as shown in Figure R 62-2 using:

$$G_{E,k} = n \cdot [l_a \cdot l_b \cdot (L - {}^1/_3 \cdot \sqrt{l_a^2 + l_b^2} \cdot \cot \varphi)] \cdot \eta \cdot \gamma'$$

Where:

$G_{E,k}$ the characteristic weight of the attached soil;
L the length of the tension element below the lower surface of the base;
l_a the greater grid dimension of the tension elements;
l_b the lesser grid dimension of the tension elements;
γ' the lower characteristic value of the unit weight of the buoyant soil;
n the number of tension elements;
η the calibration factor $\eta = 0.80$.

If the characteristic value of the unit weight of the soil to be used in calculation is not verified by investigations, no greater characteristic values may be adopted than given in Table 3.1 in Appendix A 3 or in Table 4.1 in Appendix A 4.

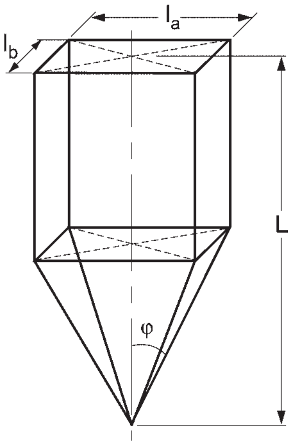


Figure R 62-2. Geometry of the ground attached to a single tension element

10. In addition to the buoyancy safety factor analysis, analysis of the 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;
    b) the retaining walls embed in a layer with a permeability less than two orders of magnitude smaller than the surrounding 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.

11. If the porewater pressure conditions are unclear, in particular in finely stratified ground, for example, or if the practically impermeable layer increases in permeability from the top downwards as shown in Figure R 62-1 a), relief wells, gravity wells or vacuum wells according to R 61, Paragraph 10 a) and 10 b) (Section 10.4) are expedient. Also see R 65 (Section 10.8).

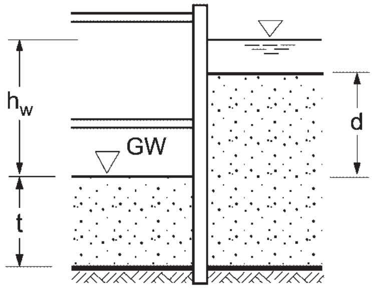
12. If the practically impermeable layer is fine-grained and the overlying layer is coarse-grained, the filter stability shall be analysed [79].

13. 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). The anticipated heave may be reduced by lengthening the tension piles, in particular if they are then better embedded in load-bearing ground.

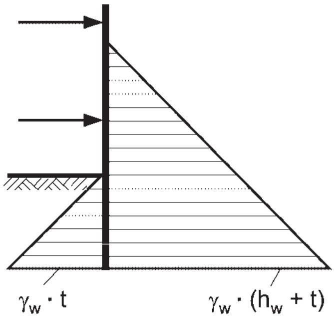
10.6 Stability analysis of retaining walls in water (R 63)

1. If the groundwater is not lowered, but 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. This approach may generally also be selected as an approximation if seepage actually occurs around the wall toe.

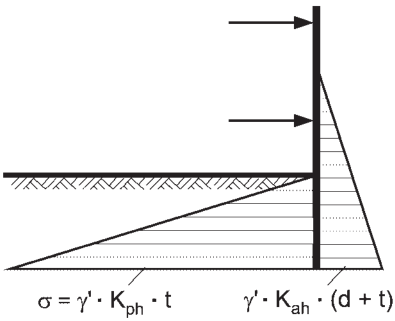
   The differential water pressure between the water pressure on the outside and that on the inside of the retaining wall is treated as the only characteristic action according to DIN 1054.



a) Designations



b) Water pressure



c) Earth pressure and ground reaction

Figure R 63-1. Earth pressure, water pressure and ground reaction for a non-percolated retaining wall in water (simplified representation)

2. If water does percolate around the wall toe, the impact of the flow shall be considered as follows for more precise investigations:

   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 seepage pressure (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:

      $\Delta \gamma'_p = -i_p \cdot \gamma_w$

   This impact shall always be taken into consideration.

   See R 59 (Section 10.2) for determination of the seepage pressure. Figure R 63-2 shows a simplified representation of linear dissipation of the differential pressure. This represents an approximation. When determining the earth pressure it is conservative; when determining the passive earth pressure and analysing the safety against hydraulic heave it is non-conservative.

3. Earth pressure redistribution as stipulated in R 5 (Section 3.3) should also be anticipated if the soil is completely or partially buoyant. However, this does not include the increase in earth pressure enforced by seepage. It may be included in the redistribution diagram as a sufficient approximation. If the water pressure on the retaining wall is greater than the earth pressure on the wall, the anticipated earth pressure redistribution may be disregarded for determination of the action effects and, where necessary, replaced by appropriate surcharges to the determined support forces.

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, beside 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 of the Deutsche Bahn AG and [62];
   d) sheet ice pressure, see [63] and Ril 804.

   Further information on adopting ice loads is given in the EAU, Recommendation R 177 [2].

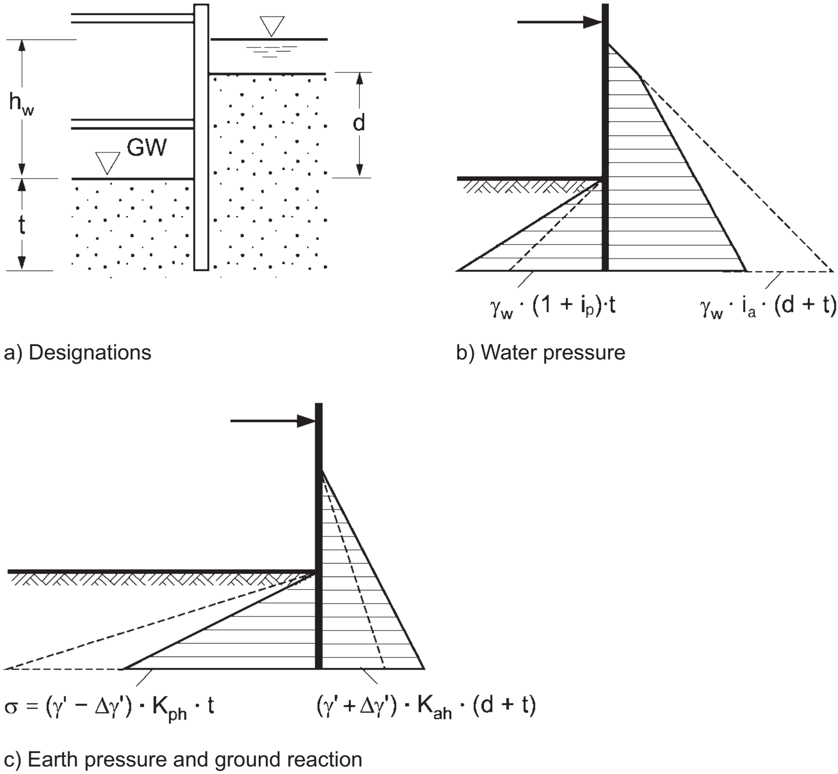


Figure R 63-2. Earth pressure, water pressure and ground reaction for a percolated retaining wall in water (simplified representation)

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 angle of passive earth pressure 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. See also [96]. A deep sealing layer manufactured using jet grouting methods forms a rigid 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 Load Case LC 2, the water level that will flood the excavation if adopted or at which the excavation shall be flooded, Load Case LC 2/3.
   b) If only the stability analysis for the STR limit state 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 fulfilled.
   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 4.12). However, when determining the vertical forces it should 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 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 of actions and of the structural system occur 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 the changes in action effects produced by these major changes. This is in contrast to R 11, Paragraph 2 b) (Section 4.2),

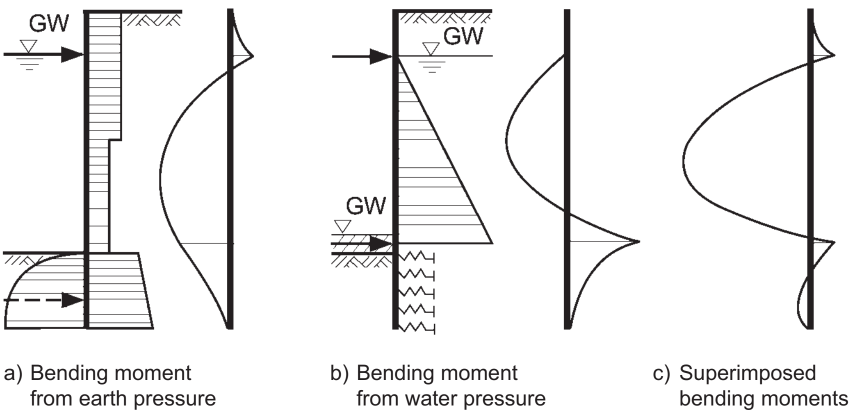


Figure R 63-3. Determination of bending moments for simultaneous change in load and structural system (example) where each construction stage may be analysed separately. This occurs if the excavation is emptied after installation of an underwater concrete base [97]. Also see Figure R 63-3.

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 4.8), 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 7 (Section 6.4) is not sufficient for walls restrained in the ground. 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 are generally to be prestressed at a minimum of 80% of the service load and at a maximum of 100% of the service load. The movement of the top of the wall caused by the anchor prestressing initially generates an increased active earth pressure at the rear face of the wall, 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 of the Recommendations of the Committee on Waterfront Structures for stability analysis of cellular and box cofferdams; see Recommendation R 69 for elastic dolphin piles.
11. As shown by monitoring and studies, see [96], considerable deflections at toe level can occur for excavations in water. This is particularly the case for anchored retaining walls adjacent to structures occurring simultaneously with a high groundwater table. If necessary, bracing should be installed in a timely manner at the excavation level or at the wall toe, e.g. using jet grouting.

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 such difficult zones, or open joints in diaphragm walls or pile walls, are not noticed until excavation has commenced, stabilising measures shall be immediately initiated, e.g. installation of a second wall (Figure R 64-1) or the manufacture of a grout curtain.

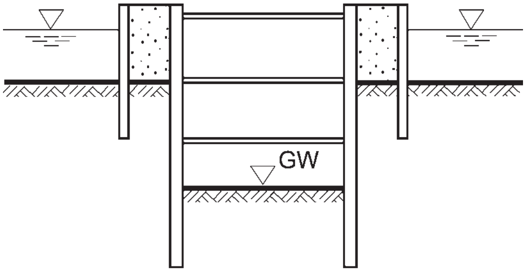


Figure R 64-1. Securing an excavation with cofferdams

2. The retaining walls shall reach the design depth at all points. 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. It can generally be assumed that a watertight connection is given if a retaining wall is embedded at least 0.50 m in stiff to nearly hard, cohesive soil or in rock, if this is not heavily jointed. If necessary, the watertight seal shall be subsequently manufactured, e.g. by grouting the region around the wall toe. Grouting should preferably be carried out before lowering the groundwater in the excavation, but at the latest before flow disturbances occur.

4. A practically impermeable layer manufactured by grouting according to R 62, Paragraph 1 c) (Section 10.5) is generally at least 1.0 m thick. The tubes used for grouting shall be so tightly spaced that grout overlapping is guaranteed. The erosion safety of the grouting medium shall be demonstrated. If initial piping phenomena (springs) are observed during excavation, countermeasures shall be implemented immediately, e.g.

   – soil placement;
   – partial flooding of the excavation;
   – water pressure relief measures.

   The weak area can then be regrouted. The measures given here may be dispensed with if rapid-curing materials are used for regrouting. See [97] for details of the toe deflections of the retaining wall with a deep, grouted base.

5. A practically impermeable layer manufactured using jet grouting methods according to R 62, Paragraph 1 c) (Section 10.5) is generally at least 1.0 m thick. Achievement of the planned diameter of the jet grouting columns shall be demonstrated by means of suitability tests. The borehole grid shall be configured such that the individual jet grouting columns safely overlap, taking

the anticipated diameter into consideration. The verticality of the boreholes shall be monitored. Deviations shall be taken into consideration by adapting the borehole grid. If springs are noticed during excavation, countermeasures according to Paragraph 4 shall be initiated immediately. Also see [144]. The defect can then be sealed by additional jet grouted columns or other measures.

6. In excavations that are to be 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, EN 206-1 and DIN 1045-3, as well as [138]. See [138, 141, 145] for details of execution of the anchoring elements and the anticipated deformations. If the vertical component of the earth pressure, the vertical component of the anchors and the dead weight of the retaining wall need to be taken into consideration for analysis of the buoyancy safety, complete force transmission 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. Any projected tension piles shall be sufficiently embedded in the concrete base.

7. See [146] for details of the various base designs and for the causes and remediation options with regard to leaks.

8. As a safeguard against 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), the configuration of a cofferdam as shown in Figure R 64-1 has proven useful in open water. As a minimum measure, securing the bed with sandbags along the length of the retaining wall should be planned. These measures are also suitable as a safeguard against erosion failure according to R 58, Paragraph 5 (Section 10.1).

9. In cases where a recognised hazardous condition cannot be otherwise eliminated, measures for purposely flooding the excavation shall be taken, in particular for excavations in open water. Occasionally it may be expedient for economical reasons to flood an excavation instead of designing for exceptionally high, rarely occurring water levels. When flooding, it shall be ensured that the inflowing water cannot cause damage. In addition, elongated excavations with a large surface area shall be divided into sections by bulwarks in order to restrict sudden water ingress to limited sections of the excavation. Sufficient embedment depth of the intermediate walls and the immediately adjacent side walls shall be demonstrated.

10. 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:

a) Configuration of dolphin piles to take up berthing shocks of ships 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) Avoiding ingress.
   b) Sump pumping.
   c) Dewatering (gravity or vacuum drainage).
   d) Groundwater relief.

   The filter criteria shall be observed. Also see DIN 4095.

2. The ingress of groundwater into the excavation can be prevented by:

   a) Watertight retaining walls embedded in an impermeable layer, see R 62 (Section 10.5).
   b) Installing a sealing curtain outside of the excavation, which is embedded in an impermeable layer, see the EAU, Recommendation R 156 [2].

   These measures can be expedient if groundwater drawdown is not permitted or could lead to settlement damage in the surroundings, or there is no possibility of disposing of the pumped water economically.

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 and limited water ingress. In soils with a tendency to liquify special measures are necessary, e.g. soil replacement methods, whereby only small areas are laid free for short periods and are immediately covered by filter material.

4. For dewatering 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 gravity is not sufficient to allow the water to flow into the filter well, e.g. in fine-sand or coarse silt.

   See also [1] and [64]. The lowered groundwater table within the excavation should generally be approximately 0.50 m below the excavation level.

Deeper drawdown within the excavation generally has no negative impact on the stability of the retaining walls. However, it may impact negatively on serviceability. It should therefore be avoided [96].

5. Groundwater relief may be necessary:

   a) If a cohesive 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.

   In these cases it may 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.

6. The following should be observed when employing overflow wells according to Paragraph 5:

   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, the filter criteria shall be observed.
   c) Dissipation of the positive water pressure shall be monitored using observation wells.
   d) Overflow wells shall generally be sealed with appropriate material after abandonment.

7. 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 the 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 should 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 low-permeability soil, in order to facilitate timely recognition of irregularities. If piping becomes apparent at any stage the soil shall be immediately refilled to prevent further spread of the flow.

4. If local conditions cannot exclude the possibility of fissure formation behind the retaining wall according to R 58, Paragraph 3 (Section 10.1), it is recommended to tap the retaining wall in the endangered area and to install transparent hose to display the local water pressure. Areas of the wall displaying large deformations are particularly threatened.

11 Excavations in unstable rock

11.1 General recommendations for excavations in unstable rock (R 38)

1. Rock is a consolidated mass of mineral material formed in-situ and consisting of similar or dissimilar individual components. Stability is demonstrated by means of rock mechanics investigations based on rigid body mechanisms. If these indicate that a rock cutting is instable, supports are needed either:

   a) by means of stabilising individual rock masses in danger of slipping by targeted or distributed installation of rock nails or rock anchors, or;
   b) by means of a distributed, supported lining, in particular if heavily fractured or decomposed rock indicates that further fracture mechanisms may act in addition to the kinematics predetermined by the principal discontinuity structures.

   The force needed to support a rock cutting is known as the rock support force. The action of the rock on the retaining wall is known as the rock pressure.

   The following recommendations for excavations in unstable rock are based on the requirements for a supporting structure and therefore on the STR limit state.

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), when determining the rock pressure it shall be assumed that deformations are prevented as far as possible in order to retain the initial strength or the strength of the untouched rock. If displacements are allowed, the initial strength can be exceeded and a lower shear strength becomes decisive, leading to a possible 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 and connected tightly to the exposed face. Struts and anchors shall be prestressed to the service load F_W immediately after installation. This is obtained from the determination of the action effects, taking the action combinations according to R 79, Paragraph 1 (Section 2.4) into due consideration as the characteristic load E_k.

3. In order to realistically estimate the rock mass properties for planning and construction of the excavation, the following shall be investigated in exposures:

   – extraction particulars, (for example by excavating, scraping, ripping, drilling, blasting);
   – the mineralogical composition and the geological development of the rock (magmatic, metamorphic, sedimentary rocks);
   – the degree of weathering (sound, partially weathered, disintegrated, decomposed);

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– nature, extent and spatial arrangement of discontinuities;
– the roughness of the discontinuities and nature of the joint infill;
– existing faults and;
– the water conditions;

according to DIN 4020 and DIN 4021 Part 2 and be continuously monitored during excavating, if possible in advance, e.g. by trenching. See also [76].

4. Regardless of the supports and the type of retaining wall lining, the magnitude and distribution of the rock pressure are primarily dependent on:

– the spatial distribution of the discontinuities;
– the extent of jointing in the rock;
– the size, unevenness, roughness or waviness of the discontinuities;
– the degree of weathering;
– the rock strength;
– the shear strength of the discontinuities or the joint infill;

and the resulting rock mass strength.

5. In addition to Paragraph 4 the following apply:

a) The rock mass strength shall be determined on a sufficient number of samples using unconfined compression tests or point load tests according to Recommendation No. 1 of the "Rock Testing Procedures" (*Versuchstechnik Fels*) Working Group of the DGEG [128]. Together with data on the discontinuities this allows an estimate of the rock mass strength [129].
b) Small-scale shear tests on discontinuity samples can also provide valuable data on rock mass strength.
c) The shear strength of the bedding 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 or joint infill shall be determined as a minimum requirement.

Large-scale tests according to Recommendation No. 4 of the "Rock Testing Procedures" Working Group of the DGEG [78] are suitable to more precisely determine the shear resistance in possible slip planes. This takes the irregularities in the joint and bedding properties sufficiently into consideration.

6. The properties of the undisturbed rock can be altered by external influences. For example:

– vibrations from blasting;
– disintegration or swelling phenomena caused by access of air or water or by relaxing movements of the rock;
– 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 pressure. Also note the information in R 4, Paragraph 5 (Section 3.2).

7. Discharge of strata and joint water shall be provided for in completely lined excavations. Otherwise, the water pressure shall be taken into consideration in addition to the rock 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 retaining walls that do not completely line the excavation.
8. The elements of the excavation lining shall be designed for the rock pressure obtained according to R 39 (Section 11.2) and R 40 (Section 11.3), whereby the partial safety factors given in Table 7.1 of Appendix A 5 for the STR limit state according to R 78, Paragraph 4 (Section 1.4) are decisive.
9. The struts or ground anchors required to support the rock 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 regulation 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 in which the grouted sections are embedded:

    a) If the complete wall is installed on a rock face it is sufficient if the grouted sections are located behind the decisive slip surface.
    b) If the grouted sections are in soil or in a disintegrated, completely weathered or decomposed rock, the anchor length is given by analysis of the stability in the low failure plane according to R 44 (Section 7.3) or from the general stability analysis according to R 45 (Section 7.4).

11.2 Magnitude of rock pressure (R 39)

1. Generally, determination of the rock pressure is based on the existing discontinuities. 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 39-1 b).
   c) Stepped slip surfaces in bedding planes and joints (Figures R 39-2 a and R 39-2 b).

   For a small spacing of the discontinuities and a high joint intensity, and consequently small rock blocks compared to the size of the sliding body, it may be necessary to determine earth pressure as for soil.
2. For continuous slip surfaces, which run in a bedding plane as shown in Figure R 39-1 a), the shear strength of the jointed rock in the slip surface is decisive and the shear strength of the weaker layer if varying rock types are present. These may be only a few millimetres thick and be decomposed to soil, and may act as a slip surface between the stronger rock strata. This

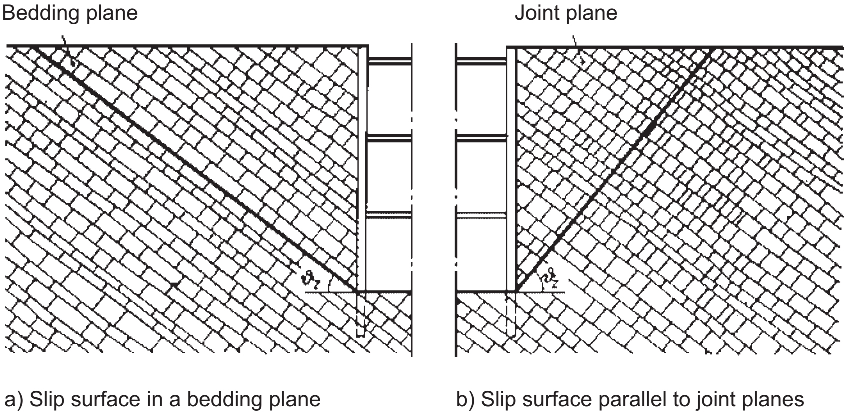

Figure R 39-1. Continuous slip surfaces in excavations in unstable rock

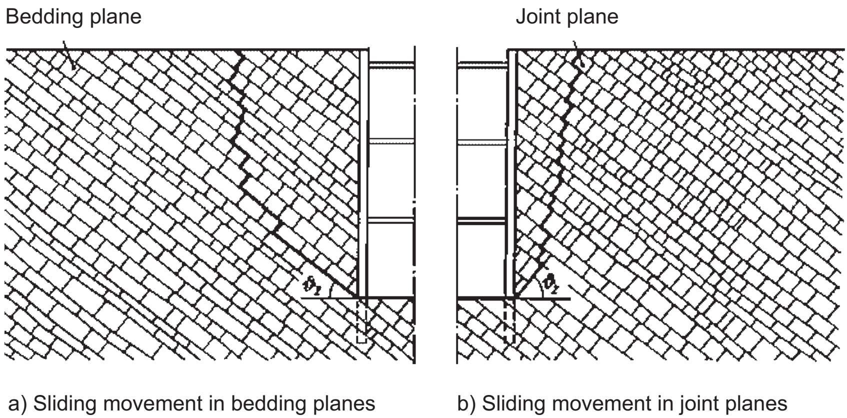

Figure R 39-2. Stepped slip surfaces in excavations in unstable rock

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 discontinuities in all construction stages, and that therefore the rock bridges have not cracked through, the rock shear strength in the material interfaces may be adopted as decisive for determination of the rock pressure.

b) If these conditions are not met it shall be assumed that the rock bridges have cracked through as a result of unavoidable movements. The shear strength of the joint infill in the existing joints and the shear strength of the material interfaces after cracking are decisive in their respective proportions. For high joint intensities the shear strength of the joint infill alone is decisive.
c) It shall be demonstrated 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 decisive for this purpose.

If there is no joint infill in cases b) and c), but a high joint intensity, analysis may be based on the shear strength of the cracked rock.

4. The shear strength of the rock and the bedding 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 grain size distribution:

   a) $\varphi'_k = 30°$ for sandy material;
   b) $\varphi'_k = 20°$ for silty material;
   c) $\varphi'_k = 10°$ for clayey material.

   The additional adoption of cohesion is generally dispensed with. Slip surfaces subject to porewater pressure shall be taken into consideration; it may be necessary to adopt the shear strength at $\varphi_{u,k} = 0$. The undrained shear strength $c_{u,k}$ may only be adopted for these slip surfaces on the basis of special 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 weight density 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 pressure and the normal to the wall may only be adopted if complete transfer of the vertical forces into the ground is guaranteed. Also see R 4 (Section 3.2).

6. If the dip is not perpendicular or the strike is not parallel to the retaining wall as viewed in plan, the required rock 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 discontinuities, which dip perpendicular or at an angle 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. Regardless of numerical determination of the rock pressure according to Paragraph 2 or Paragraph 3, a computed minimum rock pressure on the excavation lining should be adhered to analogous to the information in R 4, Paragraph 3 (Section 3.2), which is obtained according to the stipulations for earth pressure from the equivalent friction angle $\varphi'_{Equiv.} = 40°$ or $\varphi'_{Equiv.} = 45°$. This also applies if the strike is at an angle to the retaining wall.
8. If a greater rock pressure ensues on one side of a braced excavation than on the other because of the different development of the slip surfaces, the higher load is decisive for designing the whole excavation structure, if this is not lower than the computed minimum rock pressure.

11.3 Distribution of rock pressure (R 40)

1. Because the magnitude and distribution of the rock pressure are a function of the fault density of the rock, concrete rules such as for the determination of earth pressure in soil cannot be given. The actions for the rock pressure shall be selected sensibly and conservatively based on the respective determined local conditions.
2. If the complete wall is installed on a rock face, the rock pressure determined according to R 39 (Section 11.2) is generally adopted with rectangular distribution due to the rigid body mechanisms usually assumed for this case. In soil zones above the rock face and for disintegrated, completely weathered or decomposed rock, an earth pressure distribution according to the rules for soil may generally be adopted. 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 for a load from a rectangular pressure diagram.
3. To compensate for the relatively imprecise assumptions on the distribution of the rock 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 rock pressure distribution measurement results were obtained under comparable 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 service load F_w according to R 38, Paragraph 2 (Section 11.1);
   – to carry out measurements in representative sections to allow timely recognition of deviations from the analysis assumptions.

11.4 Bearing capacity of rock for support forces at the wall toe (R 41)

1. The resistance of the rock in front of the toe of a continuous retaining wall can be determined analogous to the rock pressure. Either a slip surface in a bedding plane or a slip surface parallel to the joint planes is critical. An investigation according to R 39, Paragraph 2 is decisive in the one case and according to R 39, Paragraph 3 (Section 11.2) in the other. If groundwater can occur in the area 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 is then decisive for determination of the rock resistance in front of soldier piles. A three-dimensional effect may only be adopted if the joint intensity, joint density, joint infill and joint orientation justify this. 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 without special analysis.
3. Soldier pile walls and retaining walls with a comparable support below the excavation level shall be examined for intersections of discontinuities, running upwards from the concreted borehole to the excavation level. The partial sliding masses formed in this manner can be decisive for determination of the rock resistance, in particular for shallow embedment depths.
4. A negative angle between the axis of reaction and the normal to the wall may only be adopted for determination of the rock resistance inasmuch as this is allowed by the $\Sigma V = 0$ condition according to R 9 (Section 4.8).
5. The location of the support force for a retaining wall supported below the excavation level may be adopted according to R 14, Paragraph 6 (Section 5.3) or R 19, Paragraph 4 (Section 6.3) as for cohesionless soil.
6. When determining the design resistance the partial safety factors in Table 6.3 of Appendix A 6 shall be applied to the characteristic resistance of the rock.

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:

   a) in favourable cases only above the excavation level;
   b) in less favourable cases only below the excavation level;
   c) in unfavourable cases both above and below the excavation level.

   The designation "soft soil" should be regarded as a generic term, unrelated to the consistency index definition according to DIN 18 122-1.

2. The soft soils discussed here are primarily layered, uniform, fine-grained soils according to DIN 18 196, 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:

   – soft or liquid consistency corresponding to a consistency index $I_C < 0.50$ according to DIN 18 122-1;
   – shear strength of the undrained 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 18 122-1;
   – shear strength of the undrained 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 18 196;
   – thixotropic properties, or;
   – organic constituent content.

   In individual cases, a decision to classify as soft soil on the basis of these Recommendations should not be solely dependent on a single criterion given here. However, if two of the criteria are fulfilled it can generally be assumed that a soft soil according to Paragraph 1 is present.

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5. In all cases, the situation is aggravated if more permeable soil layers or bands are intercalated in the soft soil, e.g. fine-sands, 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 a stability analysis for excavation depths up to 3.00 m and an angle of up to $\beta = 45°$ if the following conditions are adhered to according to DIN 4124 “Excavations and Trenches” (*Baugruben und Gräben*):
   a) The shear strength of the undrained 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) No buildings, pipelines, other structures or traffic areas may be endangered.
   e) The ground beside 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 1.00 m from the slope crest is permissible.
   f) On a horizontal ground surface, no earth fill inclined at more than 1 : 1 and higher than 1.50 m may be utilised beside a protective strip at least 1.00 m wide.
   g) Road vehicles and construction equipment up to and including 12 t gross weight shall adhere to a distance of at least 1.00 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.
   h) Road vehicles and construction equipment of more than 12 t 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.
   i) A berm immediately adjacent to the slope may not be subject to loads from horizontal support forces from a retaining wall.
   j) 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 18 122-1 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 demonstrate 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 18 122-1 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 18 122-1 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 with immediately following 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 “General Stability and Slope Stability Analyses” (*Gelände- und Böschungsbruchberechnungen*). The partial safety factors for Load Case LC 2 are only valid if the expected deformations do not endanger buildings, pipelines, other structures or traffic areas. If such a risk cannot be excluded as a result of the local conditions, the partial safety factors for Load Case LC 1 shall be adopted and the utilisation factor limited to $\mu \leq 0.80$ when analysing general stability. It is recommended to adopt lower utilisation factors for highly organic material. According to [110] a utilisation factor of $\mu \leq 0.75$ has proven reliable for North German tidal mud deposits with an LOI $V_{LOI} > 15\%$ 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 91 (Section 12.2) is not possible due to 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 by anchors. Only walls that will not cause appreciable settlement and horizontal movement neither in the surrounding soft ground or other structures during manufacture may be utilised as excavation linings. A settlement hazard or danger of horizontal movement exists if the soil liquefies or is displaced during installation of the wall. Generally, the following wall types are suitable for excavation 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 should 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 have 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 placed on the planned installation methods:

   a) When installing sheet piles with the aid of a pile hammer, the driving energy per impact and the impact frequency should 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 includes interbedded, saturated bands of fine sand. Sheet pile walls 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 rotation more

than 2000/min. 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 a large jacking resistance, e.g. fill ground including construction wastes, shall be prepared for jacking by pre-drilling or by soil replacement.

3. EN 1536 applies for the installation of bored pile walls. In addition, the following points should 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 by, e.g.:

      – 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 do not possess teeth but a cutting edge;
      – using drilling tools that exert as low 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 basal heave or against hydraulic heave.

   c) The following points should be observed when selecting the implementation method:

      – Drilling of 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 local projections on the wall piles. If they are manufactured using jetting techniques, the surrounding soil may be locally softened, thus presenting a settlement hazard to neighbouring structures. The cement slurry setting process is not guaranteed in organic soils.

– By driving wooden wedges, for example, squeezing of the soft soil through the unavoidable gaps can often be prevented for tangent bored piles, but does not guarantee a 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. Anger bored piles are less suitable due to the hazard 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 $I_C \leq 0.25$, direct concreting against the soil is not permissible according to EN 1536. This stipulation can be ignored above the excavation level if the pile wall is carefully examined for defects during excavation.

4. DIN 4126 and EN 1538 apply for manufacturing and analysing diaphragm walls. In addition, the following points should also be observed:

a) Where possible, the distance to neighbouring buildings, in particular to heavily loaded gable foundations, should be more than half of the trench depth, but at least 5 m, or the trench be located outside the bearing failure zone.

b) When analysing the 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:

$$\sigma_{h,k} = e_{a,k} + w_k$$

from earth pressure $e_{a,k}$ and water pressure w_k. Here, where $\delta_{a,k} = 0$,

$e_{a,k} = \sigma'_{z,k} - 2 \cdot c_{u,k}$, the initial condition of the consolidated soil

and

$e_{a,k} = \sigma'_{z,k} \cdot K_{ag} - c'_k \cdot K_{ac}$ the final condition of the unconsolidated soil

shall be investigated. The effective overburden pressure $\sigma'_{z,k}$ is obtained from the unit weight of the wet or submerged 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 slipping of single grains or grain groups, the safety against slipping of a failure wedge into the trench and the appropriate composition of the slurry should be tested on a test trench.

d) In addition, the trench execution sequence, the wet concrete pressure and the concreting technique may impact the loads on the brearing capacity of the soft soil, see [139].

5. Although not previously discussed in Paragraphs 2, 3 or 4, the selected installation or manufacturing method should always be tested on the site in question before starting work, but at a reasonable distance from existing, neighbouring structures, in order to optimise the process 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 grouting sections shall be located in soil of sufficient bearing capacity. The same applies to the grouted sections of anchors anchoring down 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 employed 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 pressure, e.g. from contact pressure or vibrations, should be utilised to install or manufacture walls or, if applicable, to manufacture wall or base slab anchorages. If necessary, an excavator with load distributing 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;
   – only 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 are partially available, the measures described may be correspondingly adjusted.

2. Regardless of the excavation depth, a continuous head beam in the shape of 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 cast-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:

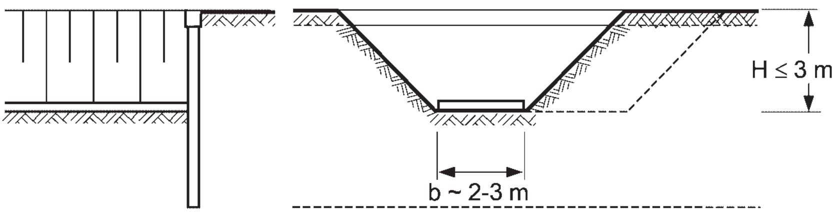
   a) Within a daily shift:

      – an approximately 2 to 3 m wide, laterally sloped trench, parallel to the narrow end of the excavation at the level of the projected excavation bottom is excavated and;
      – a stiffening strip of lean concrete is manufactured below the projected foundation level of the new building.

   b) By continuing the lean concrete in strips as shown in Figure R 93-1, a lateral support is provided to the wall at the bottom of the excavation. 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 with vertical walls;
      – bracing shall be installed at ground level according to Paragraph 4, or;
      – the core-wise construction method as described in Paragraph 6 shall be selected.



a) Longitudinal section b) Cross section

Figure R 93-1. Unsupported retaining wall in soft soil after installing the first strip of lean concrete

It may be expedient and sufficient to manufacture several lean 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 lean concrete strips in trenches sloped on one side only.

4. For medium depth excavations, generally 3 to 5 m, a low-deformation bracing strut be installed in two or more excavation stages. If the excavation plan allows, the strut 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 bottom level of this phase 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 lean 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 lean concrete.

5. The following points for manufacturing the lean concrete strips according to Paragraph 3 or Paragraph 4 shall be observed:

   a) The stiffening lean concrete should be manufactured using fast hardening concrete to facilitate a rapid work schedule.

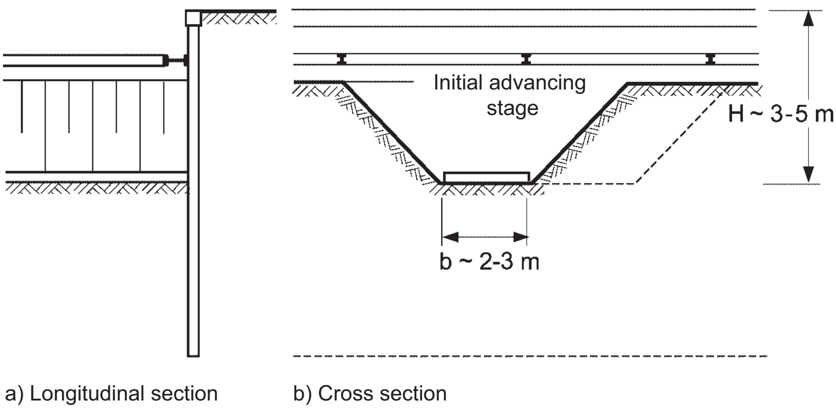


Figure R 93-2. Single-propped retaining wall in soft soil after installing the first strip of lean concrete

b) The thickness of the lean concrete depends on the structural analysis, but shall not be less than 0.20 m.
c) The lean 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 part of the foundation slab and the cellar of 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 an intermediate bracing is installed against the completed central foundation and basement, the berms then 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 bottom of the excavation.

7. In deep excavations, generally more than 5 m, in deep soft soils, it may be necessary to create a bottom 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, where necessary by anchors or using the core-wise construction method according to Paragraph 6.

8. Soft soils are particularly sensitive to dynamic loading and to changes in the initial stress condition due to 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 with fine sand and silt seems 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 realised after stabilisation of the soft soil, 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 support (bracing) system shall be implemented with little or no deformation, generally with the help of hydraulic presses and in small sections.

11. There is a permanent risk of basal or hydraulic failure in excavations in soft soil. This can lead to large swells and a substantial heave of the excavation bottom and to large settlements behind the 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 an end support.
c) Installation of base concrete slab in stages, as a vault or a reinforced flexural beam, from wall to wall.
d) Anchoring the lean concrete slab 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 absolutely necessary to monitor and measure the individual components of the retaining structure, the ground and the neighbouring structures from the outset. Also see R 31 to R 37 (Section 14) and DIN 4123. If the measurements indicate that unacceptably large movements should be anticipated, with regard to neighbouring buildings, pipelines, other structures or traffic areas, a different construction procedure shall be employed or additional measures implemented.

12.5 Shear strength of soft soils (R 94)

1. Geotechnical Category GC 3 according to DIN 4020 shall be adopted for the site investigation in conjunction with excavations in soft soils according to R 90 (Section 12.1) if the soft soil:

   a) 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 preconditions is met a geotechnical expert shall be consulted.

2. Knowledge of the prevalent and the anticipated porewater pressure conditions is of particular importance for designing retaining structures in soft soil. The geotechnical investigation should therefore clarify and detail:

   a) Whether the soft soil has already consolidated under its own weight or whether there still exist excess porewater pressure from previous construction measures.
   b) Whether excess porewater pressure is anticipated due to changes in part of the ground caused by excavating.

   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 weight density and therefore subject to undrained conditions are given in [105], [111] and [119]. Approximately drained boundary conditions can often be anticipated:

   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 in [119] 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.
   c) Remodelling of previously executed excavation structures in soft soils using the numerical methods described in [118] shows that conformity with the measured deformations can largely only be demonstrated if effective shear parameters are adopted and therefore drained conditions assumed.

   It is therefore not generally necessary to consider undrained conditions in the calculation. If excess porewater pressure development according to Paragraph 2 is anticipated and the decisive conditions are therefore undrained, local experience should also be drawn upon for the evaluation.

4. The following 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 $\varphi'_{s,k}$ of the soil according to DIN 18 137-1 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 18 137-2 or from direct shear tests according to DIN 18 137-3. 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 in installing the sample in the shear box or triaxial cell can lead to strength reductions.
   b) Increases in the 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, should be carefully assessed when stipulating the computation values. A cohesion of $c'_k \approx 0$ is normally anticipated in any case for normally consolidated soft soils without organic constituents, giving $\varphi'_k \approx \varphi'_{s,k}$.

6. Failing the relevant experience for excavations in soft soils, the in-situ undrained shear strength $c_{u,k}$ of the soil should be determined by vane shear tests according to DIN 4096, in addition to the usual site investigation measures and laboratory tests. These tests should be carried out to a depth at which the soil strength noticeably improves, but 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 different 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 18 122-1 as shown in Figure R 94-1 (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 18 122-1 for short-term construction stages according to [116] in [132].

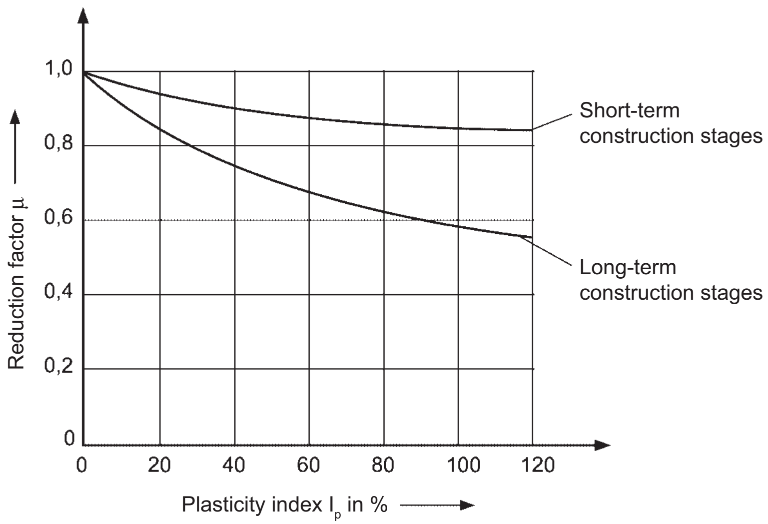


Figure R 94-1. Reduction factor μ when using the shear vane to determine the shear strength $c_{u,k}$

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 as follows, in the course of geotechnical investigations:

   a) If relevant regional experience or reliable correlations are available [112], the undrained 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 with the consistency index I_C is not recommended [106].

8. Taking the variance of the measurement results into consideration, the decisive calculation value for the shear strength $c_{u,k}$ should adopted so that analyses provide conservative results on the safe side. 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 due to soil excavation, the undrained shear strength $c_{u,k}$ of the soil shall normally be increased when determining the active 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).

10. If excess porewater pressure is anticipated in specific situations, the shear strength $c_{u,k}$ for each layer should 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:

   $$\sin(\text{equiv.}\,\varphi_{s1,k}) = \frac{c_{u1,k}}{\sigma'_{v1,k}}$$ above the groundwater table

   where $\sigma'_{v1,k} = \gamma_1 \cdot z_1$

   $$\sin(\text{equiv.}\,\varphi_{s2,k}) = \frac{\Delta c_{u2,k}}{\sigma'_{v2,k} - \sigma'_{v1,k}}$$ 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:

   $$\sin(\text{equiv.}\,\varphi_{s1,k}) = \frac{c_{u1,k}}{\sigma'_{vm1,k}}$$ above the groundwater table

   where $\sigma'_{vm1,k} = \frac{1}{2} \cdot \gamma_1 \cdot z_1$

   $$\sin(\text{equiv.}\,\varphi_{s2,k}) = \frac{c_{u2,k}}{\sigma'_{vm2,k}}$$ below the groundwater table

   where $\sigma'_{vm2,k} = \gamma_1 \cdot z_1 + \frac{1}{2} \cdot \gamma'_2 \cdot z_2$

   An additional boundary may be introduced at the bottom of 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.

11. If excess porewater pressure can be ruled out in certain concrete 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.

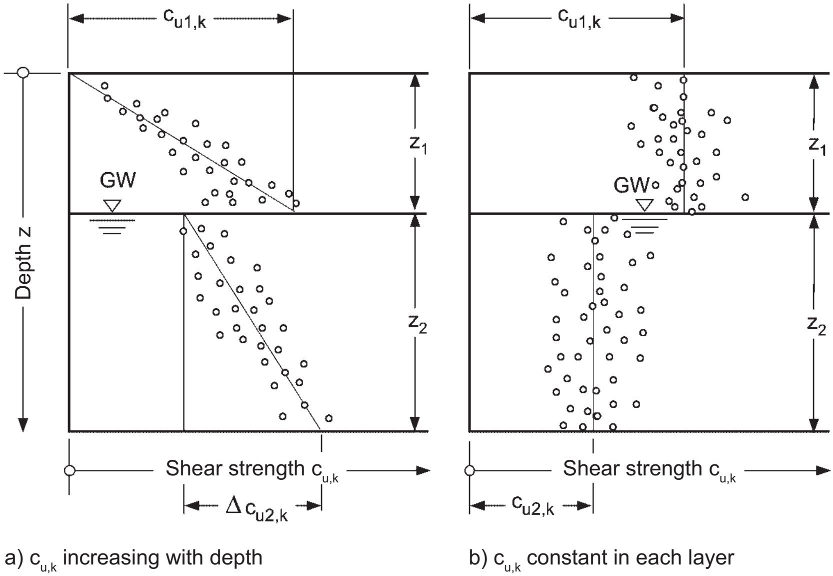


Figure R 94-2. Determination of the equivalent friction angle equiv. $\varphi_{s,k}$

12.6 Earth pressure on retaining walls in soft soils (R 95)

1. The same principles apply for determination of the earth pressure magnitude and distribution on retaining walls in soft soils as for excavations in other type of 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 active and the passive earth pressure alone, taking the water pressure into consideration separately, often produces unreliable result because:

   – arithmetically, no earth pressures acts to a depth depending on the magnitude of the parameters $c_{u,k}$;
   – active and passive earth pressure increase equally with depth, therefore leading to decreasing numerical 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);
b) as an equivalent friction angle equiv. $\varphi_{s,k}$ according to R 94, Paragraph 10, based on the shear strength $c_{u,k}$;
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 should 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 other type of 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 approximating are available for the at-rest earth pressure coefficient of normally consolidated soils under weight density:

a) The usual approach as a function of the friction angle is:

$$K_0 = 1 - \sin \cdot \varphi_s$$

b) As a function of the plasticity index the approach:

$$K_0 = 0.24 + 0.310 \cdot \log I_P$$

after *Lee* and *Jin* (1979) is adopted [112]. The plasticity index is given in %.

c) As a function of the water content w_L at the liquid limit the approach:

$$K_0 = 10^{0.00275 \cdot (w_L - 20) - 0.2676}$$

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) $K_0 = 1 - \sin \varphi_s$		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 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 towards 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 first 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 should be 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 connection 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 should always be examined whether the selected earth pressure approach approximately conforms to the computed deformations and deflections of the wall. At the least, it should not obviously contradict the determined deformations and displacements.

6. The following apply for determination of the active earth pressure:

   a) Similar to other type of 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. As a simplification, it is permissible to adopt the wall friction angle $\delta_{a,k} = {}^{1}/_{3} \cdot \varphi_k$ in the place of the 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 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 first top support is installed relatively deep;
      – if a wall support at toe is secured by the ground reaction;
      – 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 projected construction procedure. However, if an upper support is prestressed on the one hand, but the wall at support toe is secured by 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 transformed to a trapezoidal or, at the most, a rectangular type of distribution.

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 connection with R 97 (Section 12.8).
b) See R 99, Paragraph 6 (Section 12.10) for consideration of changes in layered soil.

12.7 Ground reactions for retaining walls in soft soils (R 96)

1. The ground reaction below the excavation level can take any value between the active earth pressure and the passive earth pressure in the limit state, depending on 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 with a stiffening base slab 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 to absorb the ground reaction according to R 95, Paragraph 3 (Section 12.6), was determined using the angle of total shear strength $\varphi'_{s,k}$ or the equivalent friction angle equiv. $\varphi_{s,k}$.

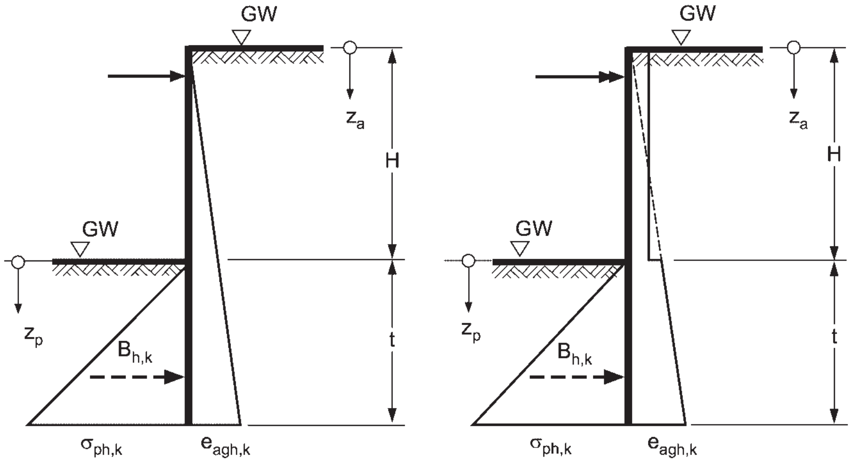
2. In construction stages without a stiffening base slab as shown in Figure R 96-1, equilibrium of horizontal forces can only be achieved if a ground reaction is mobilised in front of the wall toe. The ground reaction may be adopted as increasing linearly with depth, similar to the passive earth pressure. Assuming a submerged 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 instead of adhesion as a simplification if the special case shown in Figure R 99-3 (Section 12.10) does not apply.

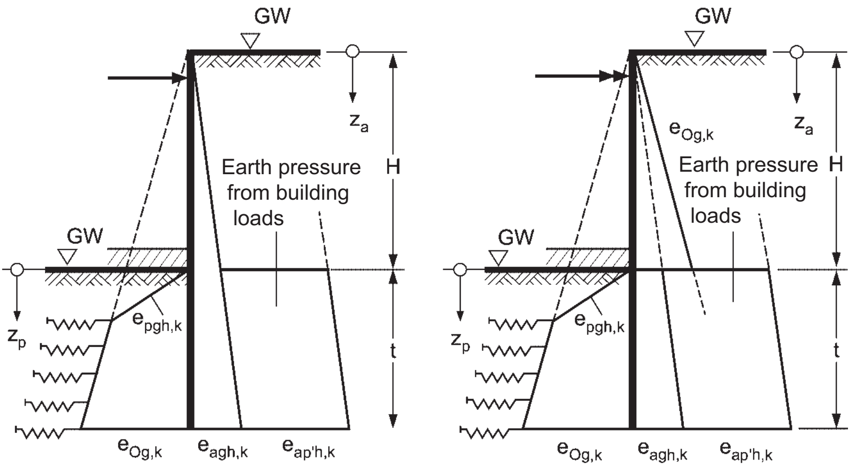
   Otherwise, the following shall be observed:

   a) Due to the anticipated deflections of the top of unsupported walls, a fixed end earth support in soft soils is only possible for shallow excavation depths.
   b) Because of the large difference in stiffness of a supported retaining wall and of the ground, a fixed end earth support shall not be adopted under any circumstances in soft soils.



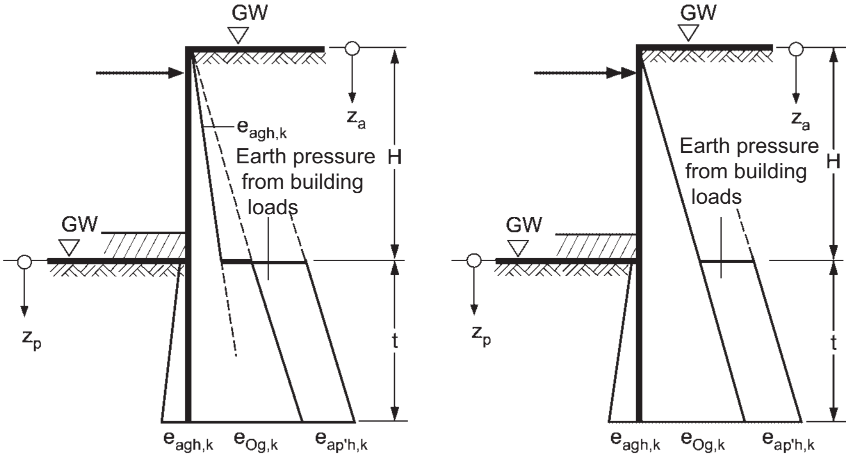
a) Without earth pressure redistribution b) With earth pressure redistribution

Figure R 96-1. Possible load diagrams for single-propped walls without support at the excavation level



a) Without strut prestressing b) With strut prestressing

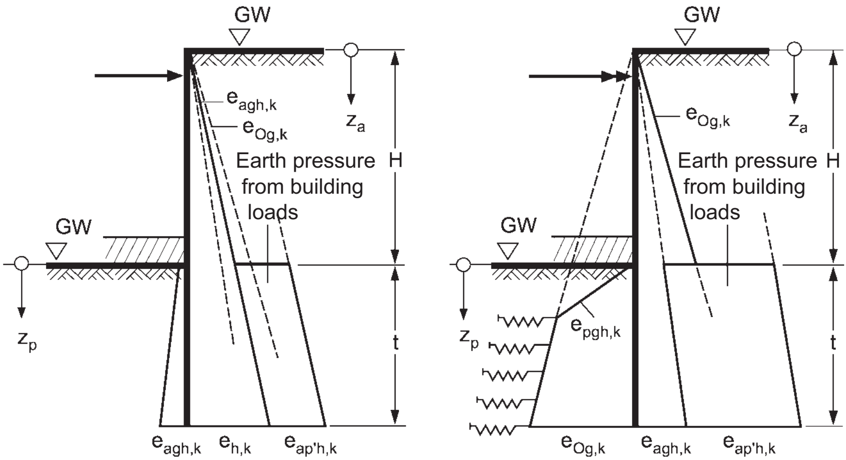
Figure R 96-2. Possible load diagrams for single-propped walls with a base slab installed in stages at excavation level



a) Without strut prestressing

b) With strut prestressing

Figure R 96-3. Possible load diagrams for single-propped, low-deformation walls with a base slab installed by jet grouting at excavation level



a) Without strut prestressing

b) With strut prestressing

Figure R 96-4. Possible load diagrams for single-propped, flexible sheet pile walls with a base slab installed by jet grouting at excavation level

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 fulfiled to using the adjustment 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 support reaction is only as large as the passive earth pressure design value:

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

   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 fulfiled to using the adjustment factor $\eta_p \leq 0.75$:

   $B_{h,k} \leq E_{0g,k} + (E_{ph,k} - E_{0g,k}) \cdot \eta_p$

   The adjustment 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 support reaction $B_{h,k}$ to a value corresponding to a coefficient of passive earth pressure of $K_{ph,mob} \leq 1.00$ is meaningless.

4. In construction stages with a stiffening base slab 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 should already be installed before appreciable wall deflections occur at this depth, it may be assumed, as an approximation, that the original at-rest earth pressure is largely retained below the base slab even after excavation is completed. Assuming a submerged soil in its initial state, 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 weight density 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 action 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 surcharge 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 $e_{ph,k}$.
- Depending on the constrained modulus $E_{S,h,k}$ for horizontal loading, the modulus of subgrade reaction may be approximately based on the constant value:

  $k_{s,k} = E_{S,h,k} / t_B$

  whereby t_B is the distance between the point of intersection of $e_{0g,k}$ and $e_{ph,k}$ and the wall toe, see R 102, Paragraph 5 (Section 4.6).
- In analogy to R 102, Paragraph 4 (Section 4.6), the most reliable values for the modulus of subgrade reaction $k_{s,k}$ are obtained from the resistance-deflection relationship using a mobilisation curve [126, 140].
- The characteristic ground reaction mobilised by the modulus of subgrade reaction shall fulfil 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 ground reaction from the subgrade stress $\sigma_{h,k}$;
  $E_{V,k}$ the characteristic resultant of the remaining at-rest earth pressure in the final excavation state, taking the original preloading condition into consideration, see R 102, Paragraph 4 (Section 4.6);
  η_p the adjustment 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 accept support reaction without noticeable displacement.

5. In construction stages with a stiffening base slab 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. Otherwise, the following points need attention:

   a) Because the retaining wall may be pushed 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 submerged 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 excavation level is so great that the wall bends back towards the excavation, e.g. as a result of the effect of surcharge 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 the determination of the passive earth pressure below the groundwater table in connection with R 97 (Section 12.8).
   b) See R 99, Paragraph 6 (Section 12.10) for consideration of changes in soil layer.

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 with a stiffening base slab 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 connection with R 98, Paragraph 2 (Section 12.9).
   b) In construction stages utilizing soil stabilization 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 basal failure according to R 99, Paragraph 2 (Section 12.10), against hydraulic failure according to R 99, Paragraph 3 or, if applicable, 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 section forces.

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 prevails. If the wall is embedded in a load-bearing, unpermeable layer, the procedure is as follows:

   a) The water pressure on the outer side 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 inner side of the wall as shown in Figure R 97-1 a) is obtained from:

   $$w_{p,k} = \gamma_w \cdot z_p$$

   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 finally superimposed, so that only the net water pressure $w_{p,k}$ 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 available for the treatment of water pressure according to R 63 (Section 10.6):

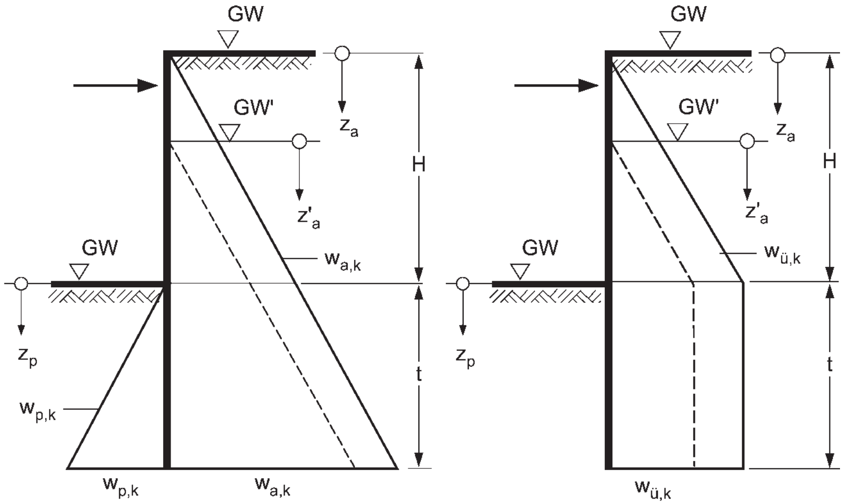
   a) As an approximate, simplified approach, it is assumed that the wall is embedded in an impermeable, load-bearing layer. The real seepage around the wall is not considered. The decisive approach is therefore according to Paragraph 1 (Figure R 97-1).

   b) In more precise procedures the seepage around the wall is considered. See also 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 outer side and
   $w_{p,k} = (\gamma_w + i_p \cdot \gamma_w) \cdot z_p$ on the inner side

   of the wall. The two components are superimposed so that only the net water pressure $w_{p,k}$ is taken into consideration for further analysis. Also see Figure R 97-2 b).

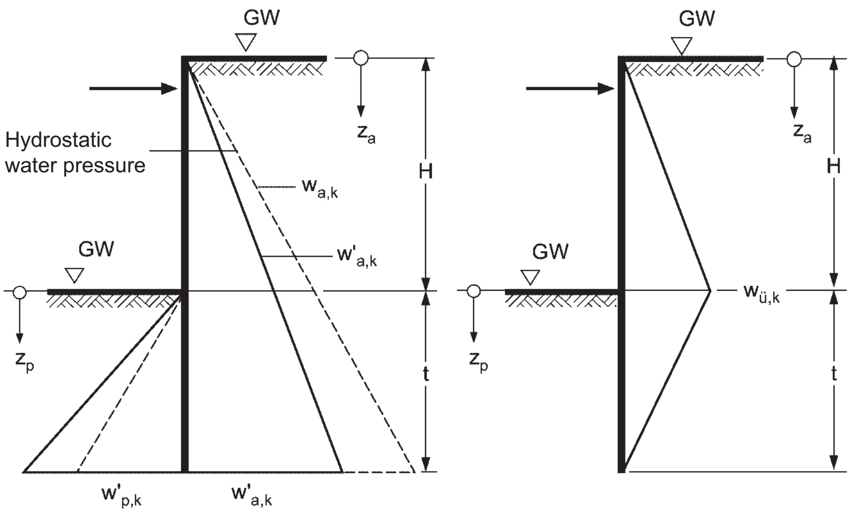
   c) The procedures discussed in R 59 (Section 10.2) for determination of the seepage pressure provide differing results depending on the simplification they are based on.



a) Hydrostatic water pressures

b) Hydrostatic positive water pressures

Figure R 97-1. Water pressure for embedment of the wall in an impermeable layer



a) Effective water pressures

b) Effective positive water pressures

Figure R 97-2. Water pressure with seepage around the wall toe
(simplified representation)

The simplified case shown in Figure R 97-2 is based on the assumption that:

$i = i_m = i_a = i_p = \Delta h / l = H / (H + 2 \cdot t)$

See also 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 approximate, simplified solution according to Paragraph 2 a) the active earth pressure and the at-rest earth pressure are determined using the submerged unit weight γ' according to R 95 (Section 12.6).

   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 outer side and
   $\gamma'_p = \gamma' - i_p \cdot \gamma_w$ on the inner side

   of the wall are decisive.

   Similar approaches are also discussed for the passive earth pressure in the following two Paragraphs.

4. If the simplified water pressure approach without consideration of seepage according to Paragraph 2 a) is adopted, the following approaches apply for the ground reactions on the inner side 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 adjustment 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 with a base slab installed in strips or a stabilised soil layer below the excavation level:

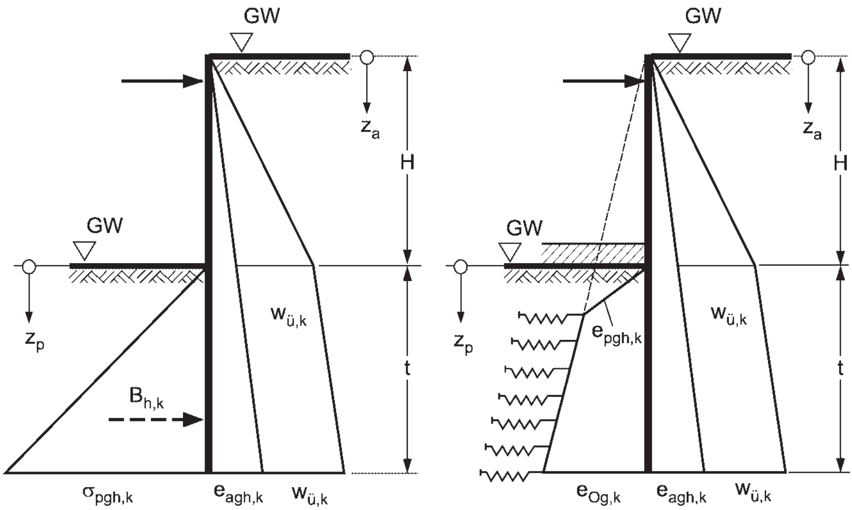
   $e_{0g,k} = \gamma' \cdot K_0 \cdot (H + z_p)$, but a maximum of
   $e_{pgh,k} = \gamma' \cdot K_{pgh} \cdot z_p$ as shown in Figure R 97-3 b)

   and not greater than sum of the characteristic loads from earth pressure and water pressure on the outer side of the wall below the excavation level. If the sum of the loads is greater, a subgrade reaction according to R 96, Paragraph 4 b) (Section 12.7) may additionally be adopted.

   c) Construction stages with a base slab installed by jet grouting:

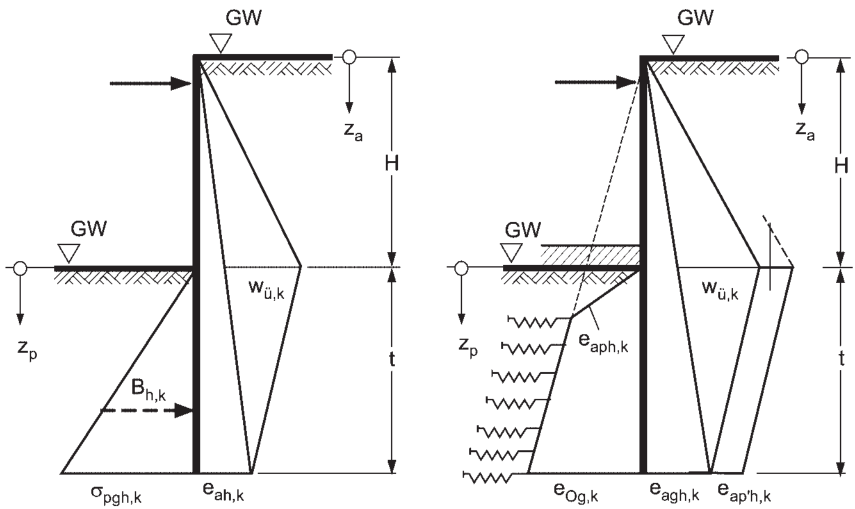
   $e_{ah,k} = \gamma' \cdot K_{agh} \cdot z_p + p_k \cdot K_{agh}$

   for a low-deformation wall; otherwise, R 96, Paragraph 5 b) (Section 12.7) applies.



a) Without stiffening base slab   b) With stiffening base slab

Figure R 97-3. Load diagrams for single-propped walls with hydrostatic water pressure



a) Without stiffening base slab   b) With stiffening base slab

Figure R 97-4. Load diagrams for single-propped walls when adopting seepage pressure (simplified representation)

5. If a more precise water pressure approach including consideration of seepage according to Paragraph 2 b) is adopted, the following approaches apply for the passive earth pressure or the at-rest earth pressure on the inner side of the wall:

   a) Construction stages without a stiffening base slab as shown in Figure R 96-1:

      $\sigma_{ph,k} = (\gamma' \cdot K_{pgh} \cdot z_p) \cdot \eta_{eff}$

      as shown in Figure R 97-3 a). The effective adjustment 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 with a base slab installed in strips or a stabilised soil layer 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 sum of the characteristic loads from earth pressure and water pressure on the outer side of the wall below the excavation level. If the sum of the loads 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 additionally be adopted.

   c) Construction stages with a base slab installed by jet grouting as shown in Figure R 96-3:

      $e_{ah,k} = \gamma'_p \cdot K_{agh} \cdot z_p + p_k \cdot K_{agh}$

      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 excess 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 as a 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) should 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 the earth pressure;
   – R 97 (Section 12.8) for determination of the 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 lean concrete on one side and sloped on the other.

   b) For analysis:

      – a temporary arching effect in the soil;
      – the load redistribution 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 lean 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 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 excavation level according to Paragraph 4;
      – the analysis of heave failure safety according to R 99, Paragraph 2 (Section 12.10), without the subsequent lean concrete surcharge;

– the analysis of safety against hydraulic failure according to R 99, Paragraph 3 and, if applicable;
– the analysis of safety against global failure according to R 99, Paragraph 4.

b) Work should commence at a non-critical location. A small trench width should initially be selected and can then be optimised in the course of work on the basis of monitoring and measurements.

c) The deflections, settlements and heave of the wall and its surroundings shall be carefully monitored during manufacture of the lean concrete strip or while installing additional bracing.

d) If it is not possible to install the lean concrete strip or other additional bracing on daily capacity basis according to R 93, Paragraphs 3 and 4 (Section 12.4), due to unforeseen circumstances, a condition that is numerically demonstrated as being safe shall be achieved by other means before work ends, e.g. by reinstating the condition prevalent before commencement of the planned 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 lean 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 excavation 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;
– but on the other hand, that lateral regions exist that are either still supported by soil or already supported by the stiffening lean concrete.

The groundwater level within the excavation shall be adopted for this analysis at the actual excavation level.

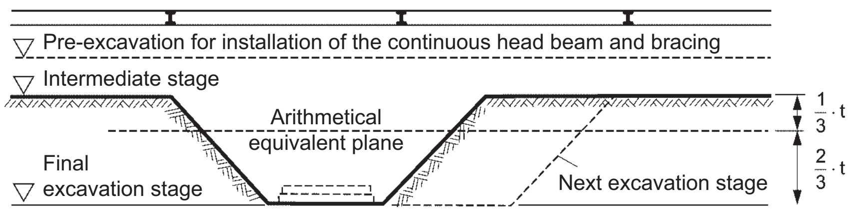


Figure R 98-1. Equivalent excavation level for the intermediate stage with trench

5. For a single-propped wall with a free earth support, both the classical earth pressure distribution and an earth pressure redistribution can be decisive according to R 95, Paragraph 7 (Section 12.6). If in doubt whether analysis should 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 ignored 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 basal heave failure, hydraulic failure and global failure, as well as the additional investigations for stratified ground.

12.10 Further 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. Further stability analyses to those described in R 98 (Section 12.9) are required in a number of the cases discussed. Also see Paragraphs 2 to 6.
2. Excavations in homogeneous soft soil as shown in Figure R 99-1 a) are seriously threatened by basal heave failure, also see R 10, Paragraph 1 (Section 4.10). 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 the basal heave failure at the excavation level is generally performed using the undrained shear strength $c_{u,k}$ of the soil. The following shall be observed in detail:

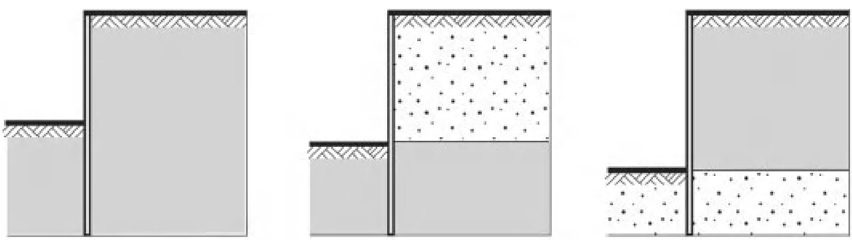


Figure R 99-1. Excavations in stratified ground (without representation of the supports)

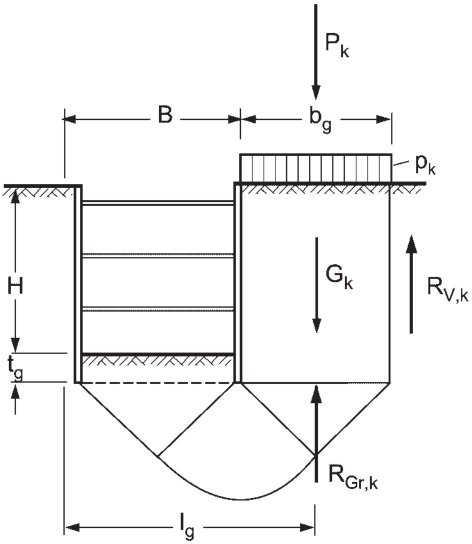


Figure R 99-2. Excavation base heave

a) For excavations of depth H and width B > 0.20 · H in homogeneous, saturated soil as shown in Figure R 99-2, the limit state condition analogous to R 10, Paragraph 1 (Section 4.10):

$$G_d + P_d \leq R_{v,d} + R_{Gr,d}$$

shall be fulfilled [130]. The partial safety factors γ_G for permanent and γ_Q for variable actions shall be taken into consideration for the soil weight density G_k and the surcharge P_k, 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:

$$R_{v,d} = \frac{c_{u,k} \cdot (H + t_g)}{\gamma_{Gr}}$$

the design value of the bearing capacity from:

$$R_{Gr,d} = \frac{b_g \cdot (\gamma \cdot t_g + 5{,}14 \cdot c_{u,k})}{\gamma_{Gr}}$$

The unit weight γ is adopted as γ_r if the soil below the excavation level is saturated, or as γ' if it is buoyant.

c) The width b_g is obtained as follows:

   – The decisive width $b_g = B$ is obtained without lateral surcharges if the undrained shear strength of the soil $c_{u,k}$ is constant with depth.
   – For lateral surcharges and variable undrained shear strength $c_{u,k}$ the width shall be varied in order to identify the maximum utilisation factor [130].

   The heave failure hazard is reduced for excavations with a width $B \leq 0.20 \cdot H$. See also [52] and [117].

d) Because of the anisotropy of the soil as a result of sedimentation and the rotation of the principal stress directions due to 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.

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 failure. 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 ground as shown in Figure R 99-1 b). The following 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 limit, lower utilisation factors should be adopted for in soft soils than for load-bearing soil types. See also 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 in the analysis as acting favourably.

5. The stability of deep rupture failure is analysed according to R 44 (Section 7.3) for anchored retaining walls. The following shall be observed:

   a) The starting point of the deep rupture failure plane is generally the toe of the retaining wall.
   b) In excavations with a change of soil layers at the excavation level as shown in Figure R 99-1 b), the anchored block shall generally be supported by a set of struts at lower positions, base concrete slab installed in stages or by jet grouting.
   c) In excavations with alternating layers of soft and load-bearing soils, a deep rupture 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.

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 shall be observed:

   a) When determining the active earth pressure the active wall friction angle shall be adopted at $\delta_{a,k} = 0$, because a transfer of the vertical component of the earth pressure to the underground is not guaranteed.
   b) In braced excavations as shown in Figure R 99-3 a), analysis of sufficient embedment depth in the load-bearing cover layer shall be performed. A wall friction angle shall be adopted at $\delta_{p,k} = 0$ to ensure that no computed slip surface through the soft soil becomes decisive. 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):

      $E_{ah,d} + W_{p,d} \leq E_{ph,d} + R_{t,d}$ 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:

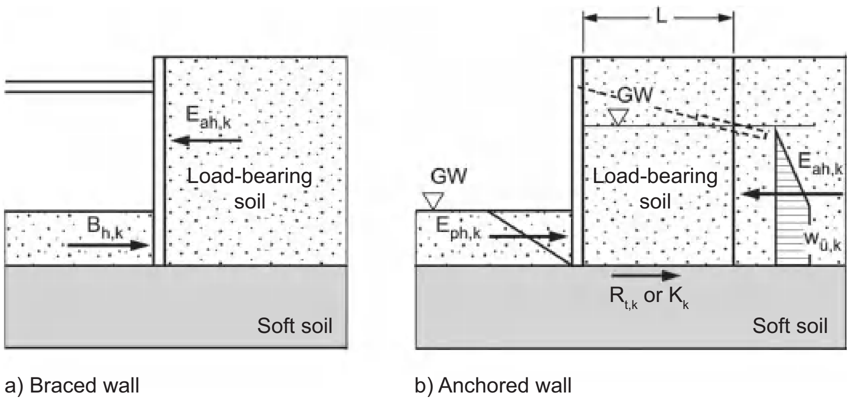


Figure R 99-3. Excavation with soft soil below the wall toe

$R_{t,k} = G_k \cdot \tan \varphi_k$

where $\varphi_k = \varphi'_{s,k}$ or φ_k = equiv. $\varphi_{s,k}$ according to R 95, Paragraph 3 (Section 12.6) or;

$K_k = c_{u,k} \cdot L$

The smaller value is decisive.

12.11 Drainage measures in 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 retained according to R 92, Paragraph 1 (Section 12.3). The wells should terminate above the toe of the retaining wall in order to limit the effects of dewatering outside the excavation. Vacuum filter wells should be employed if gravity dewatering is insufficient or if additional densification of the soil is aimed for.
4. The localised use of vacuum lances for stabilising slopes, e.g. when manufacturing trenches for installing base concrete slab 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 should always be collected in filter stable surface drains according to DIN 4095 and sent to pump sumps. The pump sumps should 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:

   – the accurate investigation and assessment of the given situation;
   – the selection of a suitable wall and base slab;
   – the selection of a suitable construction method;
   – realistic approaches for analysis and design;
   – 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 far longer 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 manufacture of the structure in 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 of prime importance 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 subsoil. It is of decisive importance to limit relaxation and loosing 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 basal failure 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 will not substantially change if struts are used instead of tie back anchors.

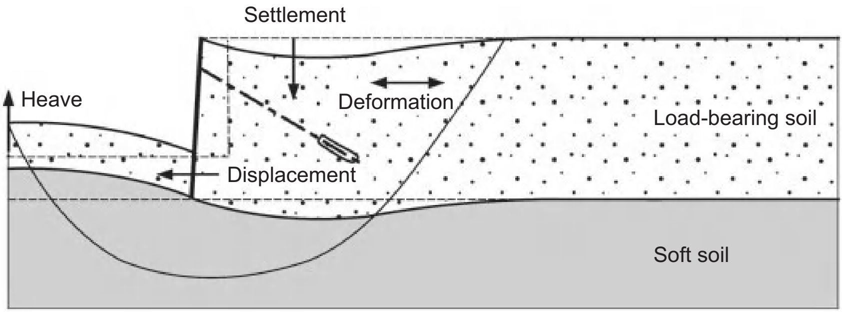
b) If additional stiffening base slab is installed as shown in Figure R 101-1 b) the toe support is largely free from deformation, but even with a numerically adequate safety against basal failure it is possible for the excavation floor to heave, leading to settlement behind the retaining wall.
c) If 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.

The favourable impact of stiffening base slab or ground anchors is increased if these are installed before excavation begins instead of in stages after reaching the excavation level.

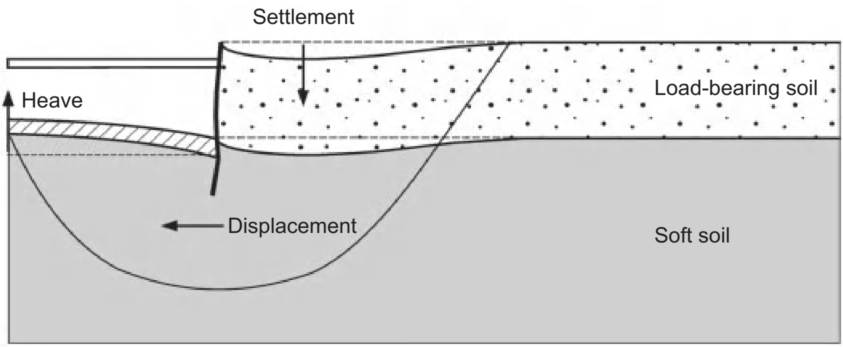
Three cases with varying depth of the layer boundary between load-bearing soil and soft soil are shown in Figure R 99-1 (Section 12.10), representing increasing demands on stabilisation measures.

4. Conservation of evidence measures should 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 should be accompanied by settlement measurements in the vicinity 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 whilst 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 be stopped and, for advanced excavations, supporting berms tipped or the excavation partly backfilled until the settlement process ceases.

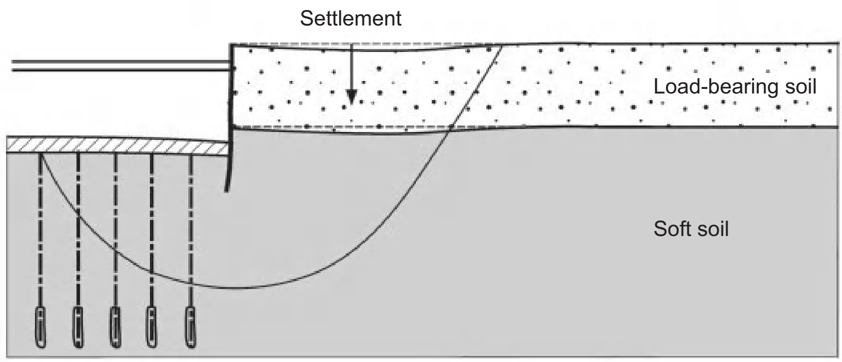
5. The manufacture of excavations in soft soil without settlement impacts on neighbouring buildings is only possible in conjunction with highly favourable boundary conditions. Deformation resulting from the excavation process is generally unavoidable, in particular with increasing excavation depth. These deformations cannot be determined with sufficient precision using classical analytical methods. In contrast, numerical methods, e.g. based on finite-element methods (FEM) according to R 103 (Section 4.7), can provide approximately correct deformation figures when realistic material properties are adopted. If possible, the FEM model employed should be calibrated using measurement results taken from an excavation in similar subsoil conditions. The use of FEM analyses is particularly valuable if the deformation-reducing effect of any additional support measures needs to be made visible. The plausibility of the adopted earth pressure distribution or earth pressure redistribution can also be visualised in an FEM analysis.



a) Without base stabilisation



b) With base stiffening



c) With base stiffening and soil anchors

Figure R 101-1. Ground movements as a function of base slab stabilisation

13 Verification of 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 state are given by:
   - DIN 1045-1:2001-08 for concrete or reinforced concrete structural elements;
   - DIN 18 800-1:1990-11 and DIN 18 800-2:1990-11, or ENV 1993-1-1:1993-04 and DASt Guideline 103 for steel structural elements;
   - DIN 1052:2004-08 for timber structural elements.
2. The following points on adopting the partial safety factors from the regulations given in Paragraph 1 should be noted:
   a) See R 24 (Section 2.1) and R 79 (Section 2.4) for definitions of load cases.
   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 Load Cases LC 2, LC 2/3 and LC 3, if not otherwise stated in individual cases.
   c) The partial safety factors for Load Case LC 1 are reproduced in Tables 6.1 and 6.2, Appendix A 6, but are given in brackets because they are not generally decisive for retaining structures according to R 79, Paragraph 1 (Section 2.4).
3. The material parameters and partial safety factors given in DIN 1045-1 for concrete or reinforced concrete structural elements are summarised in Appendix A 7.
4. The material parameters and partial safety factors given in DIN 18 800-1 for steel structural elements are summarised in Appendix A 8, taking Amendment A 1 to DIN 18 800-1 into consideration. The following points should be noted in detail:
   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 10 248-1.
   c) It is pointed out that, under certain conditions, the elastic-elastic analysis method allows the stresses to be increased by 10% for analysis, based on Amendment A 1 to DIN 18 800-1.

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d) The allowable partial safety factors given require that any weakening of steel sections due to drilling, transverse welding or significant corrosion be taken into consideration in zones with large bending moments when analysing bearing capacity.
e) See DIN 18 800-1:1990-11, Section 8 for loads and allowable stresses of connections.
f) The full table values may be used when stipulating the stiffnesses of the steel sections.

Alternatively, ENV 1993-1 and DASt Guideline 103 may be adopted instead of DIN 18 800-1. With the exception of the additional information given in Paragraph c) there are no differences in the numerical data given in the tables.

5. The material parameters and partial safety factors given in DIN 1052 for timber structural elements are summarised in Appendix A 9. The following points should be noted in detail:

a) Quality classes C 24 and C 30 correspond approximately to the previous quality classes S10/MS 10, GK II, and S 13, GK I.
b) The given material parameters and partial safety factors assume that new or practically new timber is employed.
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:

$E_d \leq R_{M,d}$

shall be analysed for the design action effects determined according to Section 5. The design value E_d consists of the loads from the most unfavourable combination of action effects, the design value $R_{M,d}$ of the resistance of the structural element. The individual analyses depend on the material used.

a) The following are obtained in the case of analysis of the normal bending stresses of timber planks with uniaxial bending according to Paragraph 5:

$$\text{from} \quad E_d = \sigma_{m,d} = \frac{M_d}{W_{y,n}} \quad \text{and} \quad R_{M,d} = f_{m,d} = \frac{k_{mod} \cdot f_{m,k}}{\gamma_M}$$

the general design equation:

$\sigma_{m,d} \leq f_{m,d}$

Where:

$f_{m,k}$ the characteristic failure stress 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 resistance 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 decisive characteristic earth pressure for determination of the bending load is obtained as follows:

   a) When adopting the active earth pressure due to soil weight density, 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 is decisive. 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), is decisive.

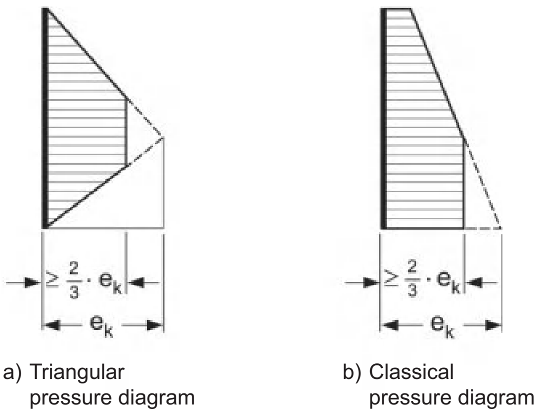

Figure R 47-l. Reduction of earth pressure from soil weight density, unbounded uniform load and, if applicable, cohesion when dimensioning infilling

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 2 (Section 9.3), as shown in Figure R 47-2.

The thickness of the infilling may be adjusted to the respective selected diagram.

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 loads on the infilling in mid-span may be taken into consideration if:

- either medium-dense or densely compacted, 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 friction 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.

See also [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%.

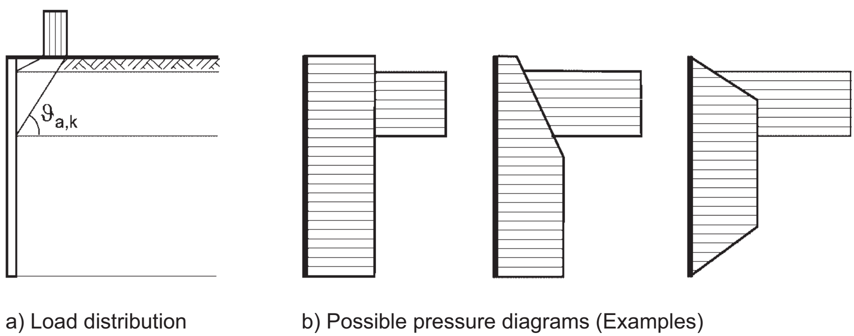

Figure R 47-2. Earth pressure from live loads for infill wall dimensioning

In order to take the arching effect into consideration for dimensioning of 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 dimensioning 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 4 or;
   b) if individual infilling 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 earth pressure angle $\delta_{a,k}$, determined according to Paragraph 3 or Paragraph 4.

5. The following routes may be taken to determine the design loads:

   a) As an approximation, the greatest load ordinate determined according to Paragraph 2 or Paragraph 3 is divided into one component from permanent actions and one from changeable actions. The characteristic bending loads $M_{G,K}$ 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 (Section 4.12) before it is superimposed by the earth pressure from soil weight density, 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 from:

      $M_d = M_k \cdot \gamma_G$

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 by:

   – DIN 1045-1 for concrete or reinforced concrete infilling, see Appendix A 7;
   – DIN 18 800 Part 1 and Part 2 for steel infilling, see Appendix A 8, or EN 1993-5 for horizontal trench sheet piles and vertical lightweight sheet pile walls;
   – DIN 1052 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:

   $E_d \leq R_{M,d}$

   shall be analysed for the design action effects determined according to Section 5. The design value E_d consists of the loads from the most unfavourable combination of action effects, the design value $R_{M,d}$ of the resistance of the structural element. For example, in the simplest case of a double-symmetry steel soldier pile with uniaxial bending according to Paragraph 3, Clause 1 and Paragraph 4, Clause 1, an elastic-elastic analysis of the normal stresses:

   $$\text{from} \quad E_d = \sigma_{y,d} = \frac{N_d}{A} \pm \frac{M_d}{W_{y,n}} \quad \text{and} \quad R_d = f_{y,d} = \frac{f_{y,k}}{\gamma_M}$$

   the general design equation:

   $\sigma_{y,d} \leq f_{y,d}$ is obtained.

   Where:

   $f_{y,k}$ the characteristic yield stress according to Appendix A 8;
   γ_M the partial safety factor according to Appendix A 8;
   N_d the design normal force;
   A the net cross-sectional area;
   M_d the design moment;
   $W_{y,n}$ the net resistance moment.

2. When designing soldier piles, the dead-load of the excavation structure may be disregarded. Beside normal stresses, however, shear stresses and equivalent stresses shall be analysed in every case.

3. If no vertical forces other than the dead-load of the excavation structure and the vertical earth pressure component need to be transmitted, a bearing capacity analysis according to DIN 18 800-1 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 DIN 18 800-2 shall be undertaken, in particular for single-propped retaining walls and for the individual retreating states of multiple-propped retaining walls.

4. The girder spacing should be as uniform as possible. If the spacing differs greatly between neighbouring girders specials measures shall be taken to prevent rotation of the girders as a result of variable loading from the infilling.

5. In the case of a soldier pile wall infill wedged behind the front flange faces, it can be assumed that the front soldier pile flanges are secured against deflection

by the infill whilst flanges on the soil side are protected against deflection by the surrounding soil. 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 flanges on the excavation side.

6. See R 51 (Section 13.6) for details of the configuration and dimensioning of waling and tie rods in front of soldier pile walls.
7. 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 stress analysis may be dispensed with if the batten spacing does not exceed 1.5 m. Stability analysis is not required if sections, except their facing sides, are fully embedded in concrete.
8. Analysis of flange bending as a result of the infill support forces can generally be dispensed with for single and double I- and U-sections.
9. The characteristic material parameters and the partial safety factors are given by DIN 18 800-1, see Appendix A 8. In standard cases the elastic-elastic method according to DIN 18 800-1:1990-11, Section 7.5.2, shall be adopted. Otherwise, the following apply:
   a) As a simplification when stipulating the stiffness design values for analysis of bearing capacity, a partial safety factor of $\gamma_M = 1.0$ may be adopted, see DIN 18 800-1:1990-11, Section 7.3, Element 721. If the stability analysis according to DIN 18 800-2 is decisive for design, $\gamma_M = 1.1$ shall be adopted.
   b) See DIN 18 800-1:1990-11, Sections 7.3 to 7.5 and R 27 (Section 4.5) for design using the elastic-plastic and plastic-plastic methods.
   c) See DIN 18 800-1:1990-11, Section 8 for the allowable stresses for connections.
10. R 50 (Section 13.5) applies accordingly to the design of reinforced concrete piles with infilling installed in the spaces.
11. See R 85 (Section 13.10) for analysis of the external bearing capacity, i.e. transmission 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:

   $E_d \leq R_{M,d}$

   shall be analysed for the design action effects determined according to Section 6. The design value E_d consists of the loads from the most unfavourable combination of action effects, the design value $R_{M,d}$ of the resistance of the structural element. In the simplest case using the analysis methods given in DIN 18 800-1 a steel sheet pile wall with uniaxial bending is obtained. For example using the elastic-elastic analysis method to analyse normal bending stresses:

$$\text{from} \quad E_d = \sigma_{y,d} = \frac{M_d}{W_{y,n}} \quad \text{and} \quad R_d = f_{y,d} = \frac{f_{y,k}}{\gamma_M}$$

the general design equation:

$\sigma_{y,d} \leq f_{y,d}$ is obtained.

Where:

$f_{y,k}$ the characteristic yield stress according to Appendix A 8;
γ_M the partial safety factor according to Appendix A 8;
M_d the design moment;
$W_{y,n}$ the net resistance moment.

2. The dead-load of the excavation structure may be disregarded when designing sheet piling. Analysis of normal stresses is sufficient for sheet piles with interlocks inside the flanges, due to the relative thickness of the webs, unless the major part of loads acting on retaining walls results from water pressure. In this case the shear stresses may also be decisive for the bearing capacity of the section.

3. If no vertical forces other than the dead-load of the retaining structure and the vertical earth pressure component need to be transmitted, a bearing capacity analysis according to DIN 18 800-1:1990-11, Section 7.5.2 will suffice. If other vertical forces act in addition to those previously discussed, e.g. resulting from excavation covers, provisional bridges or inclined anchors, a stability analysis according to DIN 18 800-2 shall be undertaken, in particular for single-propped retaining walls and for the individual retreating states of multiple-propped retaining walls.

4. A zero-line shear force transmission analysis shall be performed for sheet pile walls consisting of U sections if:

   a) the sheet pile wall is located in open water or if a significant portion of it is driven through peat, tidal mud deposits, mud or soils with high clay content;
   b) grease or a sealant are applied to lubricate against interlock friction prior to the driving process, or if interlocks are appropriately protected against penetration of soil particles or;
   c) connecting elements between individual sections exceed the tolerances stated in the EAU, Recommendation R 67 [2].

   Continuous wall section properties may be taken fully into consideration even if shear force transmission is only projected for every second interlock.

5. The characteristic material parameters and the partial safety factors are given by DIN 18 800-1, see Appendix A 8. In standard cases the elastic-elastic method shall be adopted. See DIN 18 800-1:1990-11, Sections 7.3 to 7.5 and R 27 (Section 4.5) for design using the elastic-plastic and plastic-plastic methods.

   See Appendix A 8 for the allocations for sheet pile wall steel. Sheet piling made of other steel types to those given in Appendix A 8 may be used if they have been given general technical approval.

6. Following the introduction of EN 1993-5 as building regulations the safety against structural failure of sheet piles shall be analysed according to the stipulations of this standard and the corresponding National Annex.

   The analysis format corresponds to the limit state condition discussed in Paragraph 1. The principal differences are with regard to determination of the section bearing capacity of the sheet piles. The following shall be observed:

   a) The characteristic material parameters and the partial safety factors are taken from DIN 18 800-1, see Appendix A 8.

   b) The sheet pile sections are divided into 4 classes corresponding to the ratio of flange width b to flange thickness t_f as a parameter for the full support of the flange under compressive stresses for the bearing capacity analysis. These classes determine the applicability of the various analysis methods for determining the action effects and the methods for analysing the resistances, i.e.:

      - Class 1 – cross-sections:
        Plastic–plastic method
        Plastic analysis and design are allowable. However, rotational analysis shall be performed in addition to adhering to the b : t_f limit.

      - Class 2 – cross-sections:
        Elastic–plastic method
        Elastic analysis is necessary. Utilisation of the plastic cross-section values is allowable.

      - Class 3 – cross-sections:
        Elastic–elastic method
        Elastic analysis is necessary. Only the elastic cross-section values may be adopted.

      - Class 4 – cross-sections:
        Elastic–elastic method 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.

   c) If necessary, when determining the flexural stiffness and the elastic and plastic moment resistances 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 transmission of shear forces to the sheet pile wall interlocks.

red $I_y = \beta_D \cdot I_y$ red $W_{el,y,n} = \beta_B \cdot W_{el,y,n}$ or red $W_{pl,y,n} = \beta_B \cdot W_{pl,y,n}$

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.

d) The impact of normal forces, shear forces and, if applicable, lateral bending due to high positive water pressures and transmission of concentrated loads, e.g. from anchors, shall be taken into consideration for determination of the bending limit bearing capacity. See EN 1993-5, Sections 5.2.2 to 5.2.4 and 7.4.3.

e) A stability analysis (bulging) is only required if the acting normal force is greater than 4% of the decisive normal force. See EN 1993-5, Section 5.2.3 (4).

7. See R 85 (Section 13.10) for analysis of the external bearing capacity, i.e. transmission 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:

   $E_d \leq R_{M,d}$

   shall be analysed for the design action effects determined according to Section 6. The design value E_d consists of the loads from the most unfavourable combination of action effects, the design value $R_{M,d}$ 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: $\sigma_{c,d} \leq f_{c,d}$ where $f_{c,d} = \alpha \dfrac{f_{c,k}}{\gamma_c}$ and γ_c according to Appendix A 7

   b) Reinforcing steel: $\sigma_{s,d} \leq f_{s,d}$ where $f_{s,d} = \dfrac{f_{y,k}}{\gamma_s}$ and γ_s according to Appendix A 7

   Where:

   $\sigma_{c,d}$ design value of the concrete compressive stress of the actions;
   $f_{c,d}$ design value of the concrete compressive strength;
   $\sigma_{s,d}$ design value of the reinforcing steel stress of the actions;
   $f_{s,d}$ design value of the yield stress of the reinforcing steel;

$f_{c,k}$ characteristic value of the concrete compressive strength according to Appendix A 7;

$f_{y,k}$ characteristic value of the yield stress of the reinforcing steel according to Appendix A 7;

α reduction factor according to DIN 1045-1 ($\alpha = 0.85$ for normal strength concrete).

3. DIN 1045-1 applies for the design and construction of cast in-situ concrete walls. With regard to reinforcement positioning and concrete cover, the requirements of ENV 1538 shall be met for diaphragm walls and ENV 1536 for pile walls.

4. Beside reducing the computed maximum support moment according to R 11, Paragraph 5 (Section 4.2), the moment diagram may be smoothed out at each point of support if concealed beams or reinforced concrete waling are installed. For rolled section walings, the full flange width may be considered as support only if web stiffeners are designed to sufficiently prevent flange deflection and if 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 friction bonded using dowels. Diaphragm wall elements consisting of several reinforcement cages in a single element length, and which are jointlessly concreted in a single operation, are regarded as friction bonded.

   Sufficient bonding may be achieved by suitable profiling of the joints, for example, for separately manufactured diaphragm wall elements.

6. When analysing anchoring lengths, the bonding properties according to DIN 1045-1:2001-07, Section 12.4 of the horizontal rebars are always classified as moderate, those of the vertical rebars as good.

7. Generally, an analysis for restricting the crack width in in-situ concrete walls is not required if the necessary minimum reinforcement according to DIN 1045-1:2001-07, Section 13 is adhered to during construction. An analysis is necessary if:

   a) the ambient conditions for exposure class XA 3 according to DIN 1045-1:2001-07, Table 3 need to be taken into consideration;
   b) the ambient conditions for exposure class XS and XA according to DIN 1045-1:2001-07, Table 3 need to be taken into consideration and the construction stage decisive for reinforcement is projected 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 by DIN 1045-1, see Appendix A 7.

9. See R 85 (Section 13.10) for analysis of external bearing capacity, i.e. transmission 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:

   $E_d \leq R_{M,d}$

   shall be analysed for the design action effects determined according to Section 5 and Section 6. The design value E_d consists of the loads from the most unfavourable combination of action effects, the design value $R_{M,d}$ of the sum of the resistances of the structural elements. For example, in the simplest case of a double-symmetry steel waling beam with uniaxial bending an elastic-elastic analysis of the normal stresses:

   from $E_d = \sigma_{y,d} = \dfrac{M_d}{W_{y,n}}$ and $R_d = f_{y,d} = \dfrac{f_{y,k}}{\gamma_M}$

   the general design equation:

   $\sigma_{y,d} \leq f_{y,d}$ is obtained.

   Where:

   $f_{y,k}$ the characteristic yield stress according to Appendix A 8;
   γ_M the partial safety factor according to Appendix A 8;
   M_d the design moment;
   $W_{y,n}$ the net resistance moment.

2. If waling subject to bending stresses is utilised for transmission of axial forces, it shall be analysed for both bending according to DIN 18 800-1 and buckling according to DIN 18 800-2. Only deflections on the excavation side need be taken into consideration when determining the buckling length.

3. Shear stresses and effective stresses shall always be analysed for steel section waling subject to bending.

4. If a cantilever effect is considered when determining bending moments, the impact of unintentional displacement of load transmission or support points shall be assessed.

5. If web stiffeners are welded on at load transmission or support 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 friction fit steel plate or timber web stiffeners, provided they are installed with reasonable care and accuracy.

6. If no more precise analysis is performed bracing elements are required at the load transmission and support points and, if applicable, at intermediate points, for waling consisting of single sheet piling (U-sections) in order to retain shape stability.

7. Walings designed to prevent collapse of the excavation structure only for a limited period following complete failure of an anchor or strut may be designed for Load Case 3 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 and arching effects into consideration and, in contrast to Appendix A 8, fully utilising the yield stress of the steel.

8. 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 along its length. This is also the case for unsupported soldier pile walls only restrained in the ground. 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 friction bonded to them by welding or bolting. For excavation depths up to 5 m a tie-beam of 5 cm^2 cross-section will generally suffice; in addition, a minimum cross-section of 10 cm^2 should be adopted.

9. The characteristic material parameters and partial safety factors adopted for steel section walings are given by DIN 18 800-1, see Appendix A 8. In standard cases the elastic-elastic method shall be adopted. See R 11, Paragraph 4 (Section 4.2) and R 27 (Section 4.5) for elastic-plastic and plastic-plastic design.

10. For reinforced concrete walings the characteristic material parameters and partial safety factors are given by DIN 1045-1, 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:

   $E_d \leq R_{M,d}$

   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 $R_{M,d}$ 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 an excavation structure. 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 transmission 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). For rolled section struts bulging and, if applicable, lateral torsional buckling shall be investigated according to DIN 18 800-2.

4. If no specific force transmission eccentricity is defined and ensured by appropriate procedures, the DIN 18 800-2 stability analysis carried out for steel struts shall 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 due to dead-load and live loads.

5. If the buckling length of struts is to be reduced, the walings 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 must be prevented 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:

   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 assumed 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, bulging) shall not be restricted to the individual supporting elements of the bracing, but shall also address the spatial relationships of the individual components according to DIN 18 800-1 and DIN 18 800-2.

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 Load Case LC 1 according to DIN 1054 shall be adopted for determination of the design action effects or the design action effects determined for a different case increased by 15%.

13. The characteristic material parameters and the partial safety factors are given by:

    - DIN 1045-1 for struts or stiffening concrete or reinforced concrete base slabs, see Appendix A 7;
    - DIN 18 800-1 and DIN 18 800-2 for steel struts, see Appendix A 8;
    - DIN 1052 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*"), issued 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 possible hazardous conditions when manufacturing the excavation and for its later use, e.g. from crane operation or material transport, shall be taken into consideration. It may be necessary to implement special protective measures, e.g. deflectors or covers.

16. In a special case, if the safety of the retaining structure should be verified assuming a failure of one of the struts, the reserve of the bearing capacity of the structure and the soil, e.g. the arching effect, can be explored and the

partial safety factors for the structural element resistances can be adopted. Also see R 51 (Section 13.6).

13.8 Bearing capacity of trench sheeting and bracing (R 53)

1. The safety against structural failure of trench sheeting and bracing according to the limit state condition:

   $E_d \leq R_{M,d}$

   shall be analysed for the design action effects determined according to Section 5 and Section 6. The design value E_d consists of the loads from the most unfavourable combination of action effects, the design value $R_{M,d}$ 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 either R 47 (Section 13.2) or DIN 4124:2002-10, Section 10.3.3.
   b) R 47 (Section 13.2) applies for timber design.
   c) R 51 (Section 13.6) applies accordingly for the design of soldier beams for horizontal sheeting.

3. The following points apply for vertical lining:

   a) For vertical trench sheeting, R 49 (Section 13.4) applies for the design of trench sheet piles, driven sheet plates, curtain sections or lightweight sheet pile walls accordingly.
   b) R 51 (Section 13.6) applies for the design of steel section walings.
   c) Timber walings 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 13 331-1 and EN 13 331-2 shall be observed for the design of trench sheeting equipment.

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:

   $E_d \leq R_{M,d}$

shall be analysed for the design action effects determined according to Section 5 and Section 6. The design value E_d consists of the loads from the most unfavourable combination of action effects, the design value $R_{M,d}$ of the resistance of the structural element. The partial safety factors for Load Case 2 given in Appendix A 6 apply for the determination of 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 DIN 1055-4, the characteristic values of snow loads according to DIN 1055-5 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 by:

   – DIN 1045-1 for concrete or reinforced concrete structural elements, see Appendix A 7;
   – DIN 18 800-1 and DIN 18 800-2 for steel structural elements, see Appendix A 8;
   – DIN 1052 for timber structural elements, see Appendix A 9;

   unless, as in the case of rail traffic, for example, the regulations of the respective transport company 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 Affairs (*Bundesministerium für Verkehr, Bau- und Stadtplanung (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 regard to any anticipated noise impacts caused by pavement detachment.

5. For analysis of the serviceability limit state according to R 78, Paragraph 6 (Section 1.4) it may be necessary to limit the bending of 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) Maximum permissible speed, potential hazards to the pavement, driving comfort, or impact on vehicles for provisional bridges and excavation covers designed for road and rail traffic.
   b) Strength and deformation behaviour of pipes and sleeves for pipe bridges involving rigid pipes, if deflection cannot be compensated for by appropriate structural design.
   c) The amount of water drainage required for prevention of ponding for protective covers.

   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 cast in-situ concrete walls (R 85)

1. For analysis of the vertical bearing capacity as demanded by R 84 (Section 4.9), determination of the characteristic resistances between the retaining wall elements and the ground in the ultimate limit state is required; here, this is called the "external" bearing capacity.

2. The characteristic resistances should 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) DIN 1054:2005-01, Section 8.4.4 and Annex B thereof for cast in-situ concrete walls and soldier piles set in boreholes and concreted at the toe.
b) The data in Appendix A 10 for driven sheet pile walls and soldier piles.

3. The following points apply for determination of the characteristic toe resistances according to R 84, Paragraph 2 c) (Section 4.9):

a) The actual toe and contact areas shall be adopted for cast in-situ concrete walls and concreted soldier piles according to Paragraph 2 a).

b) The effective toe or contact area for sheet pile walls is obtained as a function of the opening angle α as shown in Figure R 85-1 according to [135] using:

$A_b = \chi \cdot h \quad \text{in } m^2/m$

The reduction factor χ according to [52] may be taken from the following table as a function of the opening angle α:

α	90°	80°	70°	60°	50°	40°	30°
χ	1.00	0.85	0.70	0.55	0.40	0.25	0.10

c) The full girder cross-section may be adopted for driven soldier piles as shown in Figure R 85-2 a).

The base resistance data according to Paragraph 2 refer to conditions below the groundwater table. If this is below the influence depth of the base resistance then $q_{b1,k}$ may be increased as an approximation using the calibration factor:

$$\eta_\gamma = \frac{\gamma}{\gamma'}.$$

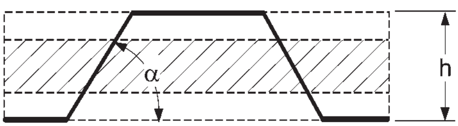

Figure R 85-1. Effective contact area and skin area of driven sheet pile walls

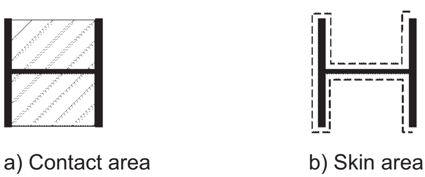

Figure R 85-2. Effective contact area and skin area of driven soldier piles

4. The following points apply for determination of the characteristic skin friction according to R 84, Paragraph 2 d) (Section 4.9):

   a) The actual toe and contact areas below the excavation level shall be adopted for cast 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 according to Paragraph 2 c).

   Generally, the skin area may be adopted for transferring 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 earth pressure angle is adopted at $\delta_a = 0$ or with a negative sign;
   – if the wall is taken deeper than numerically necessary, however, only in the region of the additional embedment depth.

5. An embedment depth of 1.5 m is generally sufficient without further analysis for excavations up to 10 m deep and favourable ground conditions, if only the dead-weight loads of the wall and the vertical earth pressure component are transmitted to the subsurface. Otherwise, the following apply:

   a) Transmission of vertical forces shall always be analysed if:

      – the excavation is deeper than 10 m;
      – there is no sufficiently load-bearing soil according to Appendix 10, Paragraph b) 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 cast in-situ concrete walls and concreted soldier piles;

      are not permissible for transmitting additional loads, beside the dead-load of the excavation structure and the vertical component of the earth pressure.

6. If the 3.00 m and 2.50 m embedment depths stipulated in Paragraph 5 are not adhered to when analysing transmission of vertical forces from dead-weight and earth pressure, in particular for an embedment 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 cast in-situ concrete walls and concreted soldier piles.

7. See R 84, Paragraph 6 (Section 4.9) for determination of the design values R_d from the characteristic resistances R_k.

13.11 Bearing capacity of tension piles and ground anchors (R 86)

1. Tension piles are deployed in excavation structures 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 excavation structures to anchor retaining walls according to Chapter 7 and to anchor excavation bases in water according to R 62 (Section 10.5).
2. Sufficient failure safety is given if the limit state condition:

   $E_{1,d} \leq R_{1,d}$

   is fulfilled, i.e. if the sum $E_{1,d}$ of the action design values is no greater than the design value $R_{1,d}$ of the axial tension pile resistance or the anchor resistance at its greatest. The design forces of the 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 ultimate limit state with the partial safety factors according to Table 6.1 given in Appendix A 6.
3. See DIN 1054 for determination of the characteristic tension pile resistances $R_{1,k}$. DIN 1054 is also decisive for determination of the decisive anchor resistance and the necessary anchors tests.
4. The use of structural measures shall ensure that the failure of a tension pile cannot lead to the failure of the structural element secured by the tension pile. If a stability analysis is performed in this case, it may take all the reserves inherent in the supporting structure and the ground into consideration.
5. The implementation of structural measures or numerical analyses of the possible failure of a ground anchor may be dispensed with if the following conditions are fulfilled:
   a) Each anchor (temporary anchors) is tested to 1.5x instead of 1.25x the characteristic anchor force during the acceptance test. The inner bearing capacity of the tendon shall be analysed for the tensioning process.
   b) Strand anchors with at least 4 strands are used as tendons.
   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.
   d) The anchors are locked-off at least at 100% of the active earth pressure and prevalent water pressure.

14 Measurements and monitoring of excavation structures

14.1 Purpose of measurements and monitoring (R 31)

1. The following points present the objectives of measurements and monitoring on excavation structures:

   a) To check the design parameters affecting the structure and the results of the structural analyses, see Paragraph 2.
   b) To examine the effects of design changes and deviations from the design during construction, see Paragraph 3.
   c) To optimise the design and the construction programme, see Paragraph 4.
   d) To apply the observational method according to DIN 1054, see Paragraph 5.
   e) To demonstrate technically faultless planning and implementation, see Paragraph 6.

2. Examination of the planned behaviour initially concerns the planning fundamentals directly involved in the structural analysis. For excavation structures these are principally:

   – the assumed ground characteristics, primarily determined by the stratigraphy and associated soil properties;
   – the groundwater levels;
   – the loads from neighbouring buildings, traffic or other actions.

   Examination of the analysis results achieved using these planning fundamentals generally consists of the following points:

   – adopted loads, i.e. the magnitude and distribution of the earth and water pressures;
   – the calculated deflections for the selected excavation structure;
   – the projected forces in anchors or struts.

3. The effects of planning changes or of deviations to plans in the course of construction can be tracked using measurements and be assessed based on measurement results. For example, deviations from planning can impact:

   – construction progress;
   – duration of the construction project;
   – embedment of the retaining wall;
   – loads on the retaining wall;
   – anchor bearing capacity.

4. Some examples of optimisation options, which can be implemented in the course of construction based on measurement results, are given below:

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– increasing the spacing of the bearing members of a soldier pile wall in linear excavations;
– reducing the number of excavation phases;
– reducing the extent of dewatering.

5. Measurements carried out when implementing the observational method according to DIN 1054 represent components of the stability and serviceability analyses. They should therefore be closely integrated in the geotechnical investigations and numerical projections. Also see R 32, Paragraph 4 (Section 14.2).

   The observational method according to DIN 1054 may generally only be adopted as a component of the stability analysis of excavation structures if the following conditions are fulfilled:

   a) Failure of the structure shall be recognisable by suitable measurements or make itself noticeable at such an early stage that structural countermeasures can be implemented in time.
   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 within the scope of the structural drawings, allowing them to be implemented immediately if required.

6. Measurements serve as a means of quality control and thus to demonstrate technically faultless planning and implementation. In this context, they may also represent part of documented evidence against third parties, e.g. authorities or neighbours.

14.2 Preparation, implementation and evaluation of measurements (R 32)

1. The following fundamental procedure is recommended for planning measurements on excavation structures:

   – identification of possible hazard scenarios and a risk estimate;
   – definition of critical areas and the type and extent of measurements;
   – documentation stipulations and forwarding of measurement results;
   – stipulation of threshold, action and alarm values, and;
   – stipulation of the type and extent of measured data interpretation.

2. Threshold, action and alarm values can be defined as follows:

   a) The threshold values are reached if the measured data fall below a previously defined margin to the action values. An appropriately enhanced level of attention in terms of the behaviour of the structure or structural element is associated with this. Stipulation of the threshold values can only project-specific.

b) The action values define the measured data boundary above which additional measures are required. Close coordination between the parties involved in planning and in construction is required to enable immediate implementation of these measures.
c) Reaching the alarm values indicates abnormal loads on the excavation structure or the surrounding ground, generally posing a threat to stability. Stabilising measures for the protection of persons and property shall be initiated immediately.

3. Possible hazard scenarios and risks involved with excavation structures can affect both stability and serviceability. A number of examples are given below:

a) The occurrence of large deflections in a retaining wall, which can lead to unacceptable displacements in neighbouring structures or an unacceptable reduction in the planned clear dimensions of the excavation.
b) Loss of anchor bearing capacity, failure of the anchored ground or deep-level sliding failure, thus endangered stability of the retaining wall.
c) Risks, avoidable or unavoidable, occurring during the manufacturing process of the excavation structure, e.g. decompaction of the ground when manufacturing encased bored piles in the sand below the groundwater table, settlement of neighbouring buildings due to the installation of ground anchors or the construction of diaphragm walls.
d) Occurrence of hydraulic heave or increased erosion due to percolation around a retaining wall, leading to a reduction in, or complete loss of, the passive earth pressure.

4. The locations of the critical areas and the type and extent of measurements should be defined jointly by the parties to the project, the responsible authorities and, if applicable, affected third parties. The following criteria should taken into special consideration:

– potential hazard to public safety and order;
– potential hazard to third party assets;
– time and financial expense for rectification of any damage or the reinstatement of a planned condition;
– type and size, as well as the sensitivity, of neighbouring structures;
– knowledge of the ground and groundwater conditions;
– duration of the construction project.

5. When planning a measurement programme the following points in particular should be stipulated:

– measured variables, e.g. displacements, pressures and forces, see R 33 (Section 14.3);
– measurement methods or measuring systems, see R 34 (Section 14.4);
– measurement locations, such as the location of measuring sections, see R 35 (Section 14.5);

- time that measurements begin and end, see R 36 (Section 14.6);
- measuring interval, e.g. for measurements at discrete times;
- action and alarm values;
- forwarding of measurement results, see R 37 (Section 14.7);
- guidelines for action if alarm values are reached;
- type and extent of measurement result interpretation;
- assimilation of measurements into the construction programme.

6. If the observational method according to DIN 1054 is agreed upon for an excavation structure, a measurement programme shall be compiled in close cooperation with the structural engineer and the geotechnical planner, allowing the system behaviour to be examined for compliance with the defined boundaries, based on meaningful measured variables. The measuring intervals, and the duration between measurements and result analysis, shall be selected as a function of construction progress and possible developments in the behaviour of the structure in such a way that any necessary countermeasures can be implemented in time, see R 31, Paragraph 5 (Section 14.1). The installation of online-measuring systems with integrated alarm function if defined boundaries are exceeded is recommended for this.

14.3 Measured variables (R 33)

1. Measurements generally involve determination of the following measured variables:
   - lengths;
   - displacements
     (e.g. bending, settlement, heave, horizontal displacement);
   - positive and negative strains;
   - torsions;
   - forces;
   - stresses and pressures;
   - water levels and porewater pressures;
   - vibration velocities and accelerations;
   - times;
   - temperatures.
2. The selected measured variables should provide the following information, for example, depending on the prevalent local conditions:
   - orientation of the retaining wall and its supporting elements after manufacture;
   - wall and ground displacements, e.g. as a result of dewatering, excavation, manufacturing the structure, backfilling;
   - earth and water pressures;
   - anchor or strut forces;
   - displacements and deformations in neighbouring structures.

3. The influence of temperature shall be taken into consideration when evaluating the measured variables or be eliminated beforehand:
   a) The impact of temperature on the structural element being measured, e.g. a steel strut, shall be determined by parallel temperature measurements on the structural element.
   b) The impact of temperature on the measuring system, e.g. the hydraulic pressure in a pressure cell, in contrast, should be compensated for within the system.

14.4 Measurement methods and measurements systems (R 34)

1. The measurement methods are principally differentiated into discrete or continuous measurements. In many cases discrete measurements can be carried out manually. Continuous measurements require automatic data collection and forwarding. Generally, robust, reliable measurement methods should be selected, which fulfil the minimum demands placed on measurement precision.
2. Retaining wall displacements can be determined geodetically using analogue or digital levelling instruments or theodolites. Motorised instruments with automatic target recognition allow automatic measurements which are continuously evaluated by a central measurement value logging system.
3. Inclinations and horizontal displacements of a retaining wall can be determined using inclinometers installed in a borehole or in the retaining wall. In addition, methods based on laser scanning are available, allowing both linear and grid-based logging of displacements.
4. On the one hand, displacements in the neighbouring subsoil or neighbouring structures can be manually logged, 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, hose levelling systems are available in particular for long-term monitoring of buildings adjacent to excavations.
5. In general, vibrating wire and glass-fibre system strain gauges and strain sensors are available for strain measurements, e.g. on steel girders or reinforcing steel. The temperature-induced strains in the structural elements shall be taken into consideration when deriving the loads from earth and water pressures. Automatic correction of the temperature-induced component of the measured values is recommended.
6. Electrical or hydraulic load cells with electrical pressure converters are in common use for the measurement of forces, e.g. strut or anchor forces. These allow remote measurements to be simply carried out.
7. In special cases, direct determination of the earth pressure acting on retaining walls may be expedient. Electrical or hydraulic earth pressure cells may be

used for measuring earth pressures. They measure total stresses in the ground, the sum of earth and water pressures. These cells present a problem inasmuch as installation is not possible without affecting the stress condition in the ground.

8. Conventional gauges are primarily available as open standpipes for measuring the groundwater table and the head in permeable soils. The piezometric heads are determined using electric contact gauges or pressure transducers. 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 measurement value logging with continuous measurement results shall be aimed for.

9. Using combined earth pressure and porewater pressure transducers, it is possible to determine the effective stresses in the ground from the difference between the total stresses, i.e. earth and porewater pressures, based on the principle of effective stresses.

10. Profile measuring devices can be employed to check the dimensional stability and verticality of boreholes and open trenches.

11. Instruments for measuring seismic acceleration are available to record the dynamic loading of neighbouring structures if vibrations are possible when installing the retaining walls, e.g. when driving sheet pile walls.

12. Geophysical measuring methods serve to explore heterogeneities in the subsoil or the structural element, e.g. localisation of obstructions, voids or leaks.

Redundancy should be aimed for regardless of the selected measurement, i.e. it should be possible to monitor a measured variable by measuring with a different measuring system.

14.5 Location of measurement points (R 35)

1. Location of measurement points can generally follow the criteria given below:

   a) The measurement results obtained should be transferable to as large a region of the retaining wall as possible. The transferability is primarily in relation to the load-bearing structure, the selected construction method, subsoil and groundwater conditions, actions and, if applicable, neighbouring structures.
   b) If logging of the linear retaining wall displacements is aimed for, the excavation length for which this section is representative should be defined before measurements commence. This governs the distance to further measurement sections.

c) The extent of the measurement sections perpendicular to the retaining walls shall be selected according to the anticipated influence of the excavation structure in the zones in front and behind the retaining wall and any possible impairment or hazard to neighbouring buildings.

2. If measurements serve to verify the numerical projections, the configuration of the measurement points shall be adapted to the type of analysis and the design criteria selected in the analyses. For example, if the head displacements are adopted for dimensioning a retaining wall, these should primarily be measured during construction.

3. Sufficient options for verification of the measurement results should be planned for during location of the measurement points. This means, for example, that the location of two measurement sections in one and the same representative area should be given preference over a great number of areas with single measurement sections. The objective should always be to obtain reliable data on the fundamental soil-structure behaviour.

14.6 Measurement times (R 36)

1. Measurements should generally be made at the following times:

   – 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.

   Calibration and zero measurements may be required before installing the measuring instruments for some measurement methods. These should be repeated after measurements are complete in order to recognise any changes to the measurement device during measuring and to compensate for them during evaluation.

2. Further measurements depend on the time behaviour of the material of both the structural element itself, e.g. the creep of an anchor, and of the subsoil, e.g. given a change in porewater pressure. Changes in groundwater conditions with time shall also be taken into consideration when stipulating measuring intervals.

3. Where possible, the time of the measurements should be selected such that the external conditions, e.g. temperature or tide levels, are similar for each respective measurement. Logging the course of the temperature of the structural element by supplementary temperature measurements should be aimed for and the influence of temperature on the measured variable taken into consideration in any assessment.

14.7 Transfer and processing of measurement results (R 37)

1. Once the type and extent of measurements are determined, the data format and the persons to whom they shall be forwarded shall be stipulated, as well as who shall interpret them. In order to give the project participants sufficient time to react, if necessary, the time between logging and forwarding of the measured data shall be kept to the minimum possible. If measurements are made automatically, the project participants can access the measurement data using online networks if appropriate data logging systems are implemented.
2. In order to allow rapid reactions to abnormal or critical changes in the projected conditions, threshold values, action values and alarm values shall be defined. If these values are reached, previously defined action and work guidelines shall be followed (see Section 14.2). For automatic data logging, alarms can implemented within the logging software in case one of the defined values is reached. Alarms can then be issued automatically, e.g. by way of telephone notifications.
3. Graphical visualisations are indispensable for assessing the measurement results, especially for large quantities of data. In most cases automatic data logging systems provide suitable options in the measuring and evaluation software.
4. Measurement reports containing all information pertaining to the measurements shall be created at regular intervals. Once measurements are complete the data and measurement reports shall be summarised and documented in their entirety. These documents shall be treated as record documents in a similar manner to execution plans or structural analyses.

Appendix

A 1: Relative density of cohesionless soils (10/05)

According to DIN 1054 "Verification of the Safety of Earthworks and Foundations" and DIN 1055, Part 2 "Actions on Structures – Part 2: Soil Properties – Unit Weight, Friction Angle, Cohesion".

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	$0.15 \leq D < 0.30$	$0.00 \leq D < 0.45$	$5.0 \leq q_c < 7.5$
Medium-dense	$0.30 \leq D < 0.50$	$0.20 \leq D < 0.65$	$7.5 \leq q_c < 0.15$
Dense	$0.50 \leq D < 0.75$	$0.65 \leq D < 0.90$	$15 \leq q_c < 25$
Very dense	$0.75 \leq D$	$0.90 \leq D$	$q_c \geq 25$

Table 1.2. Criteria for medium-dense compaction

Soil class according to DIN 18 196	Coefficient of uniformity	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 according to DIN 18 196	Coefficient of uniformity	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	$U > 3$	$D \geq 0.65$	$D_{Pr} \geq 100\%$	$q_s \geq 15\ MN/m^2$

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A 2: Consistency of cohesive soils (10/05)

Definitions

The consistency depends on the water content w (see DIN 18 121, Part 1). With decreasing water content, cohesive soil changes its state from liquid to plastic to nearly hard to firm (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 the liquid to the plastic state.
b) The plastic limit w_P is the water content at the transition from the plastic to the nearly hard state.
c) The shrinkage limit w_S is the water content at the transition from the nearly hard to the firm (hard) state.
d) The plasticity index I_P is the difference between liquid and plastic limit:

   $I_P = w_L - w_P$.
e) The range between the liquid and the plastic limit is sub-categorised into very soft, soft and stiff.

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;
b) $I_C = 0.50$ to 0.75: soft;
c) $I_C = 0.75$ to 1.00: stiff.

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 **stiff**.
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 **nearly hard**.
e) A soil that has dried out and generally appears light-coloured is **firm** (hard). This soil can no longer be kneaded but only broken apart. Subsequent balling of individual pieces is not possible.

A 3: Properties of cohesionless soils (10/05)

Table 3.1. Empirical values for the unit weight of cohesionless soils

Soil type	Abbreviation to DIN 18 196	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, widely or intermittently graded	GW, GI, SW, SI with 6 ≤ U ≤ 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 widely 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 soil, or for 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

Angle of friction			
Soil type	Abbreviation according to DIN 18 196	Compaction	Angle of friction φ'_k [°]
Gravel, sand, Uniformly, widely 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 die capillary cohesion $c_{c,k}$ are 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 (10/05)

Table 4.1. Empirical values for the unit weight of cohesive soils

Soil type	Abbreviation according to DIN 18 196	Consistency	Unit weight		
			Earth moist γ_k [kN/m^3]	Saturated $\gamma_{r,k}$ [kN/m^3]	Buoyant γ'_k [kN/m^3]
Silty soils					
Slightly plastic silts ($w_L < 35\%$)	UL	soft stiff nearly hard	17.5 18.5 19.5	19.0 20.0 21.0	9.0 10.0 11.0
Medium-plastic silts ($35\% \le w_L \le 50\%$)	UM	soft stiff nearly hard	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 stiff nearly hard	19.0 20.0 21.0	19.0 20.0 21.0	9.0 10.0 11.0
Medium-plastic clays ($35\% \le w_L \le 50\%$)	TM	soft stiff nearly hard	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 stiff nearly hard	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 stiff	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 18 196), the unit weights given in lines 1 to 9 shall be increased by 1.0 kN/m^3.

c) When analysing safety against buoyancy, 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 soil 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 18 196	Consistency	Shear strength		
			Earth moist	Cohesion	
			φ'_k [°]	c'_k [kN/m^2]	$c'_{u,k}$ [kN/m^2]
Silty soils					
Slightly plastic silts ($w_L < 35\%$)	UL	soft stiff nearly hard	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 stiff nearly hard	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 stiff nearly hard	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 stiff nearly hard	17.5–27.5	5–10 10–15 15–20	5–60 20–150 50–300
Highly plastic clays ($w_L > 50\%$)	TA	soft stiff nearly hard	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 for $c_{u,k}$ are given in the table as the shear strengths in the unconsolidated condition. The corresponding friction angles 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: Guide values for the modulus of subgrade reaction $k_{s,h}$ for wet soils

Table 5.1. Modulus of subgrade reaction for cohesionless soil as a function of the relative density

Degree of mobilisation	Relative density		
	Loose	Medium-dense	Dense
mob $E_{ph,k}$: $E_{ph,k}$ = 25%	15.0 MN/m^3	30.0 MN/m^3	60.0 MN/m^3
mob $E_{ph,k}$: $E_{ph,k}$ = 37.5%	3.0 MN/m^3	6.0 MN/m^3	12.0 MN/m^3
mob $E_{ph,k}$: $E_{ph,k}$ = 50%	1.2 MN/m^3	2.5 MN/m^3	5.0 MN/m^3
mob $E_{ph,k}$: $E_{ph,k}$ = 75%	0.5 MN/m^3	1.0 MN/m^3	2.0 MN/m^3

Table 5.2. Modulus of subgrade reaction for cohesive soil of stiff to nearly hard consistency

Degree of mobilisation	Modulus of subgrade reaction
mob $E_{ph,k}$: $E_{ph,k}$ = 25%	9.0 MN/m^3
mob $E_{ph,k}$: $E_{ph,k}$ = 37.5%	5.0 MN/m^3
mob $E_{ph,k}$: $E_{ph,k}$ = 50%	3.0 MN/m^3
mob $E_{ph,k}$: $E_{ph,k}$ = 75%	2.0 MN/m^3

A 6: Partial safety factors for geotechnical variables

Table 6.1. Partial safety factors for actions and loads

Action or load	Abbreviation	Load case			
		LC 1	LC 2	LC 2/3	LC 3
EQU: Loss of static equilibrium					
Favourable permanent actions	$\gamma_{G,stb}$	(0.95)	0.95	0.95	0.95
Unfavourable permanent actions	$\gamma_{G,dst}$	(1.05)	1.05	1.05	1.00
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
Changeable actions[1)]	$\gamma_{Q,dst}$	(1.50)	1.30	1.15	1.00
STR: Failure of structures and structural elements					
General permanent loads	γ_G	(1.35)	1.20	1.10	1.00
– Intermediate stage $E_0 : E_a = 0.25 : 0.75$	γ_G		1.18	1.09	1.00
– Intermediate stage $E_0 : E_a = 0.50 : 0.50$	γ_G		1.15	1.08	1.00
– Intermediate stage $E_0 : E_a = 0.75 : 0.25$	γ_G		1.13	1.06	1.00
Earth pressure at rest	γ_{EOg}	(1.20)	1.10	1.05	1.00
Changeable loads[1)]	γ_Q	(1.50)	1.30	1.15	1.00
GEO: Overall stability					
Permanent loads	γ_G	(1.00)	1.00	1.00	1.00
Changeable loads[1)]	γ_Q	(1.30)	1.20	1.10	1.00

[1)] If acting unfavourably

Table 6.2. Partial safety factors for geotechnical resistances in the STR limit state

Type of resistance	Abbre-viation	Load case			
		LC 1	LC 2	LC 2/3	LC 3
Ground resistances					
Passive earth pressure	γ_{Ep}	(1.40)	1.30	1.25	1.20
Sliding resistance	γ_{Gl}	(1.10)	1.10	1.10	1.10
Pile resistances					
Compressive pile capacity during load testing	γ_{Pc}	(1.20)	1.20	1.20	1.20
Tensile pile capacity during load testing	γ_{Pt}	(1.30)	1.30	1.30	1.30
Compressive and tensile pile resistance based on empirical values	γ_{P}	(1.40)	1.40	1.40	1.40
Ground anchor resistances					
Resistance of a steel tendon	γ_{m}	(1.15)	1.15	1.15	1.15
Pull-out resistance of the grouted body	γ_{A}	(1.10)	1.10	1.10	1.10

Table 6.3. Partial safety factors for geotechnical resistances in the GEO limit state

Type of resistance	Abbre-viation	Load case			
		LC 1	LC 2	LC 2/3	LC 3
Shear strength					
Friction coefficient tan φ' and tan φ_u [1)]	γ_{φ}, $\gamma_{\varphi u}$	(1.25)	1.15	1.13	1.10
Cohesion c′ of the drained soil	γ_{c}	(1.25)	1.15	1.13	1.10
shear strength c_u of the undrained soil	γ_{cu}	(1.25)	1.15	1.13	1.10
Pull-out resistances					
Soil and rock nails	γ_{N}	(1.40)	1.30	1.25	1.20
Tension anchor piles	γ_{Z}	(1.40)	1.30	1.25	1.20
Grouted body of ground anchors	γ_{A}	(1.10)	1.10	1.10	1.10

[1)] If $\varphi_u > 0$

A 7: Material properties and partial safety factors for concrete and reinforced concrete structural elements

Table 7.1. Characteristic material parameters for normal strength concrete

According to DIN 1045-1:2001-07, Table 9

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									
$f_{ck} = f_{ck,cyl}$ [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]	25,800	27,400	28,800	30,500	31,900	33,300	34,500	35,700	36,800

$f_{ck,cube}$ = characteristic compressive cube strength of concrete after 28 days
f_{ck} [$f_{ck,cyl}$] = characteristic compressive cylinder 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 parameters for reinforced concrete

According to DIN 1045-1:2001-07, Table 11

Designation	BSt 500 S (A)	BSt 500 M (A)	BSt 500 S (B)	BSt 500 M (B)	Quantile P [%]
Type of product	Reinforcing steel	Reinforced concrete mat	Reinforcing steel	Reinforced concrete mat	
Ductility	A = normal ductility		B = high ductility		
Yield stress f_{yk} [N/mm^2]	500				5
Ratio $(f_t/f_y)_k$ [1)] (f_t = tensile strength)	≥ 1.05		≥ 1.08		min. 10
Steel strain under highest load[1)] ε_{uk} [%]	≥ 2.5		≥ 5.0		10

[1)] Ductility parameters

Table 7.3. Partial safety factors

According to DIN 1045-1:2001-07, Table 2, supplemented according to R 24 and R 79

Action combination according to R 24	–	Typical	Special case	Exceptional case
Load case	LC 1	LC 2	LC 2/3	LC 3
γ_c for determining the bearing capacity of reinforced concrete	(1.50)	1.50	1.50	1.30
γ_c for determining the bearing capacity of unreinforced concrete	(1.80)	1.80	1.80	1.55
γ_s for determining the bearing capacity of reinforcing steel	(1.15)	1.15	1.15	1.00
γ_M for analysis of serviceability	(1.00)	1.00	1.00	1.00

A 8: Material properties and partial safety factors for steel structural elements

Table 8.1. Characteristic material parameters

In the sense of DIN 18 800-1:1990-11 and EAU 2004 or DIN EN 10 248-1, for product thicknesses < 40 mm.

<table>
<tr><th>Steel grade (old designation)</th><th>Designations for structural steel according to DIN EN 10 027</th><th>Yield stress $f_{y,k}$ [N/mm²]</th><th>Tensile strength $f_{u,k}$ [N/mm²]</th><th>Shear strength $\tau_{R,k} = \frac{f_{y,k}}{\sqrt{3}}$ [N/mm²]</th><th>Young's modulus E [N/mm²]</th><th>Modulus of rigidity G [N/mm²]</th></tr>
<tr><td>St 37-2</td><td>S235JR</td><td rowspan="3">240</td><td rowspan="2">360</td><td rowspan="3">139</td><td rowspan="5">210 000</td><td rowspan="5">81 000</td></tr>
<tr><td>St 37-3U</td><td>S235JO</td></tr>
<tr><td>StSp 37</td><td>S240GP</td><td>340</td></tr>
<tr><td>StSp 45</td><td>S270GP</td><td>270</td><td>410</td><td>156</td></tr>
<tr><td>St 52-3U
StSp S</td><td>S355JO
S355GP</td><td>360
355</td><td>510
480</td><td>208
205</td></tr>
</table>

Table 8.2. Partial safety factors

According to DIN 18 800-1:1990-11, supplemented according to R 24

Action combination according to R 24	–	Typical	Special case	Exceptional case
Load case	LC 1	LC 2	LC 2/3	LC 3
γ_m For analysis of bearing capacity				
a) To compute the resistances	(1.10)	1.10	1.10	1.10
b) 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

In according with Amendment A1 of February 1996 to DIN 18 800-1:1990-11 the stresses may be increased by 10% using the elastic–elastic analysis method, if analysis according to DIN 18 800-2 to 18 800-4:1990-11 is required and no use is made of the option for taking into consideration localised plastification of the cross-section according to Elements 749 and 750.

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

Extract from DIN 1052:2004-08 and EN 338 for structural timber with wood types spruce, pine, fir, larch, Douglas fir, Southern Pine, Western Hemlock and Yellow Cedar. The given values are based on the use of new or practically new timber.

Strength class Classification according to DIN 4074-1:2003-06		C 16 S 7 C16M	C 24 S 10 C24M	C 30 S 13	C35 C35M	C 40 C40M
Strength parameters in N/mm²						
Bending	$f_{m,k}$ 1)	16	24	30	35	40
Tension, parallel perpendicular	$f_{t,0,k}$ 1) $f_{t,90,k}$	10 0.4	14 0.4	18 0.4	21 0.4	24 0.4
Compression, parallel perpendicular	$f_{c,0,k}$ 1) $f_{c,90,,k}$	17 2.2	21 2.5	23 2.7	25 2.8	26 2.9
Shear and torsion	$f_{v,k}$			2.7		
Stiffness parameters in N/mm²						
Young's modulus: parallel perpendicular	 $E_{0,mean}$ 1) 2) $E_{90,mean}$ 2)	 8,000 270	 11,000 370	 12,000 400	 13,000 430	 14,000 470
Modulus of rigidity	G_{mean} 2)	500	690	750	810	880
Bulk density parameters in kg/m³						
Bulk density	ρ_k	310	350	380	400	420

1) For round softwood merely freed of bark and bast, values increased by 20% may be adopted for analysis for regions with an unweakened boundary zone.

2) Mean values; the following computed values apply for the characteristic stiffness parameters:
$E_{0.05} = {}^2/_3 \cdot E_{0,mean}$ $E_{90,0.5} = {}^2/_3 \cdot E_{90,mean}$ $G_{05} = {}^2/_3 \cdot G_{mean}$.

Table 9.2. Partial safety factors

According to DIN 1052:2004-08, Table 1, supplemented according to R 24

Action combination according to R 24	–	Typical	Special case	Exceptional case
Load case	LC 1	LC 2	LC 2/3	LC 3
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 and soldier piles

a) The following empirical values for the skin friction q_s and base resistance q_b of driven sheet pile walls and soldier piles in the STR limit state may be selected for the analysis according to R 84 (Section 4.9), based on [52]:

– skin friction: $q_{s1,k} = 60\ kN/m^2$
– base resistance: $q_{b1,k} = 600 + 120 \cdot t_w\ [kN/m^2]$

Where:

St$_w$ the effective embedment depth in m, with $t_w = t_g - 0.50$ m;
t_g the actual embedment depth in m.

b) Adoption of the given empirical values for skin friction and base resistance assume sufficiently load-bearing ground. Cohesionless soils shall have a relative density of $D < 0.40$ for $U < 3$ or a relative density of $D < 0.55$ for $U < 3$, respectively. The demanded relative density is achieved if the cone resistance below the excavation level is at least $q_{c,k} = 10\ MN/m^2$. A nearly hard texture is required for cohesive soils.

c) For particularly good load-bearing ground the empirical values given in Paragraph a) for base resistance and skin friction may be increased by 25%. In this case cohesive soils shall display a relative density of $D < 0.55$ for $U < 3$ or a relative density of $D < 0.65$ for $U < 3$. The demanded relative density is achieved if the cone resistance below the excavation level is at least $q_{c,k} = 15\ MN/m^2$. An approximately firm (hard) texture is required for cohesive soils.

d) In addition, adoption of the given empirical values assumes that the necessary plug formation occurs when penetrating the load-bearing strata. Otherwise, the following shall be observed:

– If the sheet piling or soldier piles are vibrated in the given empirical values for skin friction and base resistance shall be reduced to 75%.
– If the sheet piling or soldier 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 the author of the draft and the technical planner posses expertise and experience in the geotechnical field.

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Terms and notation

Geometrical variables

H	Excavation depth
H'	Distance between ground level and the end of 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
s	Horizontal displacement of the retaining wall
t	Actual 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	Theoretical embedment depth below the excavation level with full restraint after *Blum*
t'_1	Theoretical embedment depth below the excavation level with partial restraint 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 in the consolidated state
c_c	Capillary cohesion in cohesionless soil
c_u	Shear strength of the undrained soil
q_s	Skin friction in the limit state

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γ	Soil unit weight above water
γ'	Buoyant unit weight of soil
γ_r	Unit weight of saturated soil
μ	Correction factor for determining shear strength from vane shear tests
φ'	Friction angle of cohesionless or consolidated cohesive soil
φ'	Friction angle of the drained soil (effective friction angle)
φ_u	Friction angle of the undrained soil
equiv. φ_S	Equivalent friction angle for soft soils
$\varphi'_{Equiv.}$	Equivalent friction angle 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 in the serviceability limit state
E_V	Remaining earth pressure force below the excavation level
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 in the limit state
g	Index for soil weight density
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 a planar forced slip plane

Further loads, forces and action effects

A	Anchor or strut force
B	Resultant support force/ground reaction in the ground support
B_B	Resultant support force from the soil stresses in the ground support
C	Equivalent force after *Blum*
G	Dead load
H	Horizontal force
m	Bending moment
P	Point load
Q	Changeable load
Q	Reaction force in slip planes
V	Vertical force
p	Unbounded distributed load
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 support force)
σ_B	Soil 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
f_q	Multiplication factor for changeable loads
η	Calibration factor
η_{Ep}	Calibration factor for passive earth pressure
μ	Utilisation factor (ratio of design values of effect to resistance)
γ_F	Partial safety factor for actions
γ_R	Partial safety factor for resistances
γ_G	Partial safety factor for permanent actions
γ_Q	Partial factor for unfavourable variable actions
γ_{GQ}	Weighted partial safety factor for actions
γ_{E0g}	Partial factor for permanent actions from at-rest earth pressure
γ_{Ep}	Partial factor for passive earth pressure
γ_H	Partial factor for the action from seepage force
γ_c	Partial factor for cohesion
γ_{cu}	Partial factor for the shear strength (cohesion) of the undrained soil
γ_φ	Partial factor for the friction coefficient tan φ

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, where retaining wall support through strut or anchor forces, and ground reactions are being described.

The notations of various terms, especially those widely recognised, are also adopted as indexes.

Recommendations in numerical order

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