

Code of practice for

Earth retaining structures

ICS 93.020

Committees responsible for this British Standard

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Foreword

BS 8002 has been prepared under the direction of the Technical Sector Board for Building and Civil Engineering.

This code of practice is a complete revision of the Civil Engineering Code of Practice No. 2, which was issued by the Institution of Structural Engineers in 1951 on behalf of the Civil Engineering Codes of Practice Joint Committee.

A draft of this code of practice, was issued in 1988 for public comment and in 1992 a new committee reviewed and revised the text.

The main changes in the design of earth retaining structures in this code of practice are:

- a) the recognition that effective stress analysis is the main basis for the assessment of earth pressures with total (undrained) stress analysis being important for some walls during or immediately following construction;
- b) the need to take account of the effect of movement (or lack of it) upon the resulting earth pressures on the wall. The largest earth pressures which act on a retaining wall occur during working conditions. These earth pressures do not increase if the wall deforms sufficiently to approach failure conditions.

This code of practice takes into account that for small movements of a wall the shear strength developed in the soil is less than the maximum shear strength measured in a conventional triaxial test and furthermore that when large strains occur in the soil, the shear strength may reduce to the residual shear strength value.

It has been assumed in this code of practice that design of retaining walls is entrusted to chartered structural or chartered civil engineers who have sufficient knowledge of the principles and practice of soil mechanics as well as the principles and practice for the use of the appropriate structural materials, i.e. masonry, concrete, steel or timber.

This code of practice does not restrict designers from applying the results of research nor from taking advantage of special situations or previous experience in the design of retaining structures.

In this code of practice references have been made to non-BSI publications. The titles of these publications are given in Annex C.

The list of those engineers who have participated in the preparation of the initial draft, in the specially convened panel and in the more recently formed committee includes the majority of engineers who have a special interest in retaining walls. The Chairman throughout the long process of drafting, reviewing and complete redrafting, has been Mr. Thomas Akroyd, M.Sc. Tech, LL.B (Hons), C.Eng., a former President of the Institution of Structural Engineers.

BSI Committee B/526 whose constitution is shown in this British Standard, takes collective responsibility for its preparation under the authority of the Standards Board. The Committee wishes to acknowledge the personal contribution of:

Mr T N W Akroyd M.Sc. Tech, LL.B (Hons) (Chairman)

Dr M Bolton Ph.D., M.Sc., M.A. C.Eng., M.I.C.E.

Dr W G K Fleming B.Sc., Ph.D., C.Eng., M.I.C.E.

Dr B Simpson Ph.D., C.Eng., F.I.C.E.

Dr I F Symons Ph.D., C. Eng., F.I.C.E.¹⁾

Mr K W Vickery B.Sc., F.G.S.

Mr D Waite C.Eng., M.I.C.E., F.I. Struct.E.¹⁾

¹⁾ Deceased.

As a code of practice, this British Standard takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

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Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 134, an inside back cover and a back cover.

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Section 1. Introduction

1.1 Scope

The subject of this code of practice is the design and construction of structures to retain soils and materials with similar engineering properties, at slopes steeper than those which they would naturally assume. The code of practice provides guidance for a designer, conversant with theoretical and applied soil mechanics and experienced in structural design and construction. The code is applicable to walls with a retained height of up to about 15 m. Many of its recommendations are of general applicability. Specialist advice should be sought with regard to the design and construction of larger structures and for those where movement of the retained soils requires close control.

The code is divided into four sections.

Section 1 explains the terms used in the document and summarizes the factors influencing the choice of a retaining wall.

Section 2 describes the site and geotechnical data that is required together with material properties. It gives guidance on the determination of the values of representative soil strength necessary for design purposes.

Section 3 identifies the design philosophy and the design methods for earth retaining structures, including the determination of earth pressures and the analysis of overall stable equilibrium. It defines design soil strength and considers the loads on retaining walls and the forces available to attain equilibrium with tolerable displacements. Guidance is given on methods of simple practical design and on the influence of ground conditions.

Section 4 considers in detail various individual types of structure and application of earth pressure theory together with matters of construction and maintenance.

1.2 References

1.2.1 Normative references

This British Standard incorporates, by dated or undated reference, provisions from other publications. These normative references are made at the appropriate places in the text and the cited publications are listed on page 114. For dated references, only the edition cited applies; any subsequent amendments to or revisions of the cited publication apply to this British Standard only when incorporated in the reference by amendment or revision. For undated references, the latest edition of the cited publication applies, together with any amendments.

1.2.2 Informative references

This British Standard refers to other publications that provide information or guidance. Editions of these publications current at the time of issue of this standard are listed on the inside back cover, but reference should be made to the latest editions.

1.3 Definitions

For the purposes of this British Standard the following definitions apply and are limited to words used with special meaning in this document. Normal soil mechanics terminology is not defined.

1.3.1

active earth pressure

the earth pressure exerted on the retaining wall by the retained soil. It may be greater than the fully active earth pressure (see 1.3.11 and 3.1.9)

1.3.2

conservative values

values of soil parameters which are more adverse than the most likely values. They may be less (or greater) than the most likely values. They tend towards the limit of the credible range of values

1.3.3

design situation

a set of physical conditions for which it should be demonstrated that a limit state (see 1.3.13 and 3.2.2) will not occur

1.3.4**design soil strength**

soil strengths which are assumed will be mobilized at the occurrence of a limit state (see **1.3.13**). The design value of soil strength is the lower of either the peak soil strength reduced by a mobilization factor (see **1.3.14**) or the critical state strength

1.3.5**design surcharge load**

loading which is assumed to occur at some time during the life of the structure and for which the design should provide. See **3.2.2.2** and **3.3.4**

1.3.6**design value of a parameter**

the value of the parameter entered into equilibrium calculations

1.3.7**design value of wall friction**

the smaller of either the actual wall friction or adhesion measured by test or 75 % of design soil strength (see **1.3.4**). See **2.2.8** and **3.2.6**

1.3.8**disturbing force**

the force exerted by retained soil on a retaining wall, tending to cause the wall to move. It includes the surcharge loads, external loads and water pressure. The minimum value is the fully active earth pressure (see **1.3.11**)

1.3.9**earth pressure coefficients**

ratio of horizontal effective stress to vertical effective stress. K_a is the fully active earth pressure (see **1.3.11**) coefficient, K_p is the fully passive earth resistance (see **1.3.12**) coefficient. Both are based on the design soil strength (see **1.3.4**). Design values are determined from design values of soil parameters. Graphs are provided in Annex A for values of horizontal component of K_a and K_p . The values given in the various graphs in Annex A are for various ratios of φ' and wall friction δ

1.3.10**embedded walls**

formerly known as sheet pile walls, this term embraces walls of similar structural behaviour whether constructed of steel sheet piles, concrete piles, concrete diaphragms or timber. They are supported, at least in part, by passive earth resistance (see **1.3.15**)

1.3.11**fully active earth pressure**

the minimum value of the active earth pressure (see **1.3.1**), which occurs after sufficient movement or deflection of the retaining wall; the necessary movement is usually within the serviceability limit state (see **1.3.18**) of the wall

1.3.12**fully passive earth resistance**

the maximum value of the passive earth resistance (see **1.3.15**), which occurs after sufficient movement or deflection of the retaining wall. The necessary movement is often outside the serviceability limit state (see **1.3.18**) of the wall

1.3.13**limit state**

any state of stability beyond which the retaining wall no longer satisfies the design performance requirements. A limit state is not associated with any particular method of structural design. See ultimate limit state (**1.3.19**) and serviceability limit state (**1.3.18**)

1.3.14**mobilization factor**

a factor M of 1.2 or 1.5 (or more, see 3.2.4 and 3.2.5) applied to the representative soil shear strength to produce the design soil strength (see 1.3.4). M determines the proportion of the representative strength which may be mobilized at a limit state (see 1.3.13)

1.3.15**passive earth resistance**

the earth pressure generated by the soil when it resists movement of a retaining wall

1.3.16**rapid shearing**

in the context of total stress analysis, the shearing of a soil at a rate sufficient to prevent or inhibit any significant pore water pressure dissipation so that c_u is the operative shear strength

1.3.17**representative soil strength**

Conservative estimate of the mass strength of the soil. The value is determined from reliable site investigation and soil test data. In the absence of such data, see Table 1, Table 2, Table 3 and Table 4

1.3.18**serviceability limit state**

state of deformation of a retaining wall such that its use is affected, its durability is impaired, its maintenance requirements are substantially increased or damage is caused to non-structural elements. Alternatively such movement of the earth retaining structure which may affect adjacent structures or services in a like manner

1.3.19**ultimate limit state**

state of collapse, instability or forms of failure that may endanger property or people or cause major economic loss

1.3.20**unplanned excavation**

the minimum depth, below the nominal finished surface in front of the wall, which it is assumed, for design purposes, will be excavated at some time during the life of the retaining wall. See 3.2.2.2

1.4 Major symbols

c'	effective cohesion
c_b	base adhesion
c_u	undrained shear strength
c_w	undrained wall adhesion
D_{10}	effective grain size
D_{60}	effective grain size
E	Young's modulus
I	moment of inertia
j	flow-net parameter (see Figure 9)
K_a	fully active earth pressure coefficient
K_{ac}	active pressure coefficient for cohesion
K_i	ratio of horizontal to vertical effective stress for soil at rest (no strain)
K_o	coefficient of earth pressure at rest ($K_o \approx K_i$)
K_p	fully passive earth resistance coefficient
M	mobilization factor
N	result of standard penetration test
N'	modified value of N (see Figure 2)
N_c	bearing capacity factor
N_q	bearing capacity factor
N_y	bearing capacity factor
P_{an}	total active thrust normal to the wall
P_{pn}	total passive thrust normal to the wall
P_{WT}	pore water pressure
R	radius
q	surcharge pressure
u	water pressure
W	load
z	depth
z_w	depth to water table
α	inclination of the wall
β	inclination of the surface of the retained soil
γ	unit weight of soil (kN/m ³)
γ_w	unit weight of water
δ	angle of wall friction
δ_b	angle of base friction
σ_{an}	active pressure normal to the wall
σ_{pn}	passive pressure normal to the wall
σ_v	total vertical pressure
σ'_v	effective vertical pressure
φ'	effective angle of shearing resistance
φ'_{crit}	critical state angle of shearing resistance
φ'_{max}	maximum value of φ' determined from conventional triaxial test
φ'_r	residual friction angle
τ	base resistance

1.5 Selection and types of structure

1.5.1 General

There is a wide variety of different forms of earth retaining structure. Many structures include a combination of wall and support system.

1.5.2 Selection of type

The selection of a particular form of earth retaining structure will depend on:

- a) the location of the wall, its position relative to other structures and the amount of space available, including the necessity or otherwise to confine the support system within the site boundaries;
- b) the proposed height of the wall and the topography of the ground, both before and after construction;
- c) the ground conditions;
- d) the ground water and tidal conditions;
- e) the extent of ground movement acceptable during construction and in service and the effect of movement of the earth retaining structure on existing or supported structures and services;
- f) external live loading;
- g) the availability of materials;
- h) appearance;
- i) required life and maintenance;

Where several alternatives are suitable then an economic comparison should be made.

Section 2. Data for design

2.1 Site and geotechnical data

2.1.1 General

The design of an earth retaining structure requires information on the physical conditions in the vicinity of the structure, including the topography and layout of the site, details of adjacent foundations and services, the nature of the ground and the ground water conditions including, where applicable, the tidal and seasonal variations. An adequate site investigation should be carried out to provide the necessary information. When the site investigation has been carried out and soil test results are obtained, these are processed to provide values for the representative soil parameters (see 2.2.2). Once representative values have been established, design values should be derived for use in the equilibrium and structural design calculations.

NOTE The derivation of design values is explained in 3.1.8.

2.1.2 Site investigations

Sufficient information should be obtained on the ground and ground water conditions and the strength and deformation properties of the soils which will be retained and the soils which will support the earth retaining structure. Major earth retaining structures require an extensive site investigation. Minor earth retaining structures require sufficient information about the site together with soil data to permit the selection of representative values and design values of the soil parameters to permit a satisfactory design to be prepared. Geological maps, memoirs and handbooks should be consulted together with any other source of local knowledge.

The code of practice for site investigation BS 5930 describes the general considerations to be taken into account and details the methods of site investigation available. Information on methods of in situ and laboratory testing is given in BS 1377-1 to BS 1377-9.

The number of boreholes, or other form of investigation, should be adequate to establish the ground conditions along the length of the wall and to ascertain the variability in those conditions. The centres between boreholes will vary from site to site but should generally be at intervals of 10 m to 50 m along the length of the wall. The depth of investigation will be related to the geology of the site and to the type of wall:

- a) for a backfilled gravity or reinforced stem wall the borehole depth below founding level should be at least twice the proposed retained height;
- b) where excavation will be carried out in front of the wall the borehole depth, below excavation level, should be at least three times the proposed retained height;
- c) where the type of wall or method of construction is uncertain at the time of investigation the borehole depth, below excavation level, should be at least three times the proposed retained height.

If ground anchorages are proposed the investigation should be of sufficient extent and depth to provide data for the strata in which the anchorages will attain their bond length.

The essential properties of the soils, in the immediate vicinity of the retaining structure, should be ascertained together with the details of foundations of any adjacent structures. The relationship of the site to the overall geology should be established including the existence of any special conditions such as geological faults, movement joints, areas of landslide or any tendency of the site to shift, creep or settle, as for example in areas of mining subsidence. The possibility of externally generated vibrations and their effect upon earth pressures should be ascertained.

The process of site investigation continues during construction. Inspections should be carried out from time to time, during construction, to determine that the conditions revealed are in accordance with the design assumptions. If the conditions differ then the design should be checked against the changed conditions.

2.1.3 Ground water

An adequate design requires knowledge of the ground water levels and seepage pressures at the site, together with information as to the existence of any hydrostatic uplift pressures. Information on ground water conditions may be available from records of the site, geological maps or memoirs, or from knowledge of other similar sites in the locality. Ground water conditions may be predictable from a knowledge of the local geology. The possibility of flooding should be ascertained together with its effect on the ground water conditions.

Standpipes or piezometers should be installed where necessary to determine the ground water conditions; they should be installed in accordance with BS 5930. Where layers or strata of markedly different permeability exist, then the hydrostatic levels within each stratum should be obtained.

NOTE Water levels encountered during boring operations are unreliable; they seldom represent equilibrium conditions.

Possible changes in ground water levels due to the presence of the retaining wall and seasonal or other causes, including future trends and accident circumstances, should be investigated. Future works, in the vicinity of the wall, may give rise to changes in the long-term ground water conditions; where such future works can be reasonably anticipated, the potential changes in ground conditions should be assessed.

The presence of deleterious chemicals in the ground water and soil should be established in accordance with BS 1377-3 and the effect of such deleterious chemicals upon the corrosion of the proposed structure should be assessed in accordance with BS 8110-1 and BS 8110-2 and BS 5493.

2.1.4 Flood tides and waves

Ground water conditions, both for waterfront structures and also for structures a short distance inland, may be influenced by tidal conditions. The maximum tidal range to waterfront structure should be established including potential or possible surge tides and flood conditions. The height, length and angle of approach of waves and the resulting forces on the structure should be determined.

2.1.5 Climate

The climatic variations and their effect on the structure should be determined, including:

- a) diurnal and seasonal temperature changes and the effect on earth pressures of temperature changes, particularly ground freezing;
- b) short-term and long-term rainfall variations and the effect on earth pressures of the resulting moisture content changes;
- c) artificially induced climatic changes such as those produced in boiler houses or cold stores and their effect on earth pressures and stability.

2.1.6 Trees

Retaining walls built adjacent to existing trees may suffer deleterious effects from the penetration of root-systems.

The adverse effects of trees and root penetration includes increased loading on the structure and penetration of roots into joints or drainage systems.

- | During the course of the site investigation, the presence of trees and large shrubs should be noted so that decisions can be taken at the design stage concerning the retention or removal of such trees or shrubs.

Trees and large shrubs in general, should not be permitted nor planted within a distance from the retaining wall equal to half of their expected mature height and deciduous forest trees such as alder, beech, oak, poplar and willow should not be permitted within a distance equal to the mature height of the tree.

- | Where it is required to plant or retain trees or large shrubs close to the retaining wall after its construction, the location and choice of the tree or shrub species should be such as to minimize or eliminate the adverse effects of root penetration and the changes in the moisture content of the soil and any associated desiccation and shrinkage of the soil.
- | Useful information is provided by BS 5837.

2.2 Soil properties

2.2.1 General

The design of earth retaining structures usually involves an effective stress analysis, although in some circumstances a total stress design may be appropriate, accordingly data on the soil properties in respect of both strength (see 1.3.17) and stiffness under both drained and undrained conditions should be obtained. Soil properties are determined as part of the site investigation process but may be amplified by data from back analysis of comparable retaining structures in similar ground conditions.

The unit weights of materials, in Table 1, provide reasonable values for unit weights of soils in the absence of reliable test results.

Table 1 — Unit weights of soils (and similar materials)

Material	γ_m : moist bulk weight (kN/m ³)		γ_s : saturated bulk weight (kN/m ³)	
	Loose	Dense	Loose	Dense
A – Granular				
Gravel	16.0	18.0	20.0	21.0
Well graded sand and gravel	19.0	21.0	21.5	23.0
Coarse or medium sand	16.5	18.5	20.0	21.5
Well graded sand	18.0	21.0	20.5	22.5
Fine or silty sand	17.0	19.0	20.0	21.5
Rock fill	15.0	17.5	19.5	21.0
Brick hardcore	13.0	17.5	16.5	19.0
Slag fill	12.0	15.0	18.0	20.0
Ash fill	6.5	10.0	13.0	15.0
B – Cohesive				
Peat (very variable)	12.0		12.0	
Organic clay	15.0		15.0	
Soft clay	17.0		17.0	
Firm clay	18.0		18.0	
Stiff clay	19.0		19.0	
Hard clay	20.0		20.0	
Stiff or hard glacial clay	21.0		21.0	

2.2.2 Selection and evaluation of soil parameter values

The soil test results, obtained from the site investigation, require two stages of analysis and interpretation in order to derive satisfactory design parameters from the raw geotechnical data. In the first stage, values for the representative soil parameters are chosen (see 1.3.6). These should be conservative estimates (see 1.3.2) of the properties of the soil as it exists in situ. Care should be taken that the representative value is properly applicable to the part of the design for which it is intended. The second stage, the derivation of satisfactory design parameters from representative soil parameters, is considered in 3.1.8.

The first step in obtaining representative values of the measured soil parameters, is to make a critical examination of the raw data assisted by established calibration factors between different types of soil tests. Consistency indices, derived from moisture content and liquid and plastic limit tests, provide a useful correlation with soil strength and stiffness indices. Data from different samples and different locations will spread over a range of values. Isolated low or high values should be scrutinized to determine their accuracy; where such values are attributable to errors they should be rejected; where they are due to extreme local variations their relevance requires further consideration.

For soil parameters, such as density, for which field values can be determined with confidence from test results which show little variation, the representative value should be the mean value of the test results. Where greater variations occur or where values cannot be fixed with confidence then the representative value should be a cautious assessment of the lower limit (or the upper limit if that is the relevant bound) of the acceptable data. In the absence of detailed test information, representative values should be selected by the application of conservative bounds to generally available parameters.

The selection of representative values of soil parameters should take the following matters into account:

- a) geological and other background information;
- b) differences between the in situ conditions and the properties measured by field or laboratory tests;
- c) the effect of construction activities on the properties of the ground;
- d) changes which may occur in the field due to variations in the environment or weather;
- e) relevant data from previous projects and the performance of existing facilities.

Careful assessment of the soil parameter values is necessary to ensure selection of those values which are pertinent to the behaviour of retaining structures. The assessment of the proper parameter value is often dependent on the mechanism or mode of deformation being considered for the retaining structure, for example, different representative strengths will be required for a shear failure in a fissured material depending upon whether the shear surface is free to follow the fissures or is constrained to intersect intact material. A range of values should be considered particularly, if the soil parameter values are likely to change during the lifetime of the retaining structure.

Under serviceability conditions, where deformations are comparatively small, the soil will operate at below peak strength conditions. The appropriate strength and stiffness values may be obtained by examining the stress-strain behaviour of the soil, as given for example by laboratory triaxial tests. Under ultimate limit state conditions where deformations are comparatively large, the soil will operate at beyond peak strength conditions and may dilate to approach the critical state values consistent with the strength envelope for loose or normally consolidated soils.

Table 2, Table 3 and Table 4 provide guidance on the empirical relationship between classification and index tests and representative values of the angle of shearing resistance and the density of various materials.

2.2.3 Clay soils

The construction of a retaining wall may result in changes in the strength of the ground in the vicinity of the wall. Where the mass permeability of the ground is low the changes of strength take place over some time and therefore it is necessary to determine parameter values applicable to both short-term and to long-term conditions, i.e. undrained and drained conditions.

The undrained shear strength of a clay soil is not a fundamental soil property. Different values may be recorded in triaxial compression and extension, in direct shear and in pressuremeter tests in situ. Although conventional practice has been based on triaxial compression tests, which are consistent with active soil conditions, extension tests may be required if the behaviour of a passive zone is of particular concern.

The undrained strength of a soft clay with a small overconsolidated ratio (less than 3) increases when the positive pore pressures dissipate; but the negative pore pressures induced by shearing a stiff clay, with a high overconsolidation ratio, cause it to swell and soften in the long-term. If the undrained strength of a stiff clay is to be relied upon during temporary works construction then care is necessary to ensure that there are no sand or silt partings containing free ground water which would affect the undrained shear strength; such permeable zones are common in clays.

In assessing the strength of clay soils, particularly from undrained tests in accordance with BS 1377-7, the procedures used for sampling and testing should be taken into account. For example, U100 sampling of stiff clays leads to partial remoulding and the creation of excess negative pore pressures; these in turn cause excessive initial effective stresses which can lead to unconsolidated tests registering erroneously high undrained strengths, even when the water content has been preserved. This is due to the mode of failure of heavily overconsolidated clays, which, by strain softening, lead to shear rupture. Such failures occur at strengths lower than those applicable at the same water content but lower overconsolidation ratio. More consistent results are obtained if samples are consolidated to a best estimate of in situ effective stresses prior to shearing. Representative values for undrained strength parameters should be assessed for the peak strength and for the remoulded strength of the soil. The values for the representative peak strength should make due allowance for the influence of sampling and the method of testing, as well as for likely softening on excavation.

To determine the strength of clay soils, for an effective stress analysis, triaxial tests may be carried out either fully drained or undrained with pore pressure measurement, provided the samples are fully saturated in accordance with BS 1377-7 and BS 1377-8. The tests are carried out sufficiently slowly to ensure equalization of pore pressures. The Mohr-Coulomb failure envelope for overconsolidated clays, of initially identical samples, is generally curved, see Figure 1. At effective pressures close to the preconsolidation pressure, the soil mobilizes its critical state angle of shearing φ'_{crit} . At lower initial effective stresses, i.e. at higher overconsolidated ratios, the soil exhibits a dilatant peak at failure before its strength drops to, and possibly beyond, the critical state value. Representative values should be assessed separately for the peak strength and for the critical state strength of the soil. The representative peak strength should be appropriate to the anticipated stress state of the soil in the ground. Where the stress-strain curve never reaches a peak, during the maximum strain range achievable during test, the peak strength should be assumed to be the largest strength mobilized during the test. It may be represented by values of c' and ω' or by secant values of φ' . The representative critical state strength is represented by the critical state angle of shearing resistance, φ'_{crit} . Cohesive soils with high clay contents exhibit the greatest fall from peak to residual strength, forming a polished rupture surface. Previous shear surfaces, in plastic clays, may be reactivated at low residual friction angles φ'_r . First time slides due to new construction have been found to mobilize mass strengths no lower than φ'_{crit} .

Two approaches may be adopted for the conventional linearization of the peak soil envelope over some desired range of stress, see Figure 1. A secant φ' value can be selected as a function of stress level. If a single value is chosen, the resulting envelope is linear to the origin and falls safely inside the envelope of tests carried out from identical initial conditions. Since soil samples from the field are not identical, the method should normally be applied by selecting the lowest secant φ' for any sample tested within the target range of stress. Alternatively, the tangent parameters (c' , φ') may be used, where each is a function of stress level for identical samples. Sample variation causes scatter in the tangent parameter values and conservative values are best selected by fitting a lower bound to the relevant data, taking care to consider the range of effective stress required.

In the absence of reliable laboratory test data, the conservative values of φ'_{crit} given in Table 2 may be used, with $c' = 0$.

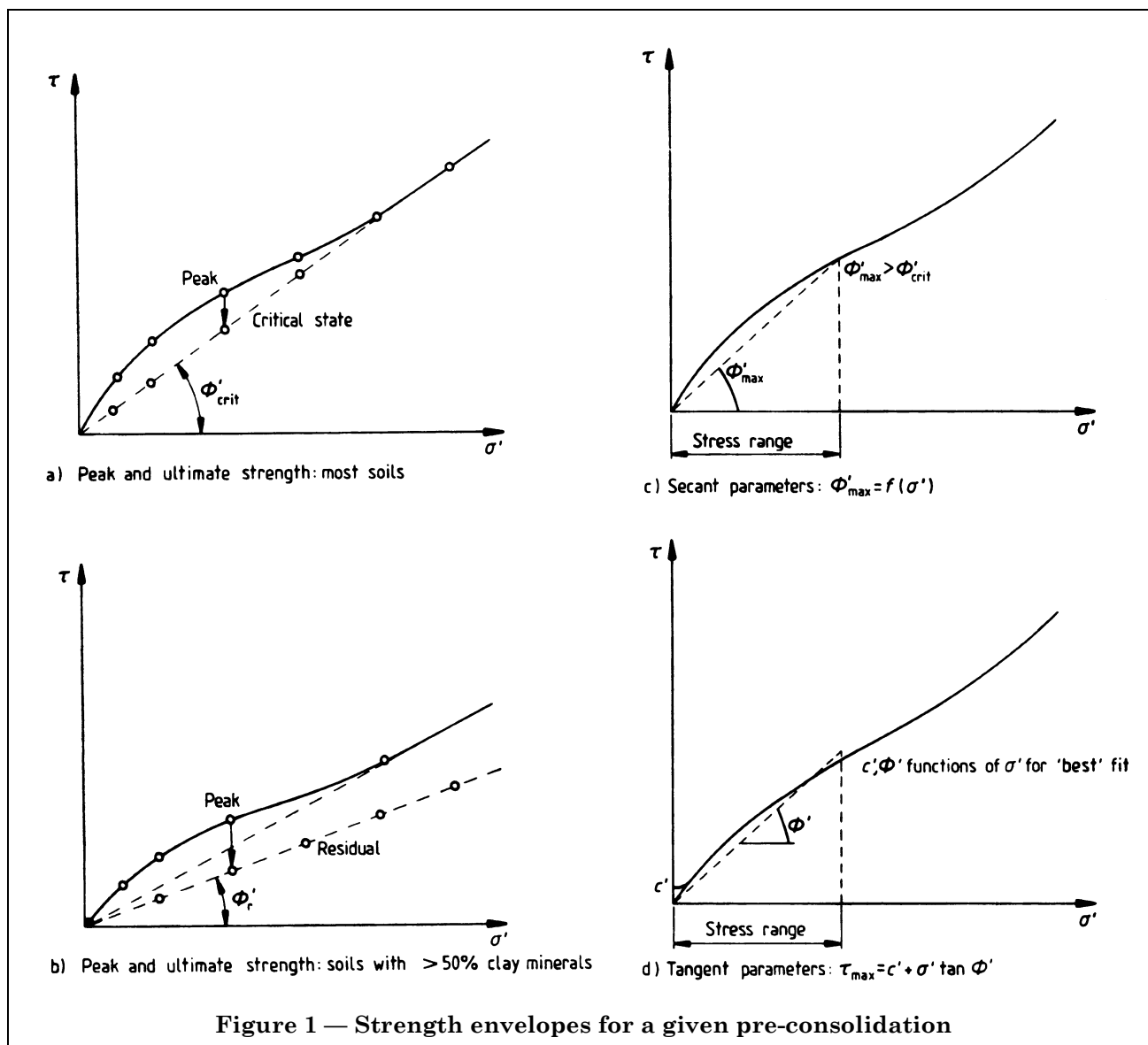
If samples of clay containing veins or seams of sand or silt are remoulded for the plasticity index tests the test results give lower plasticity indices than the clay itself. Care should be taken to carry out the tests on the clay alone. If there are doubts as to the inclusion of sand or silt then, in Table 2 use the next value of the plasticity index higher than recorded in the tests.

Table 2 — φ'_{crit} for clay soils

Plasticity index %	φ'_{crit} (degrees)
15	30
30	25
50	20
80	15

In all tests a non-linear soil response should be anticipated, so that stress-strain curves form hysteresis loops on load-unload-reload cycles. In assessing the deformation properties of soils, the stiffness measured in conventional laboratory tests in accordance with BS 1377-5 and BS 1377-6 generally underestimates in situ values derived from back analysis of instrumented field structures. Appropriate stiffness values can be measured in the laboratory by laboratories experienced in this specialist work provided particular care is taken in sample preparation and local strain measurement. Sample disturbance is corrected by first taking the sample through its most recent effective stress cycle so that its in situ state is properly recreated.

Stiffness parameters can be determined from certain field tests which cause little disturbance in accordance with BS 1377-9 and CIRIA Ground engineering report, 1987.



2.2.4 Cohesionless soils

The strength and stiffness of cohesionless soils are determined indirectly by in situ static or dynamic penetration tests. Details of three types of penetration tests as well as plate loading tests are given in BS 1377-9. The peak and critical state angles of shearing resistance for siliceous sands and gravels may be estimated from the following equations:

The estimated peak effective angle of shearing resistance is given by:

$$\varphi'_{\max} = 30 + A + B + C \quad (1)$$

The estimated critical state angle of shearing resistance is given by:

$$\varphi'_{\text{crit}} = 30 + A + B$$

The values of:

A = angularity of the particles

B = grading of the sand/gravel

C = results of standard penetration tests

are given in Table 3.

The standard penetration test (SPT) values should be corrected for the effect of overburden pressure in accordance with Figure 2 (see Thorburn, 1963); other correction effects may be necessary. See CIRIA Report FR/CP/7²⁾. Bolton (1986) has introduced empirical relations between $\varphi'_{\max}$, φ'_{crit} , initial soil relative density and mean effective stress at failure to reflect the change in the secant value of peak angle of shearing resistance with the change in the mean effective stress in the ground.

2.2.5 Silts

It is difficult and often impracticable to obtain undisturbed samples of silts and fine sands, even employing special sampling techniques. Loose silts are readily liquified by vibration, both during probing and during the life of the retaining wall; accordingly excess pore pressures should be taken into account. Inorganic siliceous silts can generate as much dilatancy as sands, at the same relative density, but they more easily soften to critical states in thin rupture bands. In the absence of other data and where disturbed samples have shown the silt is a rock flour with negligible organic or clay mineral content, the representative effective angle of shearing may be conservatively taken as φ'_{crit} in Table 3.

2.2.6 Rock

The engineering properties of rock relevant in design are controlled by the extent and orientation of the bedding planes and joints within the rock mass together with the water pressures on the discontinuity planes. The site investigation should establish the strength and orientation of the discontinuity planes. Weak rocks, particularly weakly cemented sandstones, fissured shales and chalk, are often difficult materials to sample and test.

Some correlation has been obtained between the standard penetration test in accordance with BS 1377-9 and the strength and stiffness properties for certain weak rock masses. In addition the mass rock properties may be derived from compression wave and shear wave velocity measurements.

The following indicative values of the effective angle of friction in Table 4 relate to rocks which can conservatively be treated as composed of granular fragments, i.e. they are closely and randomly jointed or otherwise fractured, having an RQD (rock quality designation) value close to zero.

²⁾ In preparation.

Table 3 — φ' for siliceous sands and gravels

A – Angularity^a	A (degrees)								
Rounded	0								
Sub-angular	2								
Angular	4								
B – Grading of soil^b	B (degrees)								
Uniform	0								
Moderate grading	2								
Well graded	4								
C – N' (blows 300 mm)	C (degrees)								
<10	0								
20	2								
40	6								
60	9								
^a Angularity is estimated from visual description of soil. ^b Grading can be determined from grading curve by use of: Uniformity coefficient = D_{60}/D_{10} where D_{10} and D_{60} are particle sizes such that in the sample, 10 %, of the material is finer than D_{10} and 60 % is finer than D_{60} . <table> <tr> <th>Grading</th><th>Uniformity coefficient</th></tr> <tr> <td>Uniform</td><td><2</td></tr> <tr> <td>Moderate grading</td><td>2 to 6</td></tr> <tr> <td>Well graded</td><td>>6</td></tr> </table> A step-graded soil should be treated as uniform or moderately graded soil according to the grading of the finer fraction. N' from results of standard penetration test modified where necessary by Figure 2. Intermediate values of A, B and C by interpolation.		Grading	Uniformity coefficient	Uniform	<2	Moderate grading	2 to 6	Well graded	>6
Grading	Uniformity coefficient								
Uniform	<2								
Moderate grading	2 to 6								
Well graded	>6								

Table 4 — φ' for rock

Stratum	φ' (degrees)
Chalk	35
Clayey marl	28
Sandy marl	33
Weak sandstone	42
Weak silstone	35
Weak mudstone	28
NOTE 1 The presence of a preferred orientation of joints, bedding or cleavage in a direction near that of a possible failure plane may require a reduction in the above values, especially if the discontinuities are filled with weaker materials. NOTE 2 Chalk is defined here as unweathered medium to hard, rubbly to blocky chalk, grade III (see Clayton, 1990).	

2.2.7 Fill

A wide range of materials may be used as fill behind retaining walls. Selected cohesionless granular fill placed in a controlled manner such as well graded small rockfills, gravels and sands, are suitable as fill. Cohesive materials, subject to the further recommendations below, may be suitable but other materials such as industrial, chemical and domestic wastes should not be used. All fill materials should be properly investigated and classified.

The use of cohesive soil as fill may involve problems during design and construction additional to those which occur with granular fill, but the use of cohesive soil may result in significant economies by avoiding the need to import granular materials.

The cohesive soil should be within a range suitable for adequate compaction; for guidance on the selection of such fill see the Transport Research Laboratory publications LR406, LR750, SR522 and RR90, the proceedings of the conference on clay fills, ICE 1979, the DoT Specification for highway works, 1991 and DoT Standard BD30/87.

The placement moisture content of cohesive fill should be close to the final equilibrium value to prevent either the swelling of clays placed too dry or the consolidation of clays placed too wet. Volume changes in clay soils will affect the pressure distribution on the wall in the medium- to long-term. Compaction pressures should also be taken into account, see **3.3.3.6**. Problems associated with swelling and consolidation will be minimized if clay fill is limited to clays with a liquid limit not exceeding 45 % and a plasticity index not exceeding 25 % (DoT Specification for highway works, 1991).

Chalk with a saturation moisture content of 20 % or less is acceptable as fill and may be compacted as a well graded granular soil. The saturation moisture content of chalk is evaluated from the dry density of individual lumps, determined in accordance with **7.3** of BS 1377-2:1990.

Saturation moisture content =

$$\left\{ \frac{1}{\gamma_d} - \frac{1}{2.7} \right\} \times 100 \% \quad (2)$$

where

γ_d = dry density in mg/m³.

Exceptionally, some granites are found which deteriorate by weathering of the feldspars. If it is proposed to use such granitic rocks, due allowance should be made for deterioration in estimating the angle of friction.

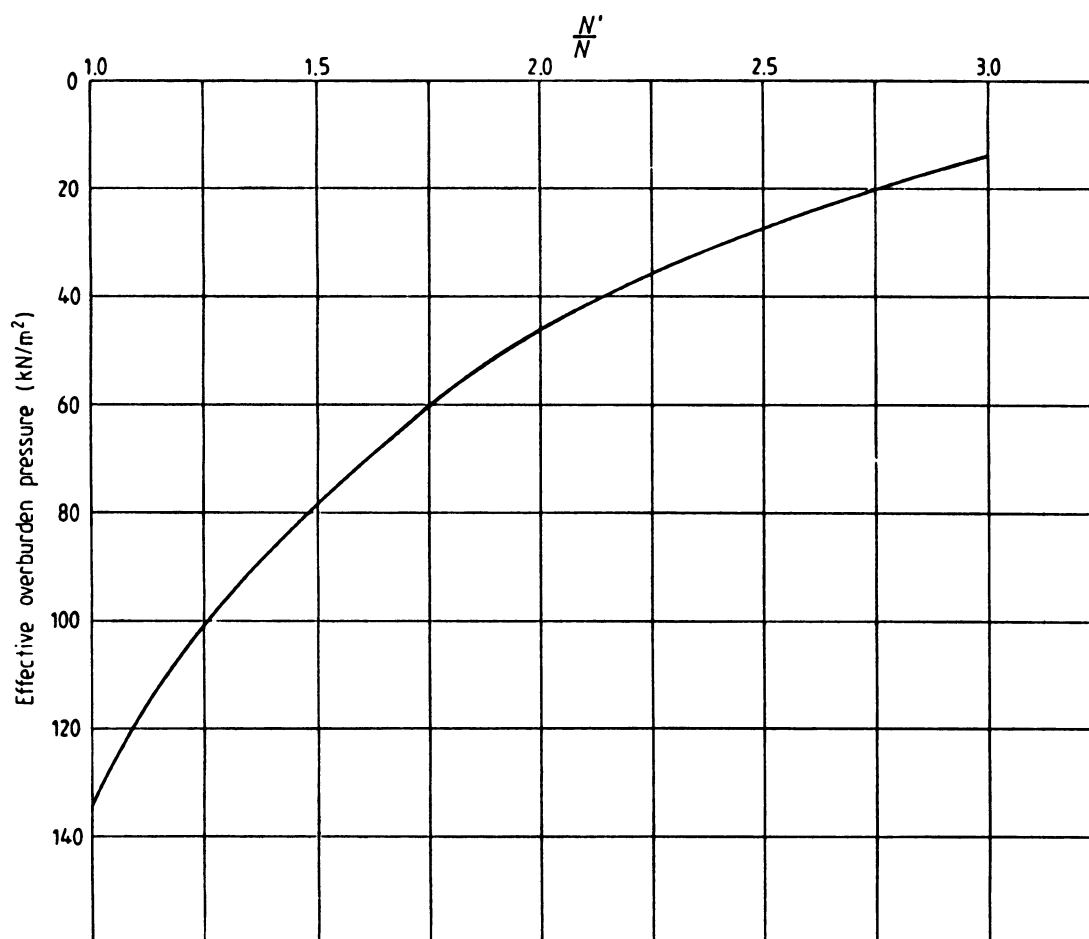


Figure 2 — Derivation of N' from SPT value N

Conditioned pulverized fuel ash (PFA) from a single source may be used as fill: it should be supplied at a moisture content of 80 % to 100 % of the optimum moisture content.

Shale, mudstone and steel slag swell when they absorb water. These materials should not be used as fill, except at some distance from the retaining wall. Peaty or highly organic soil should not be used as fill.

2.2.8 Wall friction, base friction and undrained wall adhesion

Representative values of the strength of the soil sliding as a mass against the wall can be determined from appropriate drained and undrained shear box tests. The wall material should be placed in the bottom half of the box with its interface on the plane of sliding. The soil is then placed in the upper part of the box in the required state. With large scale surface roughnesses (i.e. concrete formed on or against coarse granular soils) side and end effects of the small shear box (60 mm × 60 mm) will affect the laboratory test results and large shear boxes should be used. Tests should be carried out over the range of normal stresses likely to exist on the wall during its life. Testing should be continued to determine any reduction in strength with continued sliding.

In the absence of large shear box test results the representative strength, in terms of effective stress, should not exceed values calculated using:

- a) $\delta = \varphi'_{\text{crit}}$ for the soil, for rough surfaces with a texture coarser than that of the median particle size;
- b) $\delta = 20^\circ$, for smooth surfaces with a texture finer than that of the median particle size.

No effective adhesion c' should be taken for walls or bases in contact with soil.

The effects of wall construction on the interface friction between the soil and the wall should be taken in account. The undrained shear strength mobilized on a wall surface may be irrelevant due to the presence of drainage material creating effective friction conditions on the boundary. Cracking and air entry against the wall also tend to produce friction conditions with zero (atmospheric) pore pressures against the wall, in contrast to the possibly negative pore pressures mobilized temporarily within the clay mass. Under these circumstances the representative coefficient of effective friction on the boundary is $\tan \delta$ and the normal effective stress at the boundary is equal to the normal total stress σ_n in the soil so that the representative wall friction is $\sigma_n \tan \delta$. Where the undrained soil strength against a surface is relevant, and in the absence of appropriate tests, the representative value should not exceed the remoulded undrained strength of the soil.

2.3 Externally applied loads

All necessary details should be obtained of static, transient and dynamic loads that may be applied externally to the earth retaining structure.

Section 3. Design philosophy, design method and earth pressures

3.1 Design philosophy

3.1.1 General

The design of earth retaining structures requires consideration of the interaction between the ground and the structure. It requires the performance of two sets of calculations:

- 1) a set of equilibrium calculations to determine the overall proportions and the geometry of the structure necessary to achieve equilibrium under the relevant earth pressures and forces;
- 2) structural design calculations to determine the size and properties of the structural sections necessary to resist the bending moments and shear forces determined from the equilibrium calculations.

Both sets of calculations are carried out for specific design situations (see **3.2.2**) in accordance with the principles of limit state design. The selected design situations should be sufficiently severe and varied so as to encompass all reasonable conditions which can be foreseen during the period of construction and the life of the retaining wall.

3.1.2 Limit state design

This code of practice adopts the philosophy of limit state design. This philosophy does not impose upon the designer any special requirements as to the manner in which the safety and stability of the retaining wall may be achieved, whether by overall factors of safety, or partial factors of safety, or by other measures. Limit states (see **1.3.13**) are classified into:

- a) ultimate limit states (see **3.1.3**);
- b) serviceability limit states (see **3.1.4**).

Typical ultimate limit states are depicted in Figure 3. Rupture states which are reached before collapse occurs are, for simplicity, also classified and treated as ultimate limit states. Ultimate limit states include:

- a) instability of the structure or any part of it, including supports and foundations, considered as a rigid body;
- b) failure by rupture of the structure or any part of it, including supports and foundations.

3.1.3 Ultimate limit states

3.1.3.1 General

The following ultimate limit states should be considered. Failure of a retaining wall as a result of:

- a) instability of the earth mass, e.g. a slip failure, overturning or a rotational failure where the disturbing moments on the structure exceed the restoring moments, a translational failure where the disturbing forces (see **1.3.8**) exceed the restoring forces and a bearing failure. Instability of the earth mass involving a slip failure may occur where:
 - 1) the wall is built on sloping ground which itself is close to limiting equilibrium; or
 - 2) the structure is underlain by a significant depth of clay whose undrained strength increases only gradually with depth; or
 - 3) the structure is founded on a relatively strong stratum underlain by weaker strata; or
 - 4) the structure is underlain by strata within which high pore water pressures may develop from natural or artificial sources.
- b) failure of structural members including the wall itself in bending or shear;
- c) excessive deformation of the wall or ground such that adjacent structures or services reach their ultimate limit state.

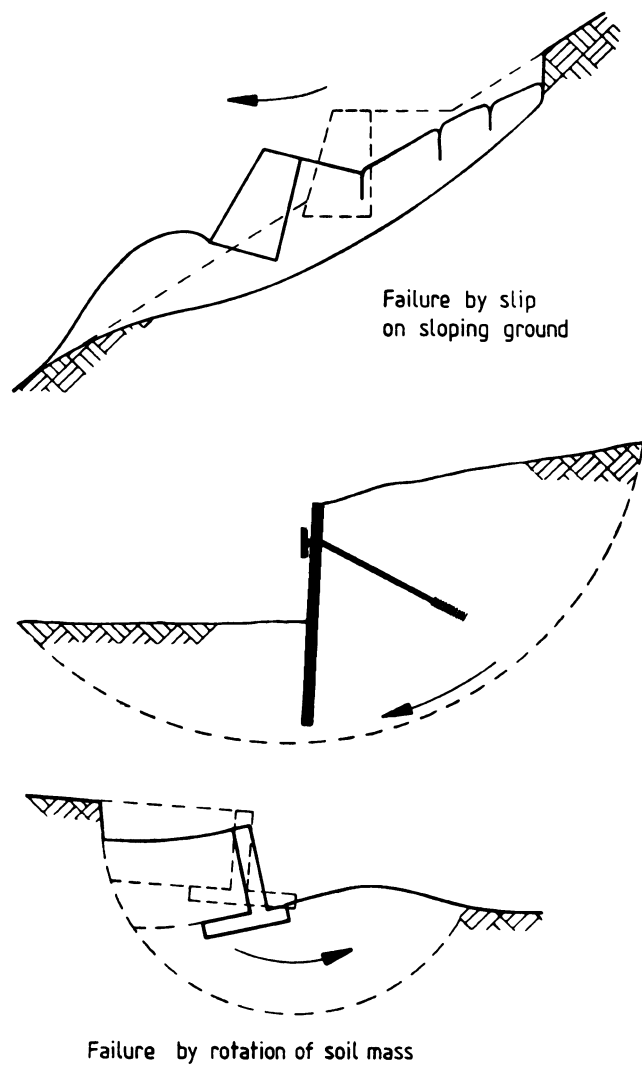
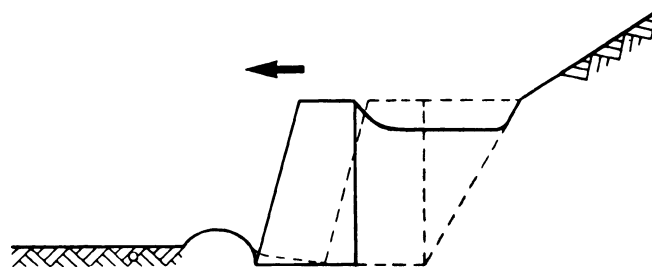
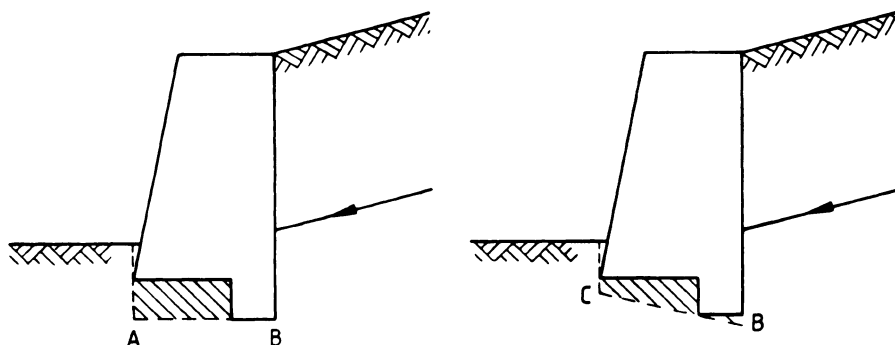


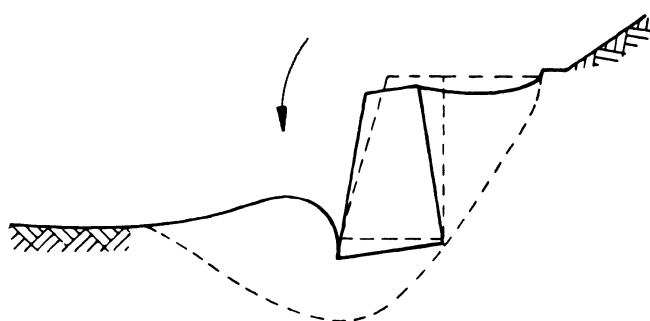
Figure 3 — Limit states for earth retaining structures



Failure of gravity wall due to sliding

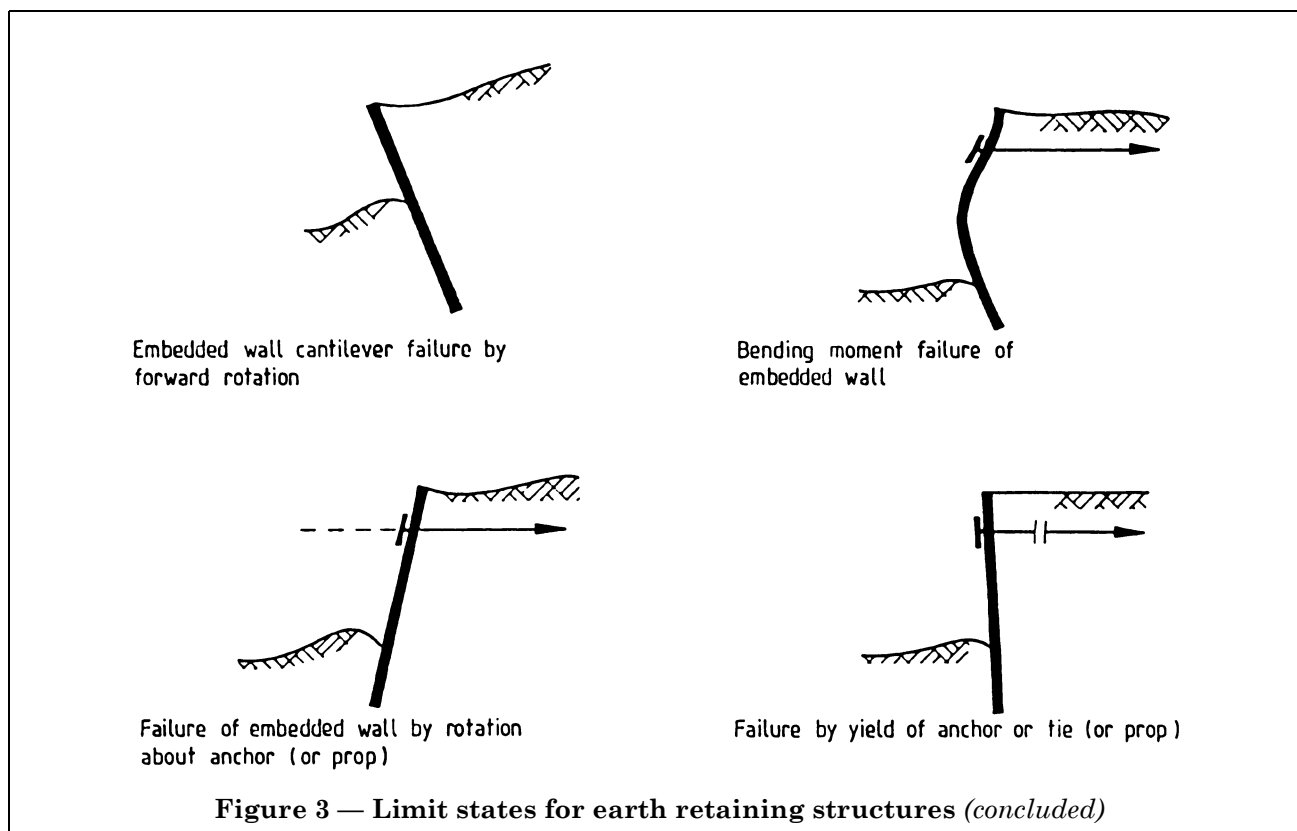


Gravity wall with shear key
Failure may be on A B. or along
B C. sloping plane



Bearing capacity failure of gravity wall
on restricted depth of soft soil

Figure 3 — Limit states for earth retaining structures *(continued)*



3.1.3.2 Analysis method

Where the mode of failure involves a slip failure the methods of analysis, for stability of slopes, are described in BS 6031 and in BS 8081. Where the mode of failure involves a bearing capacity failure, the calculations should establish an effective width of foundation. The bearing pressures as determined from 4.2.2 should not exceed the ultimate bearing capacity in accordance with BS 8004.

Where the mode of failure is by translational movement, with passive resistance excluded, stable equilibrium should be achieved using the design shear strength of the soil in contact with the base of the earth retaining structure.

Where the mode of failure involves a rotational or translational movement, the stable equilibrium of the earth retaining structure depends on the mobilization of shear stresses within the soil. The full mobilization of the soil shear strength gives rise to limiting active and passive thrusts. These limiting thrusts act in concert on the structure only at the point of collapse, i.e. ultimate limit state.

3.1.4 Serviceability limit states

The following serviceability limit states should be considered:

- a) substantial deformation of the structure;
- b) substantial movement of the ground.

The soil deformations, which accompany the full mobilization of shear strength in the surrounding soil, are large in comparison with the normally acceptable strains in service. Accordingly, for most earth retaining structures the serviceability limit state of displacement will be the governing criterion for a satisfactory equilibrium and not the ultimate limit state of overall stability. However, although it is generally impossible or impractical to calculate displacements directly, serviceability can be sufficiently assured by limiting the proportion of available strength actually mobilized in service; by the method given in 3.2.4 and 3.2.5.

The design earth pressures used for serviceability limit state calculations will differ from those used for ultimate limit state calculations only where structures are to be subjected to differing design values of external loads (generally surcharge and live loads) for the ultimate limit state and for the serviceability limit state.

3.1.5 Limit states and compatibility of deformations

The deformation of an earth retaining structure is important because it has a direct effect upon the forces on the structure, the forces from the retained soil and the forces which result when the structure moves against the soil. The structural forces and bending moments due to earth pressures reduce as deformation of the structure increases.

The maximum earth pressures on a retaining structure occur during working conditions and the necessary equilibrium calculations (see 3.2.1) are based on the assumption that earth pressures greater than fully active pressure (see 1.3.11) and less than fully passive will act on the retaining structure during service. As ultimate limit state with respect to soil pressures is approached, with sufficient deformation of the structure, the active earth pressure (see 1.3.1) in the retained soil reduces to the fully active pressure and the passive resistance (see 1.3.15) tends to increase to the full available passive resistance (see 1.3.12).

The compatibility of deformation of the structure and the corresponding earth pressures is important where the form of structure, for example a propped cantilever wall, prevents the occurrence of fully active pressure at the prop. It is also particularly important where the structure behaves as a brittle material and loses strength as deformation increases, such as an unreinforced mass gravity structure or where the soil is liable to strain softening as deformation increases.

3.1.6 Design values of parameters

These are applicable at the specified limit states in the specified design situations. All elements of safety and uncertainty should be incorporated into the design values.

The selection of design values for soil parameters should take account of:

- a) the possibility of unfavourable variations in the values of the parameters;
- b) the independence or interdependence of the various parameters involved in the calculation;
- c) the quality of workmanship and level of control specified for the construction.

3.1.7 Applied loads

The design value for the density of fill materials, should be a pessimistic or unfavourable assessment of actual density.

For surcharges and live loadings different values may be appropriate for the differing conditions of serviceability and ultimate limit states and for different load combinations. The intention of this code of practice is to determine those earth pressures which will not be exceeded in a limit state, if external loads are correctly predicted. External loads, such as structural dead loads or vehicle surcharge loads may be specified in other codes as nominal or characteristic values. Some of the structural codes, with which this code interfaces, specify different load factors to be applied for serviceability or ultimate limit state checks and for different load combinations, see 3.2.7. Design values of loads, derived by factoring or otherwise, are intended, here, to be the most pessimistic or unfavourable loads which should be used in the calculations for the structure. Similarly, when external loads act on the active or retained side of the wall these same external loads should be derived in the same way. The soil is then treated as forming part of the whole structural system.

3.1.8 Design soil strength (see 1.3.8)

Assessment of the design values depends on the required or anticipated life of the structure, but account should be taken also of the short-term conditions which apply during and immediately following the period of construction. Single design values of soil strength should be obtained from a consideration of the representative values for peak and ultimate strength. The value so selected will satisfy, simultaneously, the considerations of ultimate and serviceability limit states. The design value should be the lower of:

- a) that value of soil strength, on the stress-strain relation leading to peak strength, which is mobilized at soil strains acceptable for serviceability. This can be expressed as the peak strength reduced by a mobilization factor M as given in 3.2.4 or 3.2.5; or
- b) that value which would be mobilized at collapse, after significant ground movements. This can generally be taken to be the critical state strength.

Design values selected in this way should be checked to ensure that they conform to **3.1.6**. Design values should not exceed representative values of the fully softened critical state soil strength.

3.1.9 Design earth pressures

The design values of lateral earth pressure are intended to give an overestimate of the earth pressure on the active or retained side and an underestimate of the earth resistance on the passive side for small deformations of the structure as a whole, in the working state. Earth pressures reduce as fully active conditions are mobilized at peak soil strength in the retained soil, under deformations larger than can be tolerated for serviceability. As collapse threatens, the retained soil approaches a critical state, in which its strength reduces to that of loose material and the earth pressures consequently tend to increase once more to active values based on critical state strength.

The initial presumption should be that the design earth pressure will correspond to that arising from the design soil strength, see **3.1.8**. But the mobilized earth pressure in service, for some walls, will exceed these values. This enhanced earth pressure will control the design, for example.

- a) Where clays may swell in the retained soil zone, or be subject to the effects of compaction in layers, larger earth pressures may occur in that zone, causing corresponding resistance from the ground, propping forces, or anchor tensions to increase so as to maintain overall equilibrium.
- b) Where clays may have lateral earth pressures in excess of the assessed values taking account of earth pressures prior to construction and the effects of wall installation and soil excavation or filling, the earth pressure in retained soil zones will be increased to maintain overall equilibrium.
- c) Where both the wall and backfill are placed on compressible soils, differential settlement due to consolidation may lead to rotation of the wall into the backfill. This increases the earth pressures in the retained zone.
- d) Where the structure is particularly stiff, for example fully piled box-shaped bridge abutments, higher earth pressures, caused, for example by compaction, may be preserved, notwithstanding that the degree of wall displacement or flexibility required to reduce retained earth pressures to their fully active values in cohesionless materials is only of the order of a rotation of 10^{-3} radians.

In each of these cases, mobilized soil strengths will increase as deformations continue, so the unfavourable earth pressure conditions will not persist as collapse approaches.

The design earth pressures are derived from design soil strengths using the usual methods of plastic analysis, with earth pressure coefficients (see **1.3.9**) given in this code of practice being based on Kerisel & Absi (1990). The same design earth pressures are used in the default condition for the design of structural sections, see **3.2.7**.

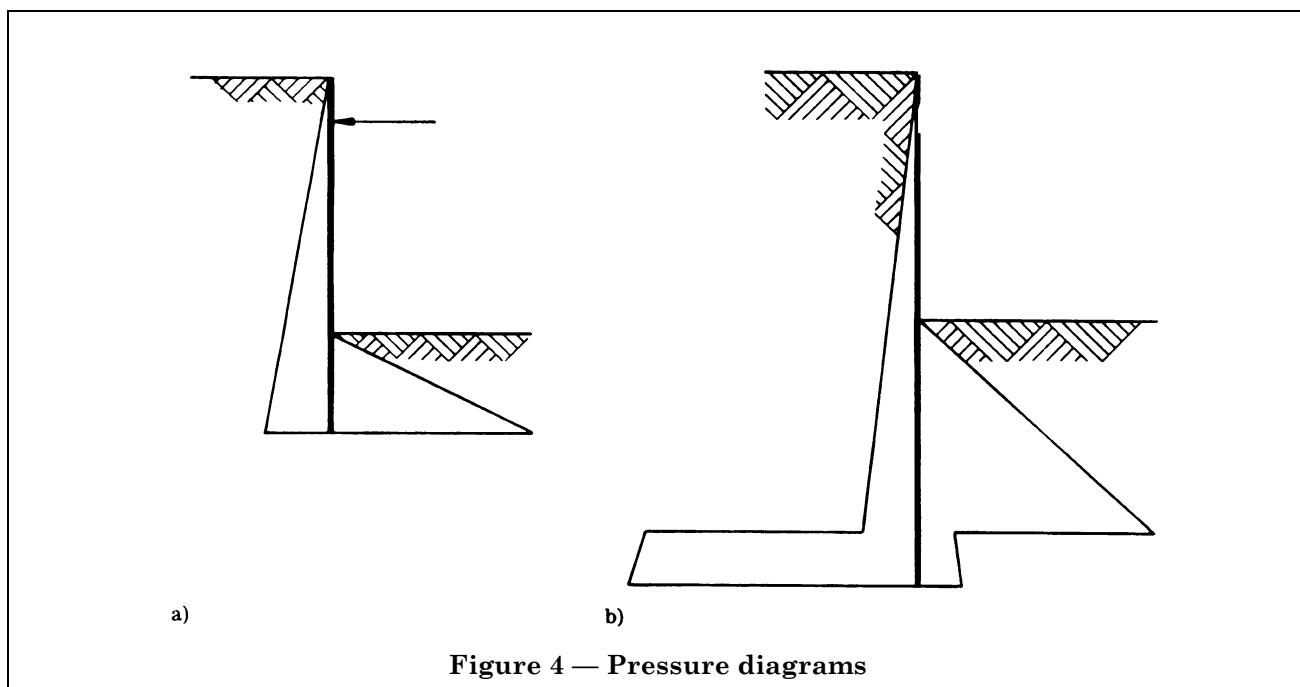
3.2 Design method

3.2.1 Equilibrium calculations

In order to determine the geometry of the retaining wall, for example the depth of penetration of an embedded wall (see **1.3.10**), equilibrium calculations should be carried out for carefully formulated design situations. The design calculations relate to a free-body diagram of forces and stresses for the whole retaining wall. The design calculations should demonstrate that there is global equilibrium of vertical and horizontal forces, and of moments. Separate calculations should be made for different design situations.

The structural geometry of the retaining wall and the equilibrium calculations should be determined from the design earth pressures derived from the design soil strength using the appropriate earth pressure coefficients.

Design earth pressures will lead to active and passive pressure diagrams of the type shown in Figure 4. The earth pressure distribution should be checked for global equilibrium of the structure. Horizontal forces equilibrium and moment equilibrium will give the prop force in Figure 4a) and the location of the point of reversed stress conditions near the toe in Figure 4b). Vertical forces equilibrium should also be checked.



3.2.2 Design situations

3.2.2.1 General

The specification of design situations should include the disposition and classification of the various zones of soil and rock and the elements of construction which could be involved in a limit state event. The specification of design situations should follow a consideration of all uncertainties and the risk factors involved, including the following:

- a) the loads and their combinations, e.g. surcharge and/or external loads on the active or retained side of the wall;
- b) the geometry of the structure, and the neighbouring soil bodies, representing the worst credible conditions, for example over-excavation during or after construction;
- c) the material characteristics of the structure, e.g. following corrosion;
- d) effects due to the environment within which the design is set, such as:
 - ground water levels, including their variations due to the effects of dewatering, possible flooding or failure of any drainage system;
 - scour, erosion and excavation, leading to changes in the geometry of the ground surface;
 - chemical corrosion;
 - weathering;
 - freezing;
 - the presence of gases emerging from the ground;
 - other effects of time and environment on the strength and other properties of materials;
- e) earthquakes;
- f) subsidence due to mining or other causes;
- g) the tolerance of the structure to deformations;

- h) the effect of the new structure on existing structures or services and the effect of existing structures or services on the new structure;
- i) for structures resting on or near rock, the consideration of:
 - interbedded hard and soft strata;
 - faults, joints and fissures;
 - solution cavities such as swallow holes or fissures, filled with soft material, and continuing solution processes.

3.2.2.2 *Unplanned excavation and surcharge*

In checking the stable equilibrium and soil deformation, retaining walls should be designed assuming a depth of unplanned excavation in front of the wall. The depth of the excavation should be not less than 10 % of the total height retained for cantilever walls or of the height retained below the lowest support level for propped or anchored walls, but the depth of the excavation may be limited to 0.5 m. This recommendation for an additional excavation as a design criterion is to provide for unforeseen and accidental events. The recommended values should be reviewed for each design; more adverse values should be adopted in particular critical or uncertain conditions but smaller values may be adopted where adverse conditions are beyond reasonable doubt. Foreseeable excavations such as service or drainage trenches in front of a retaining wall, which may be required at some stage in the life of the structure, should be treated as a planned excavation. Actual excavation beyond the planned depth is outside the design considerations of this code.

In addition to the design check for unplanned excavation a further but separate check should be carried out for stable equilibrium and soil deformation with the retaining wall designed for a design surcharge load as recommended in 3.3.4.

3.2.2.3 *Water pressure regime*

The water pressure regime used in the design should be the most onerous that is considered to be reasonably possible.

3.2.3 *Calculations based on total and effective stress parameters*

The changes in loading associated with the construction of a retaining wall may result in changes in the strength of the ground in the vicinity of the wall. Where the mass permeability of the ground is low these changes of strength take place over some time and therefore the design should consider conditions in both the short- and long-term. Which condition will be critical depends on whether the changes in load applied to the soil mass cause an increase or decrease in soil strength. The long-term condition is likely to be critical where the soil mass undergoes a net reduction in load as a result of excavation, such as adjacent to a cantilever wall. Conversely where the soil mass is subject to a net increase in loading, such as beneath the foundation of a gravity or reinforced stem wall at ground level, the short-term condition is likely to be critical for stability. When considering long-term earth pressures and equilibrium, allowance should be made for changes in ground water conditions and pore water pressure regime which may result from the construction of the works or from other agencies.

Calculations for long-term conditions require shear strength parameters to be in terms of effective stress and should take account of a range of water pressures based on considerations of possible seepage flow conditions within the earth mass. Effective stress methods can also be used to assess the short-term conditions provided the pore water pressures developed during construction are known. A total stress method of analysis may be used to assess the short-term conditions in clays and soils of low permeability, but an inherent assumption of this method is that there will be no change in the soil strength as a result of the changes in load caused by the construction. For granular materials and soils of high permeability all excess pore water pressure will dissipate rapidly so that the relevant strength is always the drained strength and the earth pressures and equilibrium calculations are always in terms of effective stresses.

3.2.4 *Design using total stress parameters*

The retaining wall should be designed to be in equilibrium when based on a mobilized undrained design clay strength (design c_u) which does not exceed the representative undrained strength divided by a mobilization factor M . The value of M should not be less than 1.5 if wall displacements are required to be less than 0.5 % of wall height. The value of M should be larger than 1.5 for clays which require large strains to mobilize their peak strength.

3.2.5 Design using effective stress parameters

The retaining wall should be designed to be in equilibrium mobilizing a soil strength the lesser of:

- a) the representative peak strength of the soil divided by a factor $M = 1.2$:

that is:

$$\text{design } \tan \varphi' = \frac{\text{representative } \tan \varphi'_{\max}}{M} \quad (3)$$

$$\text{design } c' = \frac{\text{representative } c'}{M} \quad (4)$$

or

- b) the representative critical state strength of the soil.

This will ensure that for soils which are medium dense or firm the wall displacements in service will be limited to 0.5 % of the wall height. The mobilization factor of **1.2** should be used in conjunction with the “unplanned” excavation in front of the wall, the minimum surcharge loading and the water pressure regime, see **3.2.2.2** and **3.2.2.3**.

A more detailed analysis of displacement should be performed where tighter criteria are to be applied or for soft or loose soils. The criteria a) and b), taken together, should provide a sufficient reserve of safety against small unforeseen loads and adverse conditions.

In stiff clays subject to cycles of strain, such as through seasonal variation of pore water pressure, the long-term peak strength may deteriorate to the critical state strength. The requirements of a) and b) above are sufficiently cautious to accommodate this possibility.

3.2.6 Design values of wall friction, base friction and undrained wall adhesion

These should be derived from the representative strength determined in accordance with **2.2.8**, using the same mobilization factors as for the adjacent soil.

The design value of the friction or adhesion to be mobilized at an interface with the structure should be the lesser of:

- a) the representative value determined by test as described in **2.2.8** if such test results are available; or
b) 75 % of the design shear strength to be mobilized in the soil itself, that is using:

$$\text{design } \tan \delta = 0.75 \times \text{design } \tan \varphi' \quad (5)$$

$$\text{design } c_w = 0.75 \times \text{design } c_u \quad (6)$$

Since for the soil mass:

$$\text{design } \tan \varphi' = \frac{\text{representative } \tan \varphi'_{\max}}{1.2} \quad (7)$$

this is equivalent to:

$$\frac{\text{design } \delta}{\text{representative } \varphi'} \approx \frac{2}{3} \quad (8)$$

similarly, in total stress analysis:

$$\frac{\text{design } c_w}{\text{representative } c_u} = 0.5 \text{ after taking } M = 1.5 \quad (9)$$

The friction or adhesion, which can be mobilized in practice, is generally less than the value deduced on the basis of soil sliding against the relevant surface. It is unlikely for example, that a cantilever wall will remain at constant elevation while the active soil zone subsides creating full downward wall friction on the retained side, and the passive zone heaves creating full upward wall friction on the excavated side. It is more likely that the wall would move vertically with one or other soil zone, reducing friction on that side, and thereby attaining vertical force equilibrium. The 25 % reduction in the design shear strength in b) above makes an allowance for this possibility. Further reductions, and even the elimination of wall friction or its reversal, may be necessary when soil structure interaction is taken into account. Wall friction on the retained or active side should be excluded when the wall is capable of penetrating deeper, due to the vertical thrust imparted by inclined anchors on an embedded wall, by structural loads on a basement wall, or where a clay soil may heave due to swelling during outward movement of the wall. Wall friction on the passive side should be excluded when the wall is prevented from sinking but the adjacent soil may fail to heave, due for example to settlement of loose granular soils induced by cyclic loads, or when the wall is free to move upwards with the passive soil zone, as may happen with buried anchor blocks.

3.2.7 Design to structural codes

The earth pressures to be used in structural design calculations should be determined using methods set out in 3.1.9 and 3.2.2, taking into account, where applicable, the effect of compaction stresses (see 3.3.3.6). These earth pressures are the most unfavourable that are likely to occur. They occur under working conditions.

The earth pressures which occur at ultimate limit state are less.

The earth pressures at structural serviceability limit state and ultimate limit state will be similar for relatively rigid structure, such as mass gravity walls, because the displacement criteria will be similar.

The strength of the structural sections of the retaining wall may be determined using either permissible stress methods or by the methods use in partial factor structural codes (see 3.1.2).

3.3 Disturbing forces

3.3.1 General

The disturbing forces to be taken into account in the equilibrium calculations are the earth pressures on the active or retained side of the wall, together with loads due to the compaction of the fill (if any) behind the wall, surcharge loads, external loads and last, but by no means least, the water pressure.

3.3.2 At-rest earth pressures

The earth pressures which act on retaining walls, or parts of retaining walls, below existing ground, depend on the initial or at-rest state of stress in the ground. For an undisturbed soil at a state of rest, the ratio of the horizontal to vertical stress depends on the type of soil, its geological origin, the temporary loads which may have acted on the surface of the soil and the topography.

For soil in a state of rest, the ratio of horizontal to vertical effective stress (K_i) can be estimated by a variety of means including self-boring pressuremeter tests, laboratory determination of soil suction and empirical correlations with in situ tests including static cone and dilatometer. The value of K_i depends on the type of soil, its geological history, the topography, the temporary loads which may have acted on the ground surface and changes in ground strain or ground water regime due to natural or artificial causes.

Where there has been no lateral strain within the ground, K_i can be equated with K_0 the coefficient determinable from one-dimensional consolidation and swelling tests conducted in a stress-path triaxial test using appropriate stress cycles. For normally consolidated soils, both granular and cohesive:

$$K_0 = 1 - \sin \varphi' \quad (10)$$

For overconsolidated soils, K_0 is larger and may approach the passive value at shallow depths in a heavily overconsolidated clay, (see for example Lambe and Whitman, quoting Hendron and Wroth 1975).

K_i is not used directly in earth retaining structure design because the construction process always modifies this initial value. The value of K_i is however, important in assessing the degree of deformation which will be induced as the earth pressure tends towards active or passive states. In normally consolidated soil the ground deformation necessary to mobilize the active condition will be small in relation to that required to mobilize the full passive resistance, while in heavily overconsolidated soil the required ground deformation

will be of similar magnitude. Additional ground deformation is necessary for the structure to approach a failure condition with the earth pressures moving further towards their limiting active and passive values.

Where a stressed support system is employed (e.g. ground anchorage) then the partial mobilization of the active state on the retained side is reversed during installation of the system and, in the zone of support, the effective stress ratio in the soil may pass through the original value of K_0 and tend toward the value of K_p .

3.3.3 Active earth pressures

3.3.3.1 General

Active earth pressures are generally assumed to increase linearly with increasing depth. However there may be variations from a linear relationship as a consequence, for example, of wall flexure. This can result in reduced bending moments in the structure, where the structure is flexible. Where deformations of the retaining structure are caused by transient loads, as encountered in highway structures, locked-in moments may remain after the load has been removed. These locked-in stresses will accumulate under repeated loading. This effect will limit the application of reduced bending moments in such structures.

The design soil strength, derived in accordance with 3.1.8 should be used in evaluating the active earth pressure.

3.3.3.2 Cohesionless soils

The basic formula for active pressure is applicable in the following simple situation:

- uniform cohesionless soil;
- no water pressure;
- mode of deformation such that earth pressure increases linearly with depth;
- uniformly distributed surcharge only.

In these restricted circumstances, the active pressure at depth z is given by:

$$\sigma_{an} = K_a (\gamma z + q) \quad (11)$$

where the earth pressure coefficient K_a is based on design values of soil parameters.

The total active thrust normal to the wall between ground level and depth z is then:

$$P_{an} = K_a \gamma \frac{z^2}{2} + K_a qz \quad (12)$$

If there is static ground water beneath a water table at depth z_w , then for $z > z_w$

$$\sigma_{an} = K_a \sigma_v' + u \quad (13)$$

where

$$u = \gamma_w (z - z_w) \quad (14)$$

then

$$P_{an} = \int_0^z (K_a \sigma'_v + u) dz \quad (15)$$

This equation is general; it is not limited to uniform soils or hydrostatic water pressures or to modes of deformation such that earth pressure increases linearly with depth. More than one surcharge can be accommodated, but each must be uniformly distributed.

Using the design soil strength, the value of K_a should be determined from the graphs in Annex A. K_a in these graphs is the horizontal component.

In the special case of a smooth vertical wall and horizontally retained soil surface ($\beta = 0$, $\alpha = 90^\circ$, $\delta = 0$), Rankine's formula may be used:

$$K_a = \frac{1 - \sin \varphi'}{1 + \sin \varphi'} \quad (16)$$

The design value of the angle of wall friction to be used in the graphs in Annex A should be determined in accordance with 3.2.5.

Where the ground surface is irregular the active thrust may be determined by the graphical procedure shown in Figure 5. A slip plane is chosen and the thrust on the wall is determined from the triangle of forces. The procedure is repeated with other slip planes until sufficient values have been obtained to enable the maximum thrust to be found by graphical interpolation. Not less than three planes should be used, but it is not usually necessary to have more than five. The position of the centre of pressure on the back of the wall may be taken as the point of intersection with the back of the wall of a line drawn through the centre of gravity of the wedge parallel to the slip plane of the wedge.

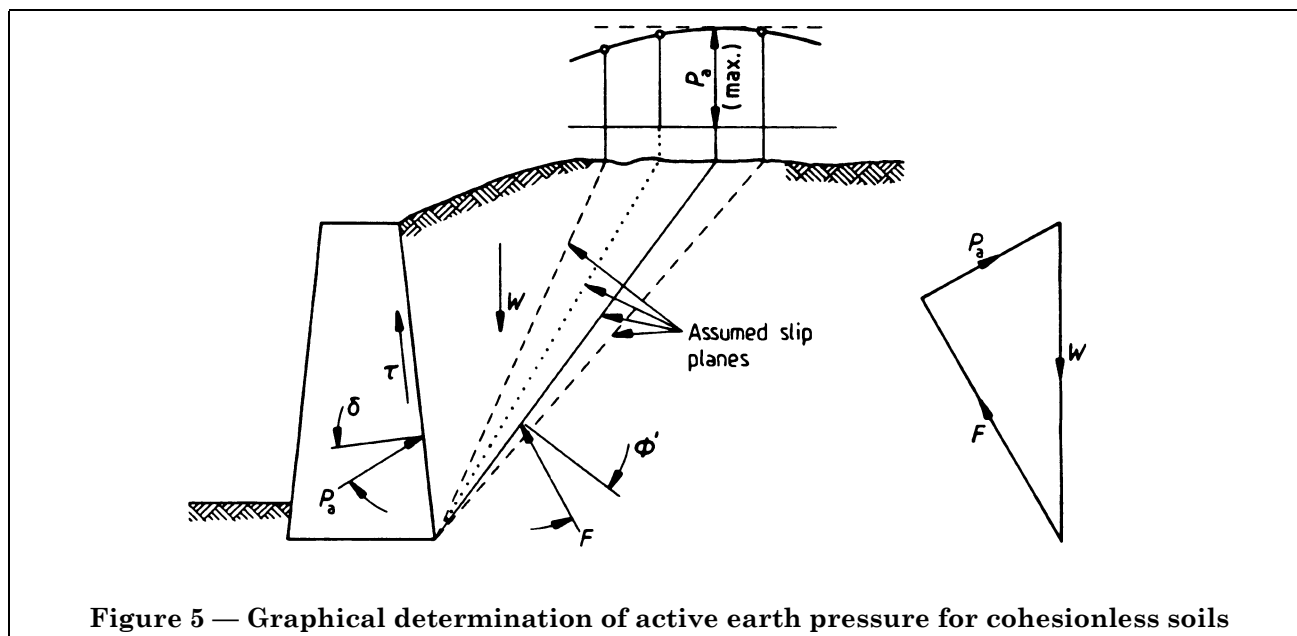


Figure 5 — Graphical determination of active earth pressure for cohesionless soils

An alternative approach is to consider the additional soil mass above a horizontal retained surface as a surcharge load, see 3.3.4. Where there is a superimposed line load for a considerable distance along and parallel to the wall, the weight per unit length of this load may be included in the force W in the diagram.

If there are several different strata of cohesionless soils behind the wall, the foregoing procedure can be used for the uppermost stratum in contact with the wall and, unless the wall is appreciably inclined from the vertical, the active pressures exerted by the lower strata can be calculated from equation 15 using an assumed average ground surface level for the estimation of the effective overburden pressure.

3.3.3.3 Clay soils

Clays in the long term behave as granular soils exhibiting friction and dilation. If a secant ϕ' value is selected, the procedures described in 3.3.3.2 will apply. If tangent parameters (c' , ϕ) are to be used, then the active thrust between ground level and depth z is given by:

$$P_{an} = \int_0^z (\sigma_{an}^* - u) dz \quad (17)$$

where

$$\sigma_{an}^* = K_a \sigma'_v - 2c' \sqrt{K_a} \quad (18)$$

Equation 17 can be applied generally for layered soils and irrespective of the mode of deformation, provided fully active earth pressures are relevant. When a clay soil is subjected to rapid shearing (see 1.3.16) then it may be assumed to behave in an undrained condition. A total stress analysis may then be carried out using design values of the undrained shear strength c_u and the undrained wall adhesion c_w :

$$P_{an} = \int_0^z (\sigma_v - K_{ac} c_u) dz \quad (19)$$

where

$$K_{ac} = 2\sqrt{\left(1 + \frac{c_w}{c_u}\right)} \quad (20)$$

and c_w is the design value of undrained wall adhesion, see 3.2.6.

In some circumstances tension cracks may develop in the retained clay soil. These may become water-filled immediately following a rain storm. If there is a tension crack care is necessary in the use of c_w . The expression $\sigma_v - K_{ac} c_u$ cannot be taken as negative. Where an increase in soil volume is possible, e.g. following deflection of the wall or shrinkage of clay during periods of dry weather, any free water will, in time, be absorbed by the clay with consequent swelling and reduction of shear strength towards the critical state value.

It is usually convenient and practical to consider the loads on retaining walls as imposed by one of two types of clay, normally consolidated clays and overconsolidated clays. These have different stress histories and therefore exhibit different pore water pressure characteristics during shear.

3.3.3.4 Normally and lightly overconsolidated clay

For normally consolidated and lightly overconsolidated clay it is satisfactory to use the design undrained shear strength c_u , because in a total stress analysis, the short-term condition is usually critical, except for excavation in front of embedded walls.

The strength of normally consolidated clays is often sensitive to disturbance. Where there is a possibility of excessive disturbance to the retained material (e.g. by construction procedures such as nearby piling operations) then the design should be based on the undrained shear strength of soil remoulded at its natural water content.

Where the possibility of a tension crack filled with water to ground level is precluded the design should be checked for the estimated equilibrium conditions using effective shear strength soil parameters in the retained soil with $c' = 0$.

3.3.3.5 Overconsolidated clay

During shearing of an overconsolidated clay negative excess pore water pressures are induced by dilation. These gradually reach equilibrium but in so doing the clay will draw in water, swell and soften.

The short-term stability of an overconsolidated clay whose mass permeability is low, of the order of 10^{-8} m/s or less, and where the consequences of failure are not severe, the pressures applied by a clay may be based on the design undrained shear strength (c_u). The possible influence of a water-filled tension crack should be taken into account.

Where the mass permeability is not of a low order and particularly where the overconsolidated clay is fissured or weathered, the design should assume that negative pore water pressures, in the retained soil, will reach equilibrium in a short time; the active earth pressure should be calculated using an effective stress analysis.

Where the ground surface is irregular or the wall is not vertical, an estimate of the actual earth pressure may be obtained by the procedure used for cohesionless soils, see 3.3.3.2. The graphical construction is shown in Figure 6. The position of the centre of pressure on the back of the wall may be taken as the point of intersection, with the back of the wall, of a line drawn through the centre of gravity of the wedge parallel to the failure surface of the wedge.

3.3.3.6 *Compaction earth pressures*

A substantial overconsolidation ratio can be imposed on a backfill by compaction. Such compaction may lead to an in situ stress ratio over the upper part of the wall which is significantly greater than the value of K_0 for a normally consolidated clay and can even lead to values nearly as high as K_p . These pressures can cause deformations and movement of a structure designed for active pressures and, where heavy compaction of a backfill is essential, account should be taken of these pressures in design. Guidance on the pressure associated with the compaction of backfill, is given by Broms (1971), Ingold (1979), Symons and Murray (1988), Clayton and Symons (1992) and TRRL publications LR766, LR946 and RR192.

3.3.3.7 *Weak rocks*

The active thrust pressures from weak rocks can vary over a wide range. Knowledge of the relation between the rock geometry, in particular the discontinuities, and the excavation geometry is essential. An analysis in terms of effective stress is generally applicable, since most weak or weathered rocks possess a relatively high mass permeability, but the influence of water pressure in discontinuities should be taken into account and particularly where fine gouge in bedding planes may lead to a perched water table. The rock should be considered to have the potential to fail either as a rock mass or on planes of discontinuity.

General guidance cannot be given because of the widely varying nature of weak rocks. Special field and analytical investigations will be required in the design of any major structure.

For minor structures it is generally adequate to take a conservative approach and treat weak rocks as being composed of interlocking granular fragments, with an effective angle of friction. The angle of friction depends upon the inter-fragment friction and upon the particle size of the grains and the mineralogy of the rock. Table 4 gives effective angles of friction for rocks which can be treated as composed of granular fragments.

For major structures an examination of exposures of the rock type should be carried out to determine in particular the stable slope angle and propensity to weathering or degradation.

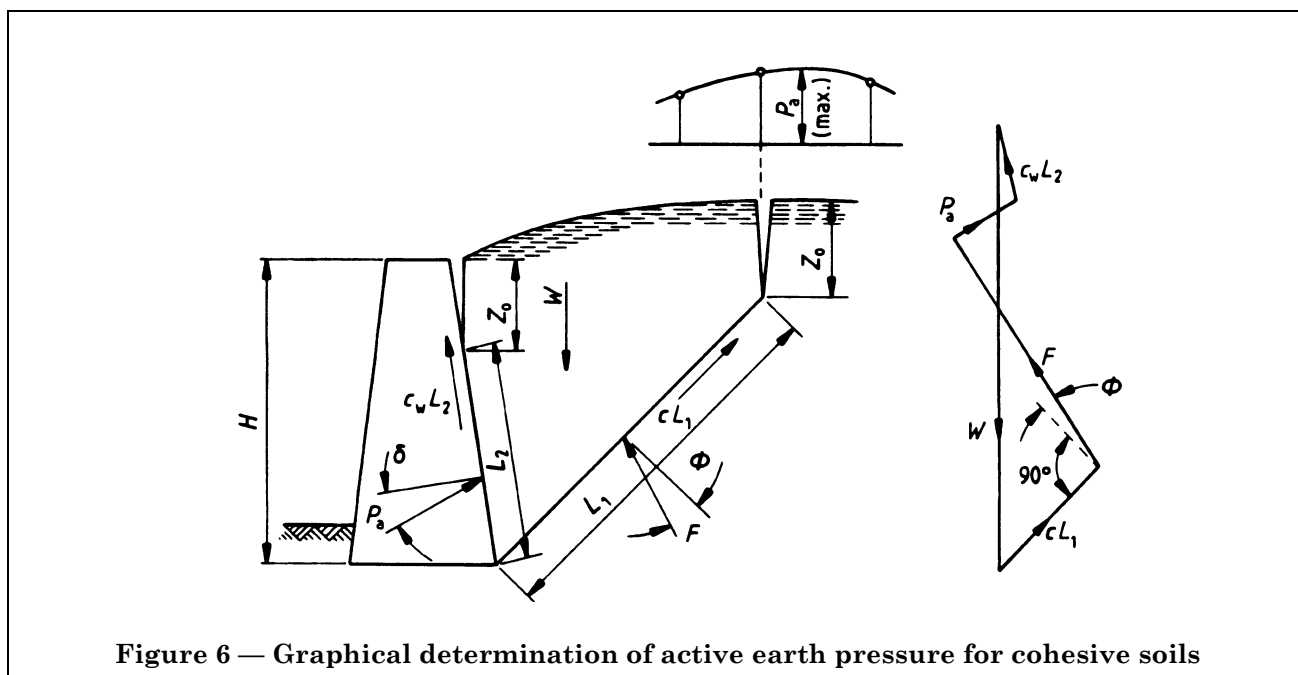


Figure 6 — Graphical determination of active earth pressure for cohesive soils

A non-zero design value of effective cohesion, c' , may be used where there is evidence from earth structures in the same geological formation in the locality, not only that it exists but that weathering or the method of wall construction does not lead to a breakdown of such cohesive properties or that the proposed retaining structure will effectively prevent such degradation. Where such information is not available it is advisable to use a conservative design with $c' = 0$.

3.3.3.8 Layered soils

Layered soils are commonly encountered in the UK. The assessment of the resulting soil pressures may be determined assuming that the soil pressures increase linearly with depth.

The soil pressure at the interface of each stratum change is calculated by modifying the common overburden pressure at the interface by the soil pressure coefficient relative to the soil immediately above and below the interface, so as to produce a change in the soil pressures on the wall at the interface of each stratum, as shown in Figure 7. In fact, such sudden jumps in lateral pressures, whilst produced by calculations, are unlikely to exist in practice.

3.3.4 Surcharge loads

3.3.4.1 Minimum surcharge

Further to 3.2.2.2 the surcharge load to be applied to the surface of the retained soil in the design of retaining walls, should have a minimum value of 10 kN/m^2 .

For walls retaining less than 3 m of earth this surcharge load may be reduced provided the designer is confident that a minimum surcharge of 10 kN/m^2 will not apply, during the life of the structure.

Additional surcharge loading should be used in the design to take account of incidental loading arising from construction plant, stacking of materials and movement of traffic both during construction and subsequently unless the nature or layout of the site precludes the need for such additional surcharge. The various surcharges imposed on a structure may be classified as:

- uniformly distributed load, consisting of a continuous load on the surface of the backing (e.g. roadways, goods stored on quays behind dock walls);
- concentrated loads (e.g. column footings);
- line loads (e.g. strip footings);
- dynamic loads (e.g. traffic, impact loads).

3.3.4.2 Uniformly distributed loads

The lateral thrust due to uniformly distributed surcharge is assumed to act at the same angle to the back of the wall as the thrust due to the earth. The surcharge pressure is considered as an initial overburden pressure at ground level.

3.3.4.3 Concentrated vertical loads and line loads

The lateral thrust induced by point or line loads acting on the retained soil should be estimated preferably by plastic analysis, using design soil strengths. The influence of line loads can be found either by analysing the plastic equilibrium of plane wedges (Pappin, 1986) or by considering safe distributions of plane stress (Bolton, 1991). The average influence of point loads, on a section of wall, can be estimated safely by considering an equivalent line load acting on that plane section, ignoring support from neighbouring sections. Elastic stress distributions may be used to estimate the vertical stress increments due to concentrated loads, but the vertical stresses should then be included in a plastic analysis using design earth pressure coefficients.

The extra thrust induced by a concentrated load may not disappear when the load is removed: more than 50 % of the effect may remain. This hysteresis is the cause of the compaction-induced stresses, see 3.3.3.6. Locked-in stresses, such as compaction stresses, will relax if the wall moves away from the compacted fill, but otherwise they should be taken into account in determining load effects on the structure. The locked-in lateral stress is a function of the greatest vertical stress previously induced at that point in the soil, Bolton (1991) offers further guidance. A reasonable estimate can be obtained by separately calculating the possible surcharge loading cases and designing for the worst case of:

- a) compaction machine in place, together with locked-in stresses in the fill beneath;
- b) future concentrated loads in place, in the absence of compaction-induced stresses.

3.3.4.4 Dynamic loads

Dynamic loads may be due to natural phenomena, for example by earthquakes, or may be man-made by traffic or machinery vibrations. This diversity precludes the listing of numerical criteria for such types of loading.

It is necessary first to assess qualitatively the vulnerability of the earth retaining structure to the dynamic loadings. This should establish whether the dynamic response is likely to be excessive in terms of either stress or motion. Factors requiring examination include resonance (or high dynamic amplification) and undue flexibility of the system. High dynamic amplification may occur when there is a close match between the forcing frequency and a dominant structural frequency, either for the whole system or locally within the system. It is undesirable to have closely matching frequencies for linking parts of the retaining structure system.

Loose sands, silty clays, saturated silts and flexible structures can all result in excessive displacements, even in the absence of high amplification effects. Where dynamic effects are not significant, estimates of peak lateral earth pressures may be made using the Mononobe-Okabe approach, see Seed and Whitman (1970). Flexible structures may incur excessive non-structural damage because of lateral drift during an earthquake. Where a retaining wall, sited on soft or loose soil, may be subjected to significant ground vibrations, then the structure should be provided with a stiff foundation so as to prevent excessive differential movement in the upper part of the structure.

Silts and loose- to medium-dense sands may undergo liquefaction during an earthquake. The depth of potential liquefaction should be assessed for the earthquake conditions appropriate to the site. It may be necessary to carry the foundations of the retaining wall below the liquefaction zone, or compact the soil within the zone. See Seed H.B. et al (1983) and Ishihara (1993). In dense cohesionless soils or overconsolidated clay soils, severe problems with dynamic loads occur only with severe machine vibrations or earthquakes.

Numerical design criteria are reasonably well developed for earthquakes and are included in the codes of practice applicable in seismic regions such as California and New Zealand, see *Earthquake resistant regulations — A world list*, IAEE (1992).

Definitive criteria for the limitation of damage to structures from vibrations from other sources, such as blast vibrations and ground motions induced by explosions and forced vibrations from rotating and impact machines are difficult to develop, although empirical and semi-empirical approaches are available. See Institution of Structural Engineers Structure-soil interaction (1977); Dowrick D.J. (1977); Blasting practice (1972); Corbett B.O. (1961); Richart F.E. (1960); Alpan I. (1961); Barkan D.D. (1962).

3.3.4.5 Loading resulting from climatic changes

The loading effects of climatic variations should be considered, including:

- thermal expansion or contraction, which may cause significant changes in strut loads in braced excavations and tie loads in anchored walls;
- expansion of the ground due to freezing, which may increase the earth pressure on the structure;
- moisture content increase in shrinkable clays, which may cause swelling pressures, while moisture content reductions may cause tension cracks which may lead to a reduction in the stability of the soil mass;
- abnormal temperatures in the soil caused, for example by artificial climates, such as in boiler houses or cold stores. These may increase the loads to be carried by the retaining wall or may lead to undesirable deformations in either the retaining wall or the ground or both.

Mass gravity walls, reinforced concrete walls and reinforced masonry walls may be damaged by movement of the soil on which they rest. Movement may be caused by frost heave, for example of a chalk soil, or by changes in moisture content where the soil is a shrinkable clay. The foundations of such walls should be designed in accordance with 3.2.2, but should be carried down to a sufficient depth to reduce the relative movement of the foundation to a satisfactory value. See also BS 8004.

3.3.5 Water pressure

3.3.5.1 General

The determination of the water pressure on an earth retaining structure is important. The assessment of the design level of the water table necessitates taking into account natural variations in the water table, the provision of effective drainage and the drainage characteristics of any fill and drainage layers provided. The influence of rainstorms on the seepage pattern behind the structure should be considered, particularly for sandy silts and silts. See Terzaghi 1943; Terzaghi, I.C.E. Proceedings 1939, 12, 106-141; Terzaghi and Peck 1967.

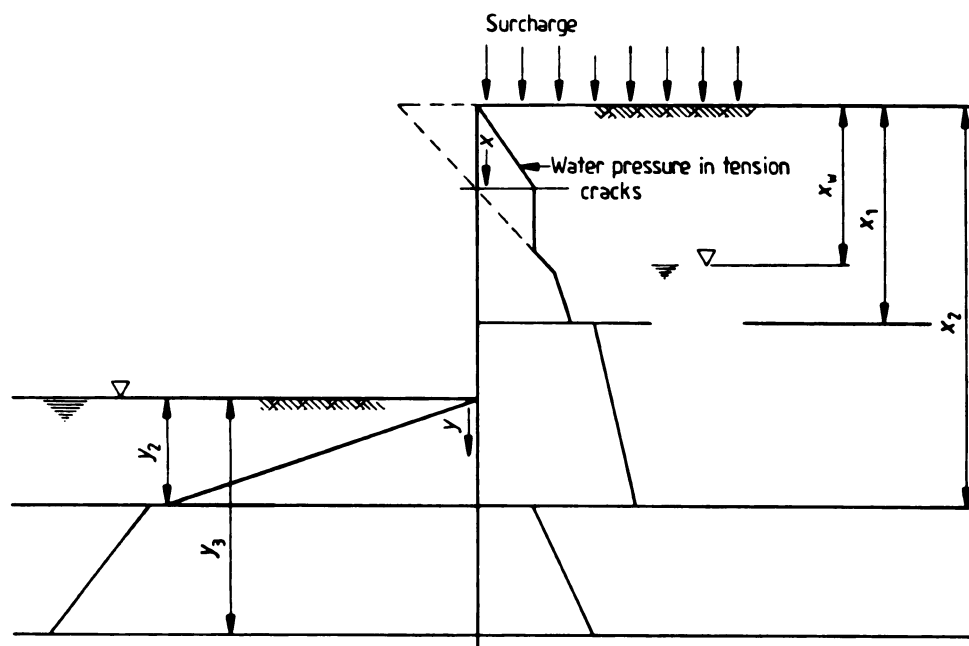


Figure 7 — Construction of earth pressure diagrams for earth retaining structures in multi-layered soil

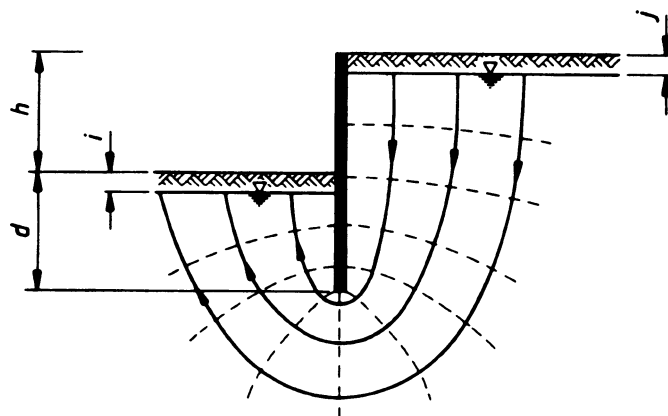


Figure 8 — Flow net determination of pore water pressure

3.3.5.2 Water table and seepage forces

If the equilibrium level of the water table is well defined and measures are taken to prevent it changing during heavy rain or flood, the design water pressures can be calculated from the position of the equilibrium water table, making due allowance for possible seasonal variations, otherwise the most adverse water pressure conditions that can be anticipated should be used in design. In clay soils the equilibrium water table can be determined only from piezometric readings taken over an adequate length of time. The water pressure due to the temporary filling of cracks in clay soils should be allowed for in “undrained” analyses in terms of total stress. The water pressure to be used in effective stress soil analyses should be calculated from the ground water regime in the vicinity of the structure. In weak rocks it will be necessary to measure the water pressures on discontinuity surfaces.

Account should be taken of seepage flow occurring around the structure where a difference in water pressures is likely to exist on opposite sides. The distribution of pore water pressures will not then be hydrostatic and should be determined from a flow net, which adequately represents the hydraulic and permeability conditions in the vicinity of the structure, see Figure 8. Where layers of markedly different permeability exist the water levels relevant to each permeable stratum should be taken into account.

Alternatively the pore water pressure distribution can be calculated based on the simplifying assumption that the hydraulic head varies linearly down the back and up the front of the wall (Burland et al, 1981). Figure 9 shows such a distribution adjacent to a wall in a soil of isotropic permeability. A linear dissipation of seepage pressure will give reasonably reliable results for retaining walls where the seepage flow upwards through the passive zone in front of the wall is free to dissipate laterally as well as vertically. This assumption should not be made for cofferdams where the width of the cofferdam is less than four times the differential hydrostatic head. The concentration of seepage flow from the opposite sides of the cofferdam in the relatively restricted passive zone in the bottom of the cofferdam makes the assumption unsafe; for such cofferdams a flow net should be constructed.

On the basis of Figure 9:

$$P_{wt} = \frac{2(h + d - j)(d - i) \gamma_w}{(2d + h - i - j)} \quad (21)$$

3.3.5.3 Drainage

The design should take account of the influence of the drainage conditions behind and through the wall. Allowance should be made for possible changes in the ground water levels due to temporary or permanent modification which may be made to the water conditions by the earth retaining structure itself, both permanently and during construction.