GEOGUIDE 1

GUIDE TO RETAINING WALL DESIGN

Geotechnical Control Office
Engineering Development Department
HONG KONG

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Civil Engineering Services Department
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FOREWORD

This Guide to Retaining Wall Design is the first Guide to be produced by the Geotechnical Control Office. It will be found useful to those engaged upon the design and construction of retaining walls and other earth retaining structures in Hong Kong and, to a lesser extent, elsewhere. This Guide should best be read in conjunction with the Geotechnical Manual for Slopes (Geotechnical Control Office, 1979), to which extensive reference is made.

The Guide has been modelled largely on the Retaining Wall Design Notes published by the Ministry of Works and Development, New Zealand (1973), and the extensive use of that document is acknowledged. Many parts of that document, however, have been considerably revised and modified to make them more specifically applicable to Hong Kong conditions. In this regard, it should be noted that the emphasis in the Guide is on design methods which are appropriate to the residual soils prevalent in Hong Kong.

Many staff members of the Geotechnical Control Office have contributed in some way to the preparation of this Guide, but the main contributions were made by Mr. J.C. Rutledge, Mr. J.C. Shelton, and Mr. G.E. Powell. Responsibility for the statements made in this document, however, lie with the Geotechnical Control Office.

It is hoped that practitioners will feel free to comment on the content of this Guide to Retaining Wall Design, so that additions and improvements can be made to future editions.

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CONTENTS

CHAPTER 1

INTRODUCTION

1.1 SCOPE OF THIS DESIGN GUIDE

These notes are intended as a Guide for use in the estimation of earth pressure forces, and for the design and construction of retaining walls and other earth retaining structures in Hong Kong. Recommended methods are given for most aspects of design, except for reinforced concrete, where guidance is given on only a few special points. Throughout the Guide, reference is made to relevant textbooks, Codes and published papers, and the reader should consult those original documents for more detailed coverage of particular aspects of the subject matter.

It is important to remember that engineering judgement should always be exercised in applying the theories and design methods given in the Guide. In particular, the practitioner must be aware of the limitations on the basic assumptions employed in a particularly theoretical or computational method.

The material contained in this Guide has been arranged so as to provide the maximum convenience to the user. In this respect, it should be noted that :

(a) Full references to the published material cited in the text are given in alphabetical order on pages 77 to 81 .

(b) A list of symbols used in the text and figures is given on pages 83 to 86 .

(c) A list of tables contained in the text is given on page 87 .

(d) A list of figures is given on pages 89 to 90 .

(e) The 32 figures referred to in the text are collected together on pages 91 to 153 at the back of this Guide.

1.2 RETAINING WALL DESIGN PRINCIPLES

1.2.1 Free-standing Retaining Walls

In the design of free-standing retaining walls, the following aspects need to be investigated :

(a) the stability of the soil around the wall,

(b) the stability of the retaining wall itself,

(c) the structural strength of the wall; and

(d) damage to adjacent structures due to wall construction.

The magnitude of the earth pressure which will be exerted on a wall is dependent on the amount of movement that the wall undergoes.

It is usual to assume for free-standing retaining walls that sufficient outward movement occurs to allow *active* (minimum) earth pressures to develop. The designer must ensure that sufficient movement can take place without affecting the serviceability or appearance of the wall.

Where it is not possible for the required outward movement to occur, for instance due to wall or foundation rigidity, higher pressures will develop and the wall must be designed for these. Further guidance on this matter is given in Section 3.2.

1.2.2 Other Retaining Structures

If a structure prevents outward movement of the soil, the wall will usually be subject to static earth pressures greater than active. This occurs where a wall retaining earth is part of a more extensive structure, such a basement wall in a building or an abutment wall of a portal structure. It also occurs when the wall is connected to another structure, such as a bridge abutment connected to the superstructure.

1.3 LOAD CASES

1.3.1 Basic Loadings

The basic pressure loading to be considered for design is :

Normal loading = static earth pressure + water pressure + pressure due to live loads or surcharge.

In general, the resulting design pressure for earth retaining structures should not be less than the pressure due to a fluid of unit weight $5kN/m^3$.

It should be noted that, in accordance with Chapter 4 of Volume V of the Civil Engineering Manual, (Public Works Department, Hong Kong, 1977), highway structures should be designed to withstand seismic forces corresponding to ground accelerations of 0.07g. It may be assumed that conventional cantilever, counterfort and gravity retaining walls of normal proportions and detailing will have adequate resistance to withstand such an acceleration. Further guidance may be obtained from the paper by Seed & Whitman (1970).

1.3.2 *Other Considerations*

The possible occurrence of other design cases, or variations of the one above, caused by construction sequence or future development of surrounding areas should also be considered. For instance, additional surcharges may need to be considered and allowance made for any possible future removal of ground in front of the wall in connection with services, particularly if the passive resistance of this material is included in the stability calculations. The effect of excavation on the wall bearing capacity may also need to be considered.

For the determination of earth pressures, it is usual to consider a unit length of the cross-section of the wall and retained soil. A unit length is also used in the structural design of cantilever walls and other walls with a uniform cross-section.

1.4 *SUPPORT OF EXISTING FILL SLOPES*

Fill slopes constructed in Hong Kong prior to 1977 are likely to have been end tipped or inadequately compacted. Such slopes may be subject to liquefaction under conditions of heavy rainfall, vibration or leakage from services, and resulting mud flows may have serious consequences. Undercutting of the slope toe, for retaining wall construction, will increase the risk of failure.

The state of existing fill slopes should be established by insitu density testing in trial pits, in conjunction with GCO probe (Modified Mackintosh probe) testing, to establish the insitu dry density and extent of the loose fill. Where appropriate, remedial measures should be carried out to ensure that failure of the fill slope cannot occur by liquefaction.

CHAPTER 2

SOIL PROPERTIES

2.1 GENERAL

For all walls higher than 5 metres, especially those with sloping backfill, the soil properties of the natural ground and backfill should be estimated in advance of design from tests on samples of the materials involved. In addition, special attention should be paid to the determination of ground water levels, particularly with respect to maximum probable values.

For less important walls, an estimation of the soil properties may be made from previous tests on similar materials. A careful visual examination of the materials, particularly that at the proposed foundation level, should be made and index tests carried out to ensure that the assumed material type is correct.

2.2 SELECTION AND USE OF BACKFILL

The ideal backfill for a minimum section wall is a free draining granular material of high shearing strength. However, the final choice of material should be based on the costs and availability of such materials balanced against the cost of more expensive walls.

In general, the use of fine-grained clayey backfills is not recommended. Clays are subject to seasonal variations in moisture content and consequent swelling and shrinkage. This effect may lead to an increase in pressure against a wall when these soils are used as backfill. Due to consolidation, long term settlement problems are considerably greater than with cohesionless materials.

For cohesive backfills, special attention must be paid to the provision of drainage to prevent the build-up of water pressure. Free draining cohesionless materials may not require the same amount of attention in this respect. They may still require protection by properly designed filter layers.

The wall deflection required to produce the active state in cohesive materials with a significant clay content may be up to 10 times greater than for cohesionless materials. This, together with the fact that the former generally have lower values of shearing strength, means that the amount of shear strength mobilised for any given wall movement is considerably lower for cohesive materials than for cohesionless materials. The corresponding earth pressure on the active side for a particular wall movement will therefore be higher if cohesive soil is used for backfill.

In Hong Kong, backfill for retaining walls usually comprises selected decomposed granite or decomposed volcanic rock. This material is in general suitable for backfill provided that it is properly compacted and drainage measures are carefully designed and properly installed to prevent build-up of water pressure.

Rock fill is a very suitable material for use as a backfill to retaining walls and consideration should be given to its use when available. In general, the rockfill should be well graded and have a nominal maximum size of 200mm. A well-graded densely compacted rockfill should not have more than about 2% finer than 75μm if it is to remain free-draining.

Movement of soil, due to seepage, into the rockfill needs to be prevented. This may require the provision of properly designed filter layers between the soil and the rockfill.

It is essential to specify and supervise the placing of backfill to ensure that its strength and unit weight properties agree with the design assumptions both for lateral earth pressure and dead weight calculations. In this regard, it is particularly important to ensure that the backfill behind a wall and on a slope is properly compacted. The backfill should normally be compacted in thin layers using light compaction plant for the reasons outlined in Section 3.10.

The active earth pressure is substantially reduced, particularly for a steeply sloping backfill, if the failure plane occurs in a material with a high angle of shearing resistance. In some circumstances, it may be economical to replace weaker material so that the above situation occurs.

2.3 UNIT WEIGHT

The unit weight of soil depends on the specific gravity of the solid particles and the proportions of solid, air and water in the soil. The average specific gravities of Hong Kong soils in general lie between 2.65 and 2.70, although values outside this range are found. The proportion of the total volume that is made up of this solid material is dependent on the degree of compaction or consolidation.

As estimate of the unit weight of backfill material to be used behind a retaining structure may be obtained from standard laboratory compaction tests on samples of the material or from records of field testing. The unit weight chosen must correspond to the compaction and moisture conditions that will apply in the actual field situation.

The unit weight of natural soil should be obtained from undisturbed samples kept at the field moisture content and volume. For initial design purposes, dry densities in the range 1750 to 1850kg/m^3 may be assumed for all soils compacted near optimum moisture content.

2.4 EFFECTIVE STRESS AND PORE PRESSURE

An effective stress may be considered to be the stress transmitted through the points of contact between the solid particles of the soil. It is this stress that determines the shearing resistance of the soil. The effective stress, σ', at any point in a saturated soil mass may be obtained by subtracting the pressure transmitted by water in the voids, u, (pore water pressure) from the total stress, σ, thus :

$$\sigma' = \sigma - u \qquad .....(1)$$

An increased pore water pressure gives a reduced effective stress and therefore a reduced soil shearing resistance. This leads to an increased force against a wall in the active case. Conversely, an increase in the negative pore pressure (i.e. a pore suction) gives an increased shearing resistance and reduces the force against a wall in the active case.

Positive pore water pressure results from a number of factors, the most important in Hong Kong being static water pressure, seepage of groundwater or rainfall and seepage from other sources, such as burst or leaking water supply mains. In some soils, shock or vibration can cause transient increases in pore pressure. In low permeability soils, changes in pore water pressure can result from changes in total stress due to ground loading, dewatering or excavation. These pore pressures dissipate with time, but may need to be considered in design. Pore water pressures due to static water pressure and seepage of water are covered in Chapter 5.

Negative pore pressures are present in many partially saturated soils in Hong Kong. Soil suction may be destroyed by surface infiltration or seepage, and, until more information on its magnitude, distribution and behaviour becomes available, its effect on the shear resistance of the soil should not be used in retaining wall design.

2.5 SHEAR STRENGTH

In all earth pressure problems the magnitude of earth pressure on a particular structure is a function of the shear strength of the soil. The shear strength is not a unique property of the material but depends upon the conditions to which the soil is subjected when it is sheared. Where a retaining structure supports a saturated clay soil of low permeability, the *undrained* shear strength can be used to calculate the earth pressure for *short-term stability*, because the shear strength of such soil does not change as it is sheared *quickly* (i.e. the excess pore water pressures cannot dissipate during shear). However, Hong Kong residual soils are not saturated and they have relatively high permeabilities. The water content, therefore, can change quite rapidly, with a consequent change in pore pressure and, hence, with a change in shear strength. It is necessary, therefore, for earth pressures in Hong Kong soils to be calculated from shear strengths expressed in terms of *effective stresses*.

The shear strength of a soil is proportional to the effective stress which acts on the failure plane. Laboratory tests can be carried out to establish the relationship between strength, S, effective stress, σ', and this is commonly termed the *strength envelope*. The envelope will generally be curved, but portions of the curve can be approximated by the relationship :

$$S = c' + \sigma' \tan \phi' \qquad .....(2)$$

where c' and ϕ' are termed the *effective strength parameters*. These parameters should be used for earth pressure calculations in Hong Kong soils. It is important to note that the design strength parameters must be those determined in the laboratory for the range of effective stress which is appropriate to the field situation.

Laboratory triaxial tests or shear box tests are commonly used to determine the strength envelope of a soil. Guidance on these methods of strength measurement and on the interpretation of test results can be obtained from Lambe & Whitman (1969) and from the Geotechnical Manual for Slopes (Geotechnical Control Office, 1979).

The following two types of triaxial tests can be used :

(a) Consolidated-undrained tests with pore pressure measurement (CU tests) carried out on specimens saturated using back pressure.

(b) Drained tests (CD tests) on saturated specimens.

Shear box tests are simpler to carry out than triaxial tests but only drained tests can be conducted on Hong Kong residual soils. Care should be taken to ensure that test specimens are soaked for a sufficient period prior to testing and that submergence is maintained during shear.

The shear strength of a *backfill* material depends on its density, and laboratory strength tests should be carried out on specimens compacted to the density that will exist insitu. Where inadequate shear strength information is available at the time of preliminary design, the following values may be taken as guidance to the properties of compacted Hong Kong soils :

For decomposed volcanics, $c' = 0$, $\phi' = 35^{o}$, $\gamma_d = 1750 kg/m^3$
For decomposed granite, $c' = 0$, $\phi' = 39^{o}$, $\gamma_d = 1850 kg/m^3$

2.6 *BASE SHEAR RESISTANCE*

The amount of shearing resistance available between the base of the wall and the foundation soil will depend on the nature of materials used to construct the base and on the construction technique.

The base friction to be used for walls without a key is $2\phi'/3$. When it can be ensured that the excavation of the base will be carried out in the dry season and that disturbance and deterioration of the subsoil is prevented by construction of an adequate blinding layer *immediately* after foundation exposure, and where there is professional site supervision it may be possible to justify a higher proportion of ϕ'. Values of base adhesion, c_b, used in calculations should be taken as zero unless specific data proving otherwise are available.

If a shallow base key is used, the failure plane will generally be through the foundation soil (see Figure 1) and, therefore, the shearing resistance may be taken as that of the soil ($\delta_b = \phi'$ and $c_b = c'$). Further comment on this is given in Section 6.2.

2.7 *WALL FRICTION*

The magnitude and direction of the developed wall friction depends on the relative movement between the wall and the soil. In the *active* case, the maximum value of wall friction develops only when the soil wedge moves significantly downwards relative to the rear face of the wall. In some cases, wall friction cannot develop. These include cases where the wall moves down with the soil, such as a gravity wall on a yielding foundation or a sheet pile wall with inclined anchors, and cases where the failure surface forms away from the wall, such as in cantilever and counterfort walls (Figure 9).

The maximum values of wall friction may be taken as follows :

Timber, steel, precast concrete, δ max. $= \frac{\phi'}{2}$

Cast in-situ concrete, δ max. $= \frac{2\phi'}{3}$

In general, the effect of wall friction is to reduce active pressure. The effect is small and often disregarded.

The effect of wall friction on *passive pressures* is large (see Section 3).

Considerable structural movements may be necessary, however, to mobilise maximum wall friction, for which the soil in the passive zone needs to move upwards relative to the structure. Generally, maximum wall friction is only mobilised where the wall tends to move downwards, for example, if a wall is founded on compressible soil, or for sheet piled walls with inclined tensioned members. Some guidance on the proportion of maximum wall friction which may develop in various cases is given in Table 1; the residual soils of Hong Kong might be taken to be covered by these data.

Table 1. Indicative Proportions of Maximum Wall Friction Developed (Granular Soils - Passive Case) (Rowe & Peaker, 1965)

Structure Type	Developed Proportion of Maximum Wall Friction	
	Loose	Dense
Gravity or free standing walls with horizontal movement. Sheet pile walls bearing on hard stratum	0	0.5
Sheet walls with freedom to move downwards under active forces or inclined anchor loads	1.0	1.0
Walls where passive soil may settle under external loads	0	0
Anchorage blocks, etc. which have freedom to move upwards on mobilization of passive pressure.	0	0

Where a wall will be subjected to significant vibration, wall friction should not be included.

2.8 COEFFICIENT OF SUBGRADE REACTION

In the design of footings and wall foundations, the simplified concept of subgrade can be used to determine wall rotations. This concept is based on the assumption that the settlement, Δ, of any element of a loaded area is entirely independent of the load on the adjoining elements. It is further assumed that there is a constant ratio, K_s, between the intensity, q, of the foundation pressure on the element and the corresponding settlement, Δ, given by :

$$K_s = \frac{q}{\Delta} \qquad \text{.....(3)}$$

The foundation pressure, q, is called the *subgrade reaction*, and the ratio, K_s, is known as the *coefficient of subgrade reaction*.

2.9 PERMEABILITY

Lumb (1975) has presented values of insitu permeability for Hong Kong residual soils. These are summarized in Table 2 and may be used to give some guidance. It should be noted, however, that other sources of permeability test results have revealed values well outside these ranges and, when the particular value is critical in design, permeability tests should be carried out. In this regard, Lumb has noted and subsequent investigations have confirmed that laboratory measurements of permeability of decomposed volcanics based on small intact tube specimens are two orders of magnitude lower than values obtained from field tests. Lumb attributed the difference to the influence of joints. Laboratory results, therefore, should be treated with caution.

Table 2 Insitu Permeabilities of Hong Kong Residual Soils

Soil	Permeability (m/s)
Decomposed granites	3×10^{-7} to 4×10^{-5}
Decomposed volcanics	2×10^{-9} to 4×10^{-7}

Procedures for determination of insitu permeability are given in Chapter 2 of the Geotechnical Manual for Slopes.

The permeabilities of granular backfill materials, in relation to particle grading, are given in greater detail in Figure 20.

CHAPTER 3

EARTH PRESSURES

3.1 STATES OF STRESS

The stresses at any point within a soil mass may be represented on the Mohr co-ordinate system in terms of shear stress, τ, and effective normal stress, σ'. In this system, the shearing strength of the soil at various effective normal stresses gives an envelope of the combinations of shear and normal stress. When the maximum shearing strength is fully mobilised along a surface within a soil mass, a failure condition known as a *state of plastic equilibrium* is reached. Reference should be made to Section 3.9 in the Geotechnical Manual for Slopes for the plotting of stresses and use of the system.

Where the combinations of shear and normal stress within a soil mass all lie below the limiting envelope, the soil is in a *state of elastic equilibrium* (Terzaghi & Peck, 1967). A special condition of elastic equilibrium is the 'at-rest' state, where the soil is prevented from expanding or compressing laterally with changes in the vertical stress. Any lateral strain in the soil alters its horizontal stress condition. Depending on the strain involved, the final horizontal stress can lie anywhere between two limiting (failure) conditions, known as the *active* and *passive* failure states.

3.2 AMOUNT AND TYPE OF WALL MOVEMENT

The earth pressure which acts on an earth retaining structure is strongly dependent on the lateral deformations which occur in the soil. Hence, unless the deformation conditions can be estimated with reasonable accuracy, rational prediction of the magnitude and distribution of earth pressure in the structure is not possible.

The minimum *active* pressure which can be exerted against a wall occurs when the wall moves sufficiently far outwards for the soil behind the wall to expand laterally and reach a state of plastic equilibrium. Similarly, the maximum *passive* pressure occurs when the wall movement is towards the soil. The amount of movement necessary to reach these failure conditions is dependent primarily on the type of backfill material. Some guidance on these movements is given in Table 3.

Table 3 Wall Displacements Required to Develop Active and Passive Earth Pressures (Wu, 1975)

Soil	State of Stress	Type of Movement	Necessary Displacement
Sand	Active	Parallel to wall	0.001H
	Active	Rotation about base	0.001H
	Passive	Parallel to wall	0.05 H
	Passive	Rotation about base	> 0.1 H
Clay	Active	Parallel to wall	0.004H
	Active	Rotation about base	0.004H
	Passive		-

For wall displacements less than those necessary to produce the failure conditions, the magnitude of the pressure on the wall lies between the extreme values. Figure 2 shows the typical variation in wall pressure with movement.

For a rigid wall free to translate or rotate about its base, the active or passive condition occurs if sufficient movement can take place, and the pressure distribution remains approximately triangular for uniform sloping ground (Figure 3(a)).

In some cases, rotation about the base or translation of a free standing wall may be limited by a strong foundation or by some other restraint such as occurs in bridge abutments or walls framed-in with the superstructure. Structural deformations for walls are not usually sufficient alone to allow development of active pressures, and hence the wall is subject to pressures near those for at-rest conditions (Figure 3(b)) or those caused by compaction (Section 3.10). Thermal expansion of the structure may force the retaining wall into the soil producing higher earth pressures (Broms & Ingelson 1971).

When the top of the wall is restrained while the base can rotate, not all of the retained soil passes into the active state. Limited movement near the top of the wall, together with arching, leads to an approximately parabolic pressure distribution, with a corresponding force on the wall 10 to 15% higher than the force for the active condition (Figure 3(c)).

An approximate calculation of the magnitude of the tilting movement that results from the backfilling of a retaining wall may be obtained by simulating the foundation soil as a series of springs with an appropriate coefficient of subgrade reaction (see Section 2.8). The base rotation, θ_b, (radians) is then given by :

$$\theta_b = \frac{12Ve_b}{K_s L B^3} \qquad \left(\text{for } e_b \leq \frac{B}{6} \right) \qquad \text{.....(4)}$$

where V is the vertical component of the foundation bearing pressure,
e_b is the eccentricity of the load on the base
L, B are length and breadth of the base, respectively,
and K_s is the coefficient of subgrade reaction (Eqn. 3).

Flexible walls allow complex deformations and redistribution of loads. Loads vary on individual supports depending largely on the stiffness characteristics of the supports themselves.

Strutted walls have approximate final deformation patterns as shown in Figure 3(d). This profile is strongly influenced by construction details and procedures, and so pressure envelopes covering possible actual pressure distributions are used for retained heights of greater than 6 metres. (Figure 24).

Compaction of the backfill can produce pressures higher than active. This is discussed in Sections 3.10 & 3.11.

3.3 RANKINE EARTH PRESSURE THEORY

Rankine's equations give the earth pressure on a vertical plane which is sometimes called the *virtual back* of the wall. The earth pressure on the vertical plane acts in a direction parallel to the ground surface and is directly proportional to the vertical distance below the ground surface. The pressure distribution is triangular.

Rankine's conditions are theoretically only applicable to retaining walls when the wall does not interfere with the formation of any part of the failure wedges that form on either side of the vertical plane, as shown in Figures 1 & 9 or where an imposed boundary produces the conditions of stress that would exist in the uninterrupted soil wedges. These conditions require that the angle of wall friction is equal to the backfill slope ($\delta = \omega$).

Passive calculations using Rankine are not recommended, since the direction of wall friction will be incorrect and an underestimation of passive resistance will result.

3.4 COULOMB EARTH PRESSURE THEORY

Coulomb theory assumes that a wedge of soil bounded by a planar failure surface slides on the back of the wall. Hence shearing resistance is mobilised on both back of the wall and the failure surface. The resultant pressure can be calculated directly for a range of wall frictions, slopes of wall and backfill slopes.

Where the wall friction is at angles other than the backfill slope angle the equations are an approximation due to the curved nature of the actual failure surface and the fact that static equilibrium is not always satisfied. The error is slightly on the unsafe side for the active case, and more serious for the passive case. For simple geometries, the charted values of K_a given in Figures 4 & 5 (Caquot & Kerisel, 1948) may be used; these were obtained for the more accurate failure mechanism involving curved failure surfaces.

3.5 TRIAL WEDGE METHOD

Difficulties arise in the use of charts or equations where the ground surface is irregular, where the backfill possesses some cohesion, where water pressures exist in the backfill or where the backfill comprises more than one soil type.

The simplest approach for earth pressure determination in these cases is to use a graphical procedure making the assumption of planar failure surfaces based on Coulomb theory. The method is very powerful in that solutions to most active pressure problems are possible and it also has the advantage that the designer can see the solution developing and gains an appreciation of the significance of the contributory factors involved. There are, however, certain limitations in the use of the method for the determination of passive pressures. The procedure is known as the Trial Wedge Method or the Coulomb Wedge Method.

The method is outlined in Figures 6,7 & 8. The backfill is divided into wedges by selecting planes through the heel of the wall. The forces acting on each of these wedges are combined in a force polygon so that the magnitude of the resultant earth pressure can be obtained. A force polygon is constructed, although the forces acting on the wedge are in general not in moment equilibrium. This method is therefore an approximation with the same assumptions as the equations for Coulomb's conditions, and, for a ground surface with a uniform slope, gives the same result. When the wall friction corresponds to that implied by the Rankine case, the value of earth pressure obtained from the Trial Wedge Method is equal to that obtained from Rankine's equation.

Figure 8 shows the general method of dealing with active pressures in more complex ground conditions using the Trial Wedge Method. It should be noted that the method can be rather laborious in these situations.

The adhesion of the soil to the back of the wall in cohesive soils is usually neglected, since its value is difficult to determine and the simplification is conservative. For the active case, the maximum value of the earth pressure calculated for the various wedges is required. This is obtained by interpolating between the calculated values (see Figure 6). For the passive case, the required minimum value is similarly obtained. The direction of the resultant earth pressure in the force polygons should be obtained by considering the direction of the relative movement between the wall and soil. For cases where this force acts parallel to the ground surface, a substitute constant slope should be used for soil both with and without cohesion (Figure 10).

Theoretically, in cohesive soils, tension exists to a depth Y_o below both horizontal and sloping ground surfaces.

$$Y_o = \frac{2c}{\gamma} \tan \left(45^o + \frac{\phi}{2}\right) \qquad \text{.....(5)}$$

where c is the cohesion of the soil in terms of total stress,

γ is the bulk unit weight of the soil, and

ϕ is the angle of shearing resistance of the soil in terms of total stress. Shear strength parameters in terms of effective stress (c' & ϕ') may be used in equation (5).

Vertical tension cracks will develop in this zone since soil cannot sustain tension and will become water filled. One of these cracks will extend down to the failure surface and so reduce the length on which cohesion acts. The effect of this, together with the slightly smaller wedge weight, is the same as neglecting the reduction in total pressure provided by the tension zone according to the Rankine and Coulomb equations. Figure 7 shows the wedge analysis for this case.

For an irregular ground surface the pressure distribution against the wall is not triangular. However, if the ground does not depart significantly from a plane surface, a linear pressure distribution may be assumed, and the construction given in Figure 11 used to determine the point of application of the active force. A more accurate method is given in Figure 12. The latter should be used when there are abrupt changes in the ground surface, or there are non-uniform surcharges involved.

3.6 PASSIVE EARTH PRESSURES

The shape of the failure surface for passive failure is curved, more strongly when wall friction is present. Both Coulomb and the Trial Wedge theories assume plane failure surfaces and lead to substantial errors in calculated values of passive resistance.

Methods using curved failure surfaces, such as log-spiral and circular, may be used without introduction of significant error. Caquot & Kerisel(1948) have presented charts for simple geometries (Figures 4 & 5) based on a combination of log-spiral and a plane. For more complex geometries, passive pressure may be calculated using the circular arc method outlined in Figure 13. This method is quite laborious for even relatively simple conditions.

The trial wedge method may be used to determine passive resistance. However, serious overestimation of the passive pressure results when the angle of wall friction δ is greater than $2\phi'/3$ (Morgenstern & Eisenstein, 1970). Care should be taken then to ensure that δ is not overestimated, as the error is on the unsafe side, and the trial wedge method should not be used for the determination of passive pressures when $\delta > \phi'/3$.

3.7 EARTH PRESSURES FOR SMALL WALL DEFLECTIONS

For certain wall types, such as propped cantilevers and anchored diaphragm walls, only small wall movements occur and elastic conditions apply.

Where no lateral movement takes place from the insitu condition, the 'at-rest' earth pressure applies. For the case of a vertical wall and a horizontal ground surface, it has been shown empirically by Jaky (1944) that the coefficient of 'at-rest' earth pressure, K_o, for normally consolidated materials may be taken as :

$$K_o = 1 - \sin \phi' \qquad .....(6)$$

where ϕ' is the angle of shearing resistance of the soil in terms of effective stress.

Because of the lack of data on the values of K_o for Hong Kong soils, values adopted for design should not be less than 0.5 even for soils with high friction angles. It should be noted that, in some situations, values much higher than $K_o = 0.5$ may be found.

For a sloping ground surface, K_o varies from that given by equation (6). The Danish Code (Danish Geotechnical Institute, 1978) suggests for a vertical wall and ground sloping at an angle, ω, that the 'at-rest' earth pressure coefficient is K_o $(1 + \sin \omega)$. For other wall angles and backfill slopes, it may assumed that the at-rest pressure coefficient varies proportionally to the 'active' earth pressure coefficient, K_a. 'At-rest' earth pressures, except for over-consolidated soils, may be assumed to increase linearly with depth from zero at the ground surface. The total at-rest earth pressure force is given by $P_o = \frac{1}{2} K_o \gamma H^2$. This acts at H/3 from the base of the wall or from the bottom of the key for walls with keys.

In cohesionless soils, full 'at-rest' earth pressures occur only with the most rigidly supported walls (see Section 3.10). In highly plastic clays, pressures approaching at-rest may develop unless wall movement can continue with time.

3.8 INFLUENCE OF GEOMETRICAL SHAPE OF RETAINING STRUCTURE ON WALL FRICTION

When relative movement can occur between a wall and the supported soil, the effect of wall friction must be taken into account. In some cases the wall is free to move with the soil, such as in the case of lagging between soldier piles. In these cases little or no wall friction is mobilised.

When the outer failure surface from the heel of the wall intersects or lies within the wall Coulomb's conditions apply. Rankine's conditions only apply to cases where this failure surface does not intersect the wall, as shown in Figure 9.

3.9 INFLUENCE OF LIMITED BACKFILL

The methods given above assume that the soil is homogeneous for a sufficient distance behind the wall to enable an inner failure surface to form in the position where static equilibrium is satisfied (Figure 12). Where an excavation is made to accommodate the wall, the undisturbed insitu material may have a strength differing from the backfill. If equations are used, the position of two failure planes should be calculated, one using the properties of the backfill material and one using the properties of the undisturbed material. If both fall within the physical limit of the backfill, the critical failure plane is obviously the one calculated using the backfill properties. Similarly, if they both come within the undisturbed material, the critical one is that for the undisturbed material properties.

Two other possible situations may arise: firstly where critical failure planes occur in both materials, in which case the one giving the maximum earth pressure is used, and secondly where the failure plane calculated with the backfill properties would fall within the undisturbed material,and the failure plane for undisturbed material would fall within the backfill. In the latter case, which occurs when the undisturbed material has a high strength, the backfill may be assumed to slide on the physical boundary between the two materials. The earth pressure equations do not apply in this case, but the wedge method may be used with the already selected failure plane and the backfill soil properties. The total pressure thus calculated is less than the active value assuming uniform material behind the wall. The variation of pressure with depth is not linear, and should be determined by the procedure given in Figure 12.

The boundary between the two materials should be constructed so that there is no inherent loss of strength on the surface. Benching the insitu material ensures that the failure surface is almost entirely through insitu or well compacted material.

3.10 PRESSURES PRODUCED BY COMPACTION

Proper compaction of backfill to a retaining wall is necessary in order to increase the backfill shearing strength and to prevent its excessive settlement later. Care should be taken to ensure that the compaction process does not cause damage to the wall, as pressures produced by compaction can vary considerably in magnitude and distribution and can be much larger than those predicted using classical earth pressure theories.

Aggour & Brown (1974) give guidance on the formulation of numerical solutions to compaction problems and include in their paper graphical solutions which indicate the influence of some factors affecting residual pressures, e.g. backfill geometry, wall flexibility, end wall restraint.

Broms (1971) has presented a method for the determination of lateral earth pressures due to compaction against unyielding structures and proposes the earth pressure distribution shown in Figure 14(i) for use in design. The associated data relating to the figure are for a limited range of compaction plant.

Ingold (1979) has presented a simple analytical method which can be used to give a working approximation of compaction induced pressures for routine designs. The method is based on the following assumptions :

(a) An idealised stress path is followed in the compaction process

(b) Below a critical depth Z_c there is no reduction in horizontal stress after removal of compactive force. Ingold shows that approximate values of Z_c may be obtained from the following expressions :

$$Z_c = \frac{K_a{}^2\Delta\sigma'_v}{\gamma} \qquad \text{or } Z_c = K_a\sqrt{\frac{2p}{\Pi\gamma}} \qquad .....(7)$$

(c) The increase in vertical effective stress, $\Delta\sigma'_v$, at a depth Z due to a dead weight of vibratory roller applying a unit weight p/unit length may be obtained from the expression :

$$\Delta\sigma'_v = \frac{2p}{\Pi Z} \qquad .....(8)$$

The depth, h_c, below which active pressure due to the weight of the overlying soil exceeds the compaction induced pressure is obtained from :

$$h_c = \frac{1}{K_a}\sqrt{\frac{2p}{\Pi\gamma}} \qquad .....(9)$$

The effect of compaction on lateral pressure is shown in Figure 14(ii)(a) & (b) and the resulting pressure distribution for use in design, based on this simplified theory, is shown in Figure 14(ii)(c). Ingold's design pressure distribution can be seen to be very similar to that of Broms shown in Figure 14(i).

3.11 EFFECTS OF COMPACTION ON CONVENTIONAL WALL DESIGN

The lateral pressures induced by compaction (Figure 14) can be up to twice the active pressures obtained by conventional analysis. These compaction pressures lead to higher structural loads, which may cause distress or result in serviceability problems with a wall.

If movement of the wall is allowed to take place these compaction-induced pressures are reduced. Translations or rotations of the order of H/500 are sufficient to reduce the pressures to near the active state. The final pressure distribution is parabolic rather than triangular, and thus the line of thrust is raised.

It is satisfactory to use the active pressure distribution when determining the factor of safety against sliding. The bending moments after sliding has taken place may still be up to 50% higher than those predicted using a triangular active pressure distribution. Calculations of bearing pressures and overturning moments should take into account the higher position of the line of thrust.

Reference should be made to Ingold (1979) for more detailed discussion of the above.

CHAPTER 4
EFFECTS OF SURCHARGES

4.1 UNIFORM SURCHARGES

Loads imposed on the soil behind the wall should be allowed for in design.

Uniform surcharge loads may be converted to an equivalent height of fill and the earth pressures calculated for the correspondingly greater height. In this case the depth of the tension zones in cohesive material is calculated from the top of the equivalent additional fill. The distribution of pressure for the greater height is determined by the procedures given in Chapter 3. The total lateral earth pressure is calculated from the pressure diagram, neglecting the part in tension and/or the part in the height of fill equivalent to the surcharge, as shown in Figure 12.

Buildings with shallow foundation may be taken as a uniform surcharge of 10kPa per storey.

The standard loadings for highway structures in Hong Kong are expressed in terms of HA and HB loading as defined in BS 5400 : Part 2 : 1978. In the absence of more exact calculations, the nominal load due to live load surcharge may be taken from Table 4.

The two loading cases shown in Figure 16 need to be considered.

Table 4 Suggested Surcharge Loads to be Used in the Design of Retaining Structures (Public Works Department, 1977)

Road class	Type of live loading	Equivalent surcharge
Urban trunk Rural trunk (Road likely to be regularly used by heavy industrial traffic)	HA + 45 units of HB	20kPa
Primary distributor Rural main road	HA + 37½ units of HB	15kPa
District and local distributors Other rural roads Access Roads, Carparks	HA	10kPa
Footpaths, isolated from roads Play areas		5kPa

Note : 1. It is recommended that these surcharges be applied to the 1 in 10 year storm condition.
2. For footpaths not isolated from roadways, the surcharge applying for that road class should be used.

4.2 LINE LOADS

Where there is a superimposed line load running for a considerable length parallel to the wall, the Wedge Method of design may be used, and the weight per unit length of this load can be added to the weight of the particular trial wedge to which it is applied. A step thus appears in the active force locus, as the weight of the trial wedge suddenly increases when the line load is included. The increased total earth pressure will be given from the trial wedge procedure, but the line load will also change the point of application of this total pressure. The method given in Figure 15 may be used to give the distribution of pressure.

When the line load is small compared to the active earth pressure, the effect of the line load on its own should be determined by the method given in Figure 15. This is based on stresses in an elastic medium modified by experiment. The pressures thus determined are superimposed on those due to active earth pressure and other pressures as appropriate.

4.3 POINT LOADS

Point loads cannot be taken into account by trial wedge procedures. The method based on Boussinesq's equations given in Figure 15 may be used, but it should be noted that the method is only approximate as the stiffness of the wall is not taken into account.

CHAPTER 5

EFFECTS OF WATER

5.1 GENERAL

The presence of water behind a wall has a marked effect on the pressures applied to the wall. When the phreatic surface intersects the wall, a hydrostatic pressure is exerted against the wall, together with uplift pressures along the base of the wall. Even when there is no water in direct contact with the wall, such as when adequate drainage is provided, there is an increased pressure on the wall due to the increased earth pressure (Section 5.2). The effect of water behind the wall is significant; the total force may be more than double that applied for dry backfill. Many recorded wall failures can be attributed to the presence of water.

The height to which water can rise in the backfill, and the volume of flow, are both of prime concern. To determine these the ground water conditions must be established. These may be best derived from the observation of groundwater conditions prior to construction using piezometers and by applying the principles outlined in this Section and in Chapter 4 of the Geotechnical Manual for Slopes. Notwithstanding the results of groundwater monitoring, the groundwater level assumed for design should be not lower than one-third of the retained height.

The effect of leakage from services can be significant. There is evidence from field measurements and failures in Hong Kong that this leakage contributes substantially to both perched and main groundwater tables. The provisions for services outlined in Sections 9.18 & 9.19 of the Geotechnical Manual for Slopes are appropriate for retaining walls, and these should be applied.

Where inadequate drainage is provided behind a retaining structure, there may be a damming effect which would result in raising groundwater levels locally and in the general area. Such a rise may adversely affect the stability of slopes and retaining walls. Effective drainage measures should always be provided in such cases.

5.2 EFFECT OF WATER ON EARTH PRESSURES

5.2.1 *Static Water Level*

When a soil is submerged, its effective unit weight is reduced to $\gamma' = \gamma_{sat} - \gamma_w$. The lateral earth pressure should, in this case, be calculated using γ' in equations or charts. Alternatively, in graphical procedures such as the trial wedge method, all forces acting on the soil wedge, including the hydrostatic normal uplift pressure on the failure plane and the lateral hydrostatic pressure, may be included in the trial wedge procedure. This is illustrated in Figure 6 to 8.

In low permeability cohesive soils, the pore water pressures set up during construction may be in excess of any hydrostatic pore pressure, so an undrained analysis may be more appropriate.

When tension cracks occur, lateral hydrostatic water pressure should be included for the full depth of the crack, as given in Section 3.5 or for H/2, whichever is less. Full lateral water pressure must be allowed for below the invert of the lowest weep holes or other drainage outlets.

5.2.2 *Flowing Water*

If the water in the soil voids is flowing, the pore water pressures are changed from the hydrostatic values to values determined by the seepage pattern. These values have to be used in a trial wedge solution to determine the earth pressure.

The actual flow pattern developed is very dependent on the uniformity and homogeneity of the ground, and on the position of any drains. Figure 17(a) shows the flow net produced by steady seepage into a vertical drain when the phreatic surface is below ground level and the backfill uniform and isotropic. Rainfall of intensity equal to or greater than the permeability of the backfill will change this flow net to that shown in Figure 17(b) if there is no surface protection to prevent infiltration. There is a significant increase in water pressure on the failure surface for this latter case. It is thus desirable, for this drainage arrangement, to prevent water entering the backfill from the surface. Figure 17(c) shows the flow net due to heavy rainfall infiltration into an inclined drain. The effect of this drainage arrangement is to reduce the water pressure in the backfill to zero; this is therefore a very effective drainage measure.

The pore water pressures normal to the active or passive wedge failure surface affect the forces acting on a wall. The resultant thrust on the failure surface, determined from a flow net, is applied in the force polygon for the soil wedge together with any lateral water pressure at the wall as shown in Figures 6 to 8. The method of determining water pressures from the flow net, and hence the water force, is shown in Figure 17.

For methods of dealing with seepage through anisotropic and non-homogeneous backfills, reference may be made to Cedergren (1977).

5.3 DRAINAGE PROVISIONS

Water pressures must be included in the forces acting on the wall unless suitable drainage is provided. Good practice requires that drainage is always provided.

For walls less than 2 metres high, drainage material is usually only provided on the back face of the wall, with weep holes to relieve water pressure. In some low risk situations, it may be geotechnically tolerable and economically advantageous to omit the drain and design for the hydrostatic water pressure.

With correctly designed inclined drainage systems, such as those shown in Figures 18(a) & (c), water pressures may be neglected both on the wall itself and on the soil failure plane. Alternative drainage details as shown in Figures 18(b) & (d) may be used. In these cases, the appropriate water pressure should be considered in design. Hydrostatic pressure will act on the wall below the lowest drainage outlet.

For a drain to be effective it must be able to carry the design flow of water without backing up or blocking. This design flow should include the flows from leaking or burst service conduits where appropriate.

To prevent blockage, the drain must be protected by an adequate filter, designed according to the rules given in Section 5.4.

The rate of seepage into the drain from the soil can be determined from a flow net together with a knowledge of the permeabilities of the soils involved and a flow-net. Methods for determining permeabilities are outlined in Section 2.9.

The water flow rate that the drainage layer can accommodate depends on the permeability of the drainage medium, the thickness of the drain and the hydraulic gradient in the drain. In some cases, it may be intended that the filter itself should act as a drain; if so, it should be designed to have adequate capacity.

By the use of a conventional flow net sketch, the approximate rate of flow into the drain may be estimated. Using an appropriate value of hydraulic gradient, i, and the value of permeability for the drainage material, k_d, the required area of drainage material, A, normal to the direction of flow can be determined by application of Darcy's law :

$$A = \frac{q_d}{k_d i}, \qquad \text{.....(10)}$$

where q_d is the flow rate through the drain.

As a very general guide drainage material should have a permeability at least 100 times that of the material it is meant to drain. If this is achieved, pore water pressures due to seepage will be minimised at the boundary, and the soil mass will drain as though it had a free boundary. Permeabilities of granular (drainage) materials are given in Figure 20.

In some cases, Figure 19 (Cedergren 1977) may be useful in determining the thickness of the filter or drain, but it should be noted that construction considerations often govern thickness.

The maximum allowable hydraulic gradient in the drain depends on the largest hydrostatic head that can safely develop without causing undesirable hydrostatic pressures or infiltration into the backfill.

A common practice in Hong Kong is to use no-fines concrete or hand-packed rubble as a drainage layer behind retaining walls. These materials should be protected with a transition zone of gravel or crushed rock which will not migrate into the voids of the rubble or no-fines concrete, and which conforms to the filter design rules applied to the soil-protecting filter. However, when this is done, it is probable that the hand-packed rubble or no-fines concrete can be omitted and that the transition zone can be used as the drainage layer.

It should be noted that a clean well-graded rock backfill protected by an appropriate filter would be an excellent solution in any location where seepage from the soil or leakage from service conduits may be a problem.

5.4 *FILTER REQUIREMENTS*

5.4.1 *Graded Filters*

All drainage that is provided should be adequately protected by properly designed filter layers against blockage due to the movement of the finer soil particles. Filters should be more permeable than the protected soil, and filter materials should be transported and placed carefully so that segregation, and contamination by fines, does not occur.

Table 5 gives details of the normal filter design criteria applicable to soils in Hong Kong. Where the base soil contains a large percentage of gravel or larger sized particles, the finer fraction should be used for the filter design.

Where filter materials are used in conjunction with a coarser free-drainage material such as crushed rock, the grading of the coarser material should conform to the filter design criteria given in Table 5, to protect the filter from erosion.

Table 5 Filter Criteria to be Used in Hong Kong (Geotechnical Manual for Slopes, 1979)

Rule Number	Filter Design Rule
(1)	$D_{15}F_c < 5 \times D_{85}S_f$
(2)*	$D_{15}F_c < 20 \times D_{15}S_f$
(3)	$D_{15}F_f > 5 \times D_{15}S_c$
(4)	$D_{50}F_c < 25 \times D_{50}S_f$
(5)	Uniformity coefficient $4 < \frac{D_{60}F}{D_{10}F} < 20$
(6)	Should not be gap graded
(7)	Maximum particle size : 75mm
(8)	Not more than 5% to pass 63µm sieve, and this fraction to be cohesionless

* For well-graded base soil this criterion can be extended to $40 \times D_{15}S_f$.

In this table, $D_{15}F$ is used to designate the 15% size of the filter material (i.e. the size of the sieve that allows 15% by weight of the filter material to pass through it). Similarly, $D_{85}S$ designates the size of sieve that allows 85% by weight of the base soil to pass through it. $D_{60}F_c$ indicates the D size on the coarse side of the filter envelope. $D_{10}F_f$ indicates the D_{10} size on the fine side of the filter envelope.

When certain gradings of decomposed volcanic materials with an appreciable fines content are being used as backfill, the filter design may require special care.

Reference should be made to Section 4.19 of the Geotechnical Manual for Slopes for further discussion on the design of filters.

5.4.2 *Geotextiles*

In some cases, it may be possible to use man-made fibrous woven and non-woven fabrics, known as *geotextiles*, to protect the drainage facilities.

As yet, there is very little experience in Hong Kong with the long-term performance of fabric filters for permanent drainage measures. Consequently, it is recommended that they should only be used in low risk situations, as defined in Table 2.1 of the Geotechnical Manual for Slopes, and where failure could not be expected to occur even if total blockage of the fabric occurred. It is also recommended that they should only be used in locations where they can be replaced if found to be defective after a period in operation.

There are objections to the use of some of these materials, such as serious deterioration on exposure to sunlight and ultra-violet light, clogging due to movement of fines, reduction in permeability due to compression, constructional difficulties and materials forming planes of weakness in the works. If these objections are overcome by attention to design, construction and quality control, then the availability of geotextiles provides new opportunities for innovative filter/drain design and construction.

Fabric filters should be properly designed to be in filter relationship with the surrounding soil. Care must be taken to select a geotextile which is appropriate to the grading of the soil it is intended to protect and has adequate drainage capacity for the particular application. A summary of design criteria for fabric filters is given in the book by Rankilor (1981).

Available literature suggests that fabrics with an equivalent opening size of less than 150μm (or an open area of less than 4%) and the thicker non-woven fabrics, may be more prone to clogging than other varieties. The use of these types should therefore be avoided unless the satisfactory performance of the particular soil/fabric/drainage-medium system has been demonstrated by permeability test. On the other hand, some of the very thin fabric varieties exhibit quite large visible gaps caused by uneven distribution of fibres, and the use of such defective materials should also be avoided.

During construction, stringent measures are required to ensure that the manufacturer's instructions concerning storage and handling are strictly followed, and that storage, placement and backfilling of fabrics are carefully controlled to avoid excessive exposure to ultra-violet light, mechanical damage and ineffective overlapping. It is prudent to use two layers of fabric as a precaution against impairment of the filter function by mechanical damage during placement.

5.5 *CONTROL OF GROUNDWATER*

Groundwater levels may need to be controlled in excavations, particularly in sheeted excavations. The dewatering method chosen should assure the stability of the excavation and the safety of adjacent structures. Techniques for dewatering are outlined in the Code of Practice for Foundations, CP2004 (British Standards Institution, 1972) and in Terzaghi & Peck (1967).

When pumping is carried out inside a sheeted excavation, flow will occur under the sheeting and up into the excavation. Piping may occur in dense sands if the seepage exit gradient at the base of the excavation equals about 1.0. Heave associated with groundwater flow may occur in loose sands if the uplift force at the sheeting toe exceeds the submerged weight of the overlying soil column. Both failure modes may be prevented by increasing the depth of penetration of the sheeting.

Design charts for determining the stability against piping in excavations are given in Figure 21.

CHAPTER 6

STABILITY OF RETAINING WALLS

6.1 GENERAL

The stability of a free standing retaining structure and the soil contained by it is determined by computing factors of safety (or *stability factors*), which may be defined in general terms as :

$$F_s = \frac{\text{Moments or forces aiding stability}}{\text{Moments or forces causing instability}} \quad \text{.....(11)}$$

Factors of safety should be calculated for the following separate modes of failure and should apply to the 1 in 10 year groundwater condition :

(a) sliding of the wall outwards from the retaining soil,

(b) overturning of the retaining wall about its toe,

(c) foundation bearing failure, and

(d) larger scale slope or other failure in the surrounding soil.

The forces that produce overturning and sliding also produce the foundation bearing pressures and, therefore, (a) and (b) above are inter-related with (c) in most soils.

In cases where the foundation material is soil, overturning stability is usually satisfied if bearing criteria are satisfied. However, overturning stability may be critical for strong foundation materials such as rock, or when the base of the wall is propped, or when the base of the wall is small, for instance with crib walls.

In general, to limit settlement and tilting of walls on soil materials, the resultant of the loading on the base should be within the middle third. For rock foundation material, the resultant should be within the middle half of the base.

When calculating overall stability of a wall, the lateral earth pressure is calculated to the bottom of the blinding layer, or in the case of a base with a key, to the bottom of the key where the actual failure mechanism extends to that point.

If the passive resistance of the soil in front of a wall is included in the calculations for sliding stability, only 50% of the calculated passive resistance should be used, because of the large deformations required to mobilise the full passive resistance.

Stability criteria for free standing retaining walls are summarised in Figure 22.

6.2 SLIDING STABILITY

6.2.1 *Base without a Key*

Sliding occurs along the underside of the base (see Section 2.6 for further discussion).

The factor of safety, F_s, against sliding should not be less than 1.5.

$$F_s \text{ (sliding)} = \frac{(W_t + P_v)\tan \delta_b + c_b B + 0.5P_p}{P_H} \qquad .....(12)$$

where W_t is the weight of the wall

P_V is the vertical component of earth pressure force

P_H is the horizontal component of earth pressure force

δ_b is the angle of base friction

c_b is the adhesion at the base of the wall

B is the base width, and

P_p is the passive pressure force.

The effects of water forces should be taken into account in this equation, including uplift pressures below the wall base, unless drains that permanently and effectively eliminate uplift water pressures are provided.

6.2.2 *Base with a Key*

Huntington (1961) suggests that walls with shallow keys should be analysed assuming that sliding occurs on a horizontal plane through the soil at the bottom of the key. Both active and passive forces should be adjusted to take into account the depth of the key. The weight of soil in front of the key and below the base, down to the failure surface, should be included in the total weight, W_t. Figure 1 shows the forces involved. The factor of safety against sliding should be as given in Section 6.2.1, with the angle of base friction, δ_b, replaced by the angle of shearing resistance, ϕ', of the foundation soil.

6.2.3 *Sliding on a Rock Foundation*

It is possible to analyse the sliding of a retaining wall on a rock foundation in a similar manner to sliding of rock along a rock joint. The basic friction angle may be increased by a waviness angle, i_w, based on the measured waviness of the exposed rock surface.

The waviness must be of a sufficient size so that shearing through the asperity does not occur. In addition, there must be a significant component of the rock surface inclined at i_w in the direction of sliding.

6.3 *OVERTURNING STABILITY*

6.3.1 *General*

Moments calculated about the bottom of the front of the toe should give a factor of safety, F_s, against overturning of not less than 2.

$$F_s \text{ (overturning)} = \frac{M_r}{M_o} \qquad \text{.....(13)}$$

where M_r is the algebraic sum of moments resisting overturning and
M_o is the algebraic sum of moments causing overturning.

For semigravity cantilever and counterfort walls, only the overturning factor of safety for the wall as a whole is significant. For crib walls and solid gravity walls for which the base and the upper portion of the wall are usually separate units, the factor of safety of the upper portion against overturning about its toe should be checked.

Passive resistance should not be included in calculations for F_s (overturning) for conventional walls.

6.3.2 *Factor of Safety against Overturning*

There are a number of ways in which a factor of safety against overturning may be determined, and these lead to significant differences in the computed value of F_s.

In order to understand why some of these differences occur, the forces acting on the simple retaining wall illustrated in Figure 22(a) will be examined. Dry backfill only is considered, and terms are defined on the diagram.

Application of equation (13) gives (Figure 22) :

$$F_s \text{ (overturning)} = \frac{W_t . a}{P_A . m} \qquad \text{.....(14)}$$

It may be noted that, for the usual proportions of solid gravity retaining walls, the batter of the back is usually such that the line of action of P_A passes below the toe. The lever-arm, m, is thus negative and P_A contributes to the stability of the wall. A negative value of F_s thus indicates that the wall cannot overturn.

It is usual in retaining wall design to work in terms of the horizontal and vertical components of the overturning force P_A. These forces, multiplied by their respective lever arms and substituted into equation (14) for the simple case as illustrated in Figure 22(a).

give

$$F_s = \frac{W_t . a}{P_H . \bar{y} - P_V . f} \qquad \text{.....(15)}$$

It is commonly assumed however that the component P_V contributes to resisting overturning and on this basis, the factor of safety becomes

$$F_s = \frac{W_t . a + P_V . f}{P_H . \bar{y}} \qquad \text{.....(16)}$$

Equations (15) and (16) do not, of course, give the same value of factor of safety.

It can be seen that, according to equation (16), the overturning factor of safety is that number by which the horizontal component of the earth pressure would need to be multiplied to cause overturning, the vertical component of this pressure remaining unchanged. It is unlikely, however, that the horizontal component of the resultant earth pressure would increase and the vertical component remain unchanged. On this basis, it would appear that the procedure represented by equation (16) is not logical.

Although equation (16) leads to a more conservative result than the procedure based on equation (15), it is not recommended and the design data given in Figure 22 is based on the more logical procedure represented by equation (15). Huntington (1961) discusses this topic.

6.3.3 *Walls with Deep Keys*

Application of an analysis of rotational stability of walls with deep keys to the real situation is found to be very uncertain, as the forces acting are dependent on the relative stiffness of the wall and the supporting soil, and on the deformation that takes place. In view of constructional difficulties and likely large deformations, walls with deep keys should in general be avoided (see Section 11.7).

6.4 *FOUNDATION BEARING PRESSURE*

6.4.1 *General*

The ultimate bearing capacity of the foundation soil on which an earth retaining structure rests should generally be determined from a theoretical analysis of the foundation, using the soil properties obtained from laboratory tests. Where appropriate, these shear strength properties should be reviewed as the construction proceeds. The applied loading should provide a factor of safety of 3.0 against ultimate bearing failure.

Foundations of retaining walls are usually subjected to inclined and eccentric loads, the foundation itself may be tilted at an angle to the horizontal and sometimes the wall is founded on sloping ground. A general expression for the ultimate bearing capacity of shallow foundations which can deal with these situations has been given by Vesic (1975), and this is presented in Section 6.4.2.

Other factors which may influence the bearing capacity are the foundation depth, soil compressibility, scale effects and non-homogeneous soil conditions. These are discussed by Vesic (1975).

6.4.2 *Bearing Capacity Factors*

The ultimate bearing capacity of a shallow ($D \leq B$) strip foundation is given by :

$$
\begin{aligned}
q_{ult} = \frac{Q}{BL} &= c\ N_c\ S_c\ i_c\ t_c\ g_c && \text{- term relating to effects of cohesion} \\
&+ \tfrac{1}{2}\ \gamma\ B_\gamma\ N_\gamma\ S_\gamma\ i_\gamma\ t_\gamma\ g_\gamma && \text{- term relating to influence of unit weight of soil} \\
&+ q\ N_q\ S_q\ i_q\ t_q\ g_q && \text{- term relating to surcharge effects}
\end{aligned}
\quad \ldots.(17)
$$

The bearing capacity factors, N_c, N_γ, N_q are functions of the angle of shearing resistance, ϕ, of the soil and are modified as appropriate using factors for the shape of footing, S_c, S_γ, S_q, inclination of load, i_c, i_γ, i_q, tilt of footing base , t_c, t_γ, t_q, and slope of ground, g_c, g_γ, g_q. Values for these factors are given in Figure 23.

The above bearing capacity factors have been determined on the assumption that the foundation material is reasonably incompressible, so that failure would occur by general shearing. For compressible materials, failure occurs by local or punching failure. For these materials Terzaghi (1943) recommended that the value of cohesion used should be reduced to $2c'/3$, and the angle of shearing resistance to $\tan^{-1}((2 \tan \phi')/3)$. A more accurate solution considering both compressibility and size effects is given by Vesic (1975).

In using the above expression, it should be noted that foundations constructed on the relatively high permeability residual soils usually encountered in Hong Kong, decomposed granites and volcanics, require the analysis of bearing capacity to be carried out in terms of effective stresses. Under these conditions, the contribution to the bearing capacity of the cohesive terms is in general very small and may be neglected.

For foundations constructed on saturated clayey soils of low permeability, the short-term stability is critical, and they are usually analysed in terms of undrained strength ($\phi' = 0$ analysis).

Where a wall is founded on compacted fill overlying either soft clay or loose fill, particular care must be taken. Reference should be made to Vesic (1975).

6.4.3 *Effect of Groundwater Level*

Equation (17) applies when the groundwater table is at a distance of at least B below the base of the foundation. When the water table is at the same level as the foundation, the submerged unit weight of the soil below the foundation should be used. For intermediate levels of the water table, the ultimate bearing capacity should be interpolated between the above limiting values.

6.5 *ECCENTRIC LOADS*

When the load on the foundation is eccentric, this substantially reduces the bearing capacity. To allow for this, the base width, B, is reduced to an effective width B' given by :

$$B' = B - 2e_b \qquad \text{.....(18)}$$

where e_b is the load eccentricity ($e_b \leq \frac{B}{6}$).

For a footing eccentrically loaded in two directions, the effective dimensions of the base become such that the centre of an area, A', coincides with the vertical component, V, of the applied load. Then :

$$A' = B' \times L'$$

where $L' = L - 2e_l$, and $B' = B - 2e_b$, and e_l, e_b are the load eccentricities in the two directions.

L' and B' are then used in place of L and B in all equations.

The factor of safety is given by :

$$F_s \text{ (bearing)} = \frac{q_{ult}}{q_{all.}} \qquad \text{.....(19)}$$

where $q_{all.} = \frac{V}{A'}$ for a rectangular footing, and $q_{all.} = \frac{V}{B'}$ for a continuous strip footing (unit length considered).

6.6 FOUNDATIONS CONSTRUCTED ON SLOPING GROUND AND NEAR SLOPE CRESTS

The ultimate bearing capacity of foundations constructed on slopes is lower than that for foundations constructed on level ground. The ground slope factors of Vesic (1975), given in Figure 23, are devised to take this into account.

Where a foundation is constructed on the crest of a slope, the bearing capacity increases with distance from the crest to a maximum value at distances from the crest greater than approximately four times the foundation width. No exact solution is available for this case. The procedure outlined by Bowles (1977) could be applied to the values given by Vesic in Figure 23. Alternatively, as a conservative assumption, a linear variation between the two extreme values may be used.

The bearing capacity calculations do not consider the fact that the soil on the slope is already under stress. This is particularly important where the inclination of the slope is greater than $\phi'/2$. The overall stability of the slope under the influence of the loaded footing must therefore be checked, in addition to the bearing capacity calculation.

6.7 FOUNDATIONS ON ROCK

Foundations on continuous sound rock seldom present problems since the rock is stronger than most foundation materials. Structural defects and discontinuities, or the compressibility of the rock mass below the foundation, usually control the allowable bearing pressure.

Where discontinuity-controlled failure mechanisms are possible, joint surveys should be carried out in the excavation and adjacent slopes.

The compressibility of the rock mass below foundation level depends on the frequency of joints and on the amount and type of infilling of these joints in the zone of influence of the foundation. RQD (Rock Quality Designation) is defined as :

$$\text{RQD (\%)} = 100 \times \frac{\text{Length of unweathered core} \geq 100\text{mm}}{\text{Length of borehole}} \qquad .....(20)$$

In unweathered rocks, RQD indicates the joint intensity, whereas in weathered rock it gives a measure of the amount of compressible material but no indication of the infill compressibility.

Where only tight clean joints are present, the correlation between RQD and allowable bearing pressure proposed by Peck et al (1974), given in Table 6, may be used.

Table 6. Allowable Bearing Pressure on Jointed Rock (Peck, Hanson & Thornburn, 1974)

RQD (%)	Allowable Pressure (kPa)	
100	30000	Note :
90	20000	(1) Use allowable pressure or
75	12000	unconfined compressive strength of
50	6500	intact rock, whichever is less.
25	3000	(2) RQD is for rock in the zone
0	1000	of influence of the foundation.

For infilled joints deformation will be larger, and estimates of the joint infill compressibility may be required. The effect of joint infilling on allowable bearing pressure for a limited range of joint spacing and thickness is given in the Canadian Foundation Manual (Canadian Geotechnical Society, 1978).

6.8 SLOPE FAILURE IN SURROUNDING SOIL

The overall stability of the ground surrounding the retaining wall should be investigated, and calculations should be carried out on the full range of potential failure surfaces to ensure that an adequate factor of safety against overall slope failure is maintained. The calculations should include the influence of the surcharge from the wall on the slope. The minimum factor of safety required at a site is dependent on its hazard potential.

Reference should be made to Chapter 5 of the Geotechnical Manual for Slopes, where detailed guidance is given on the *Risk Category* of a slope and the minimum factor of safety required. The factor of safety should be determined for groundwater conditions associated with a 10 year return period rainfall. Chapter 5 of the Geotechnical Manual also gives guidance on methods that may be used for carrying out the analysis.

CHAPTER 7

SHEET RETAINING STRUCTURES

7.1 GENERAL

Walls which have uniform cross-section with depth are considered in this chapter. These include flexible sheet structures, such as sheet-piled and soldier-piled walls, and more rigid walls, including diaphragm and caisson walls.

The earth pressure which acts on an earth supporting structure is strongly dependent on the amount of lateral deformation which occurs in the soil. For flexible sheet walls, the determination of deformations, and hence the earth pressures, is not simple, because the yield of one part of a flexible wall throws pressure on to the more rigid parts. Hence, the pressures in the vicinity of the supports are higher than in the unsupported areas, and the loads on individual supports vary depending on the stiffness characteristics of the supports themselves.

Deformation of the ground adjacent to excavations may cause breakage of water-carrying services. In situations where large flows may result, the prudent designer will allow for the water table being at the ground surface when calculating loads to be retained.

7.2 STRUTTED EXCAVATIONS

Strutted sheet piling is often used to provide temporary support for the sides of deep excavations. The sheet piles are usually driven first with support struts being installed as the excavation proceeds. The final deformations of the wall are highly dependent on the construction sequence and detailing. This is depicted in a simplified manner in Figure 28.

Failure of a strutted wall often results from the initial failure of one of the struts, resulting in the progressive failure of the whole system. The forces in identical struts in any particular support system may differ widely because they depend on such factors as the way in which the struts are preloaded and the time between excavation and installation of struts. Loads in similar struts in any set of observations have been found to vary from the average value by up to ± 60 percent (Lambe et al, 1970).

Since failure of strutted cuts often occurs by structural failure, particular attention should be paid to the structural detailing of the internal strutting. Guidance on the structural design of such walls, together with typical details of connections and strutting systems, are given by Goldberg et al (1975). Struts must be sufficient for all stages of construction.

The distribution of pressure on a strutted excavation is complex, and it is normal to use a pressure envelope covering the normal range pressure distributions. The envelopes (Figure 24) given by Peck (1969), and the Japan Society of Civil Engineers (1977), together with loadings from groundwater and surcharge, should be used to determine strut loads for all internally strutted excavations. In assessing loading from groundwater, the effect of accidental breakage of water carrying services should be considered.

The load carried by each internal strut is estimated by assuming that the sheet pile is simply supported between struts, and that a reaction below the base of the excavation exists. This reaction is provided by the passive resistance of the soil beneath the cut.

The depth of penetration of the wall below the base of the excavation should be sufficient to provide this reaction.

Since the wall moves towards the excavation, it may be assumed that active and passive pressures develop against the wall below the excavation level, and horizontal equilibrium may be used to determine the depth of penetration. The passive resistance should be factored by 2.0.

For soft clays, neglible passive resistances develop, and the lower section of the wall must be designed as a cantilever, and the bending moment and deflection must be checked.

The maximum bending movement at, or below, the lowest strut should be checked against overstressing of the wall.

Instability of the base of an excavation can occur due to shear failure in soft to firm clays (known as *base heave*). In granular materials, piping or heave associated with groundwater flow can occur.

The factor of safety with respect to shear failure is given by :

$$F_s = \frac{N_b c}{\gamma H + q} \qquad .....(21)$$

where the terms are defined in Figure 25. Where F_s is less than 2 substantial deformations may occur with consequent loss of ground, and the probability of failure exists. Where soft clay extends to considerable depth below the excavation, the effect of increased sheeting stiffness,or depth, is minimal. However driving the sheeting into a hard stratum before commencing the excavation can appreciably reduce the deformations.

Control of the groundwater may be necessary to prevent piping or heave associated with groundwater flow. Methods to achieve this are discussed in Section 5.5.

7.3 ANCHORED FLEXIBLE WALLS

7.3.1 *Walls Anchored near the Top*

The deformation of an anchored sheet pile depends on the relative stiffness of the pile/soil system. For a relatively rigid system, such as a heavy pile section in a loose sand, the earth pressure distribution corresponds closely to the triangular active and passive conditions. The toe of the pile is assumed pinned, and the Free Earth Support design method as outlined by Teng (1962) is appropriate.

As the stiffness of the system decreases the pressure distribution alters in such a way as to reduce the bending moment in the pile. As a consequence, the sheet pile section used may be reduced as compared with an infinitely stiff wall. Rowe's Theory of Moment Reduction (1952, 1955, 1957) takes this effect into account; it is summarised by Teng (1962) and in CIRIA Report No. 54 (1974).

When calculating the toe penetration, it is recommended that no factor of safety should be applied to the active pressures. The passive resistance may be factored by 2.0, or, as recommended in the CIRIA report, the following factored values of $\emptyset'$ and δ, i.e. $\emptyset'_F$ and δ_F, may be used to calculate the passive resistance :

$$\phi'_F = \tan^{-1}\left(\frac{\tan\phi'}{F_s}\right) \quad \text{and} \quad \delta_F = \tan^{-1}\left(\frac{\tan\delta}{F_s}\right) \qquad .....(22)$$

For sands, $F_s = 1.5$ should be used, which gives an approximate factor of 2.0 on the derived K_p values. If, however, the values of ϕ' and δ are uncertain, then $F_s = 2.0$ should be used.

For the short term stability of walls in clays, a factor $2.0 \leqq F_s \leqq 3.0$ should be applied to the value of undrained cohesion, c, depending on the reliability of the parameters. For long term stability, the factor on tan ϕ' can be taken as $1.2 \leqq F_s \leqq 1.5$.

Passive and active pressures should be calculated using the methods given in Chapter 3.

7.3.2 *Multiple Anchored Walls*

The multiple-anchored system of wall support results in the retaining structure being progressively fixed. Consequently, the lateral deformations are limited to such an extent that failure within the retained soil is unlikely. The earth pressure which finally acts on the wall depends on the relative stiffness of the wall to the soil, the anchor spacing, the anchor yield and the prestress locked into the anchors at installation.

The earth pressure distribution has been shown to be similar to that obtained for internally braced excavations. A rectangular pressure envelope similar to that adopted by Peck (Figure 24) is appropriate. The earth pressure coefficient may be taken as K_a. However, it is common to use a value between K_a and K_o, such as $(K_a + K_o)/2$, in an attempt to control surface movements.

Successful designs have been made using triangular pressure distributions with earth pressure coefficients varying between K_a and K_o. However, because of the mechanism involved, the rectangular distribution is considered more appropriate (Hanna, 1980). Anchor loads may be checked using both distributions, and the worst case taken.

The determination of vertical and horizontal spacing of anchors using the procedure for internal strut spacing gives acceptable results. Another approach is the semi-empirical design method of James & Jack (1974) which simulates the field construction procedure using triangular pressure distributions. This method allows determination of the depth of penetration required, and results correspond well to field and laboratory tests.

7.3.3 *Effects of Anchor Inclination*

Anchors are usually inclined downwards, transmitting the vertical component of the anchor force into the anchored member. This force should be considered in design, together with the weight of the member itself (White, 1974).

A number of cases have been recorded where soldier piles have failed in end bearing due to the vertical component of the anchor force.

7.4 *CANTILEVERED WALLS*

Relatively rigid cantilevered caisson walls are used in Hong Kong. These rely entirely on the development of passive resistance in front of the wall for their stability. As a consequence, considerable movement must occur before equilibrium is reached, and deep penetration is required. The deflection at the top of the wall may be the governing criterion. Such walls should not normally be used as permanent structures to retain a height of more than 5m unless cantilevered from rock.

The pressure distribution at failure approximates the classical triangular pattern. Full active pressure should be used and the passive pressure should be factored with $F_s = 3$ on tan ϕ' and tan δ (refer to Section 2.7 for appropriate values of δ). This higher factor of safety is required because of the large deformations needed to develop full passive resistance. However, if it can be shown that wall deformations will not cause distress to neighbouring structures or services, then a lower factor may be appropriate.

The depth of penetration is obtained by taking moments about the toe. The maximum bending moment may be obtained by taking moments of the pressures, above various cuts, until the maximum value is determined.

Installation of a drainage and filter medium behind the wall may be difficult and so full hydrostatic pressure may have to be considered for the design.

CHAPTER 8

REINFORCED EARTH RETAINING WALLS

The technique of reinforced earth is used for retaining walls in various parts of the world. Such walls are relatively new to Hong Kong, and there is little experience under Hong Kong conditions.

It is recommended, at present, that designs should be in accordance with the Technical Memorandum (Bridges) BE 3/78 (Department of Transport, UK, 1978). It is also recommended that for the backfill, the grading and plasticity index requirements of the Federal Highways Administration (1978), outlined in Table 7, should also be met, because of the limited documented experience of reinforced earth retaining walls constructed using materials with a high fines content and plasticity index.

It is considered that difficulty will be experienced in obtaining suitable backfill material from natural sources. Decomposed volcanics will not meet the specifications. For decomposed granites, it is likely that the variability of grading and plasticity index within a local area will present difficulties. Consideration should therefore be given to the use of crusher-run or similar materials. Designers are advised not to commit the design of a wall to a reinforced earth system until a sufficient source of fill that will meet the specification has been identified.

Close supervision is required to ensure that construction proceeds according to specification, particularly all aspects of the backfill specification. Difficulties with later provision of services and the sterilization of land above for building development may preclude the use of reinforced earth in certain circumstances.

Table 7 Minimum Specification for Select Backfill for Reinforced Earth Retaining Walls (after Federal Highway Administration, 1978)

Sieve Size	Percentage Passing
150mm	100
75mm	75 - 100
75µm	0 - 25

and PI $<$ 6

OR If percentage passing 75µm is greater than 25%, and percentage finer than 15µm is less than 15%, material is acceptable if $\phi \geq 30^{o}$ as determined by the appropriate test and P.I. $<$ 6.

CHAPTER 9

CRIB WALLS

9.1 GENERAL

Crib walling, although commonly used in some countries (e.g. New Zealand, Australia and the U.S.A.), has not been much used in Hong Kong. The technique can provide walls that are economical, aesthetically pleasing, and relatively rapid to construct.

A crib wall structure is made by placing a number of criblike cells together and filling them with soil or rock fill to give them strength and weight. The wall essentially acts as a gravity retaining wall. Crib wall units may be built of precast concrete, steel or of treated timber. The manufacturers of crib wall units produce design data for crib walls, but in general care must be exercised in the interpretation and application of this data.

The front face of a crib wall usually consists of a grid of concrete members so spaced that the soil infill at its angle of repose does not spill through the spacers. Horizontal members of such a grid are termed *stretchers*. The face members are connected by transverse members termed *headers* to a similar grid of stretchers, parallel to the face, forming the back face of the wall (Figure 26). The minimum thickness of walls should be one metre, except where the wall is *non-supporting* for landscaping. A 1.2 m thickness is usually a better engineering solution. Additional spacers between the stretchers within the front and back grids may be used if the system requires it, and these are termed *false headers* or *pillow blocks*. Headers should in general be prependicular to the face of the wall, although some available systems have variations to this.

The system usually allow for the addition of one or more grids of members parallel to the face and situated behind the structure described above, so forming multiple depth walls of greater height. Such additional grids are connected to the grid in the front by a header system.

9.2 DESIGN

The general design criteris for gravity walls apply to crib walls. The pressures acting on a crib wall should be determined by the methods given in Chapter 3. The resultant should always lie in the middle third of the wall cross-section. Figure 26 shows the earth pressure distribution acting on a typical wall and some typical construction details. Figure 27 gives design curves which may be used for preliminary design only.

To a great extent, the performance of a crib wall depends on the ability of the crib members to contain the enclosed soil. Analysis of the stresses and loadings in the crib members and connections is based on the earth pressure inside the crib. The individual units for crib walls should be designed to withstand the torsion, bending moments, shear forces and tensile forces exerted on them. The theoretical determination of the forces on crib units and the actual strength of the units is difficult and is usually based on earth pressures from bin pressure theories (Schuster et al, 1975; Tschebotarioff, 1951), the structural form of the crib units and the earth pressure from the backfill. However, it has been found by Schuster et al (1975) that stresses measured in crib wall units are much higher than those predicted using loads on the units from bin pressure theories. Specification CD209 - Crib walling and Notes (Ministry of Works and Development N.Z., 1980) specifies that crib units be able to withstand loadings which imply earth pressures twice those given by bin pressures. This requirement followed an examination of satisfactory and unsatisfactory crib wall units. Good detailing and design is required at the connection between units to ensure the satisfactory transfer of forces. Crib wall failures have occurred because of poor steel reinforcement detailing.

The Specification CD209 also gives useful advice on requirements for the strength and testing of crib units and the construction of crib walls. Careful quality control during manufacture of the crib units is required especially with regard to concrete cover, the placement of steel reinforcement, concrete mix design, and the dimensional tolerances of individual units.

Many crib walls have failed because of differential settlement of the wall structure. Because of this, all crib walls should be founded at least 300mm below ground level on a cast in-situ reinforced concrete base slab of 150mm minimum thickness over the whole plan area of the wall.

9.3 *BACKFILL*

The crib wall units should always be infilled with a free-draining material placed and well compacted in layers in a way that does not disturb the crib units. Where soil is used, a relative compaction of at least 98% to BS 1377 : 1975 Test 12 should be obtained. Where rock fill is used, the relative density to be obtained should be specified. The strength of the completed wall depends on the standard of this backfilling.

9.4 *PROVISION OF DRAINAGE*

Adequate drainage of the whole crib structure is essential. Many of the failures in crib walls have occurred because material of low permeability was used as backfill, thus developing high static or seepage water pressures. A subsoil drain should be installed at the heel of the wall wherever possible, otherwise ponding may occur.

9.5 *MULTIPLE DEPTH WALLS*

The stability of walls of more than single depth should be checked at the changes from single to double and double to triple, etc., to ensure that the resultant force lies within the middle third of each section considered, and that the overturning criterion stated in Figure 22 is met.

9.6 *WALLS CURVED IN PLAN*

Crib walls with a convex front face are much more susceptible to damage by transverse deformations than are concave walls.

CHAPTER 10

SETTLEMENTS ADJACENT TO LARGE EXCAVATIONS

10.1 GENERAL

The formation of large excavations causes movements of the surrounding ground and settlement of adjacent ground surfaces, associated services and structures. The magnitudes of these settlements and their distribution depends on the dimensions of the excavation, the support system employed, the sequence and timing of the works, and most importantly the characteristics of the surrounding soil and the quality of workmanship involved.

The process of excavation reduces the vertical load on the excavation base and the horizontal stress on the sides. The underlying ground moves upward and the ground alongside tends to move inwards. This inwards movement occurs even at levels below the base of the excavation. The base heave and inward lateral movement cause settlements of the surrounding ground surfaces and structures. The magnitude and distribution of these settlements may cause damage to adjacent structures and services.

10.2 MINIMISING SETTLEMENTS

Prevention of all settlements is virtually impossible, because some of the movements causing them occur before support can be installed. The stiffness of ordinary soldier piles or heavy section steel sheet piling is not usually large enough to have a significant effect on the magnitude of the lateral wall movement. Lateral movements of walls of these types may be limited by the insertion of supports such as struts or anchors, at relatively close vertical spacing as soon as possible after excavation. In addition, good workmanship and detailing is required so that soil movement into the excavation is minimised, unfilled voids are not left outside the supports, and losses of fines through seepage and changes of water table are prevented.

Very stiff walls such as caisson and diaphragm walls will, to some extent, reduce the inward lateral movements associated with an excavation. Under comparable conditions, the intervals between supports need not be as small as for more flexible types.

It should also be noted that settlements of the same magnitude as those that occur during excavation can occur on removal of struts. Therefore, care should be taken during this stage of the construction operation to ensure that these movements are minimised. The location of the supporting system components needs to take account of works which have to be completed prior to the removal of the supports. Where possible, allowance should be made for changes in construction sequence and methods.

10.3 PREDICTIONS OF SETTLEMENTS

The estimation of settlement around excavations is a considerable exercise in engineering judgement. There have been several well documented case histories published on this topic, and a very useful summary of some of these has been provided by Peck (1969) and updated by O'Rourke et al (1976). Figure 28 gives the approximate magnitude of settlements likely to occur in the vicinity of excavations. It is based on North American experience and should only be used for approximate guidance in residual soils.

The construction of the Mass Transit Railway in Hong Kong has provided some data for Hong Kong conditions, and a number of papers have been published. It is not recommended that this data be used to predict movements in Hong Kong until additional case histories and supportive evidence is available to prove the general values given.

Morton et al (1980) described the methods of construction, and the ground conditions encountered, and they provided information on existing buildings adjacent to the railway and a description of the measures adopted to monitor building settlements. The data collected were separated into that resulting from station wall installation, that from dewatering, and that from station box excavation (Figure 29). The following observations were made :

(a) *Settlements resulting from station wall installation* - The magnitude of the ground and building settlements which occurred during the installation of permanent walling systems were under-predicted and were the most significant phenomena encountered during construction. In the case of diaphragm walling, settlements of up to 63mm were recorded. Davies & Henkel (1980) suggested that such movements are due to lateral swelling of the decomposed granite during construction of individual panels of the wall.

(b) *Settlements resulting from dewatering* - Settlements occurred as a result of dewatering for hand dug caisson construction and during dewatering to facilitate station box excavation. It was estimated that the resulting settlements varied between 8mm per metre of drawdown for buildings on shallow foundation and 3mm per metre for those with piled foundations.

(c) *Lateral wall movements and settlement during excavation* - The maximum observed lateral movements of station walls was between 9mm and 43mm for secant piles, and 18mm and 58mm for diaphragm walls. The movement of the station walls, as measured by inclinometers, occurred to their full depth, even though the walls in some cases penetrated to some 30m or more, and movements of the toes of up to 20mm were recorded.

Despite the relatively large wall deflections, building settlements were low, and ratios of maximum lateral wall movement to building settlement of the order of 4:1 were reported.

Whilst the causes of settlement of adjacent buildings differed from site to site, the total settlements recorded were related to their foundation depths. Figure 29 shows the relationship between total building settlement and depth factor.

CHAPTER 11

SOME ASPECTS OF REINFORCED CONCRETE DESIGN AND DETAILING

11.1 INTRODUCTION

This chapter does not aim to cover all aspects of reinforced concrete design as it applies to retaining walls. There are, however, several aspects of the design and detailing which are not adequately covered in the commonly available literature or present Codes and Regulations, and some guidance is given here on these. In particular, the junctions between members are often poorly detailed and suggestions are contained in Section 11.9 for improvements.

Reference should be made to comprehensive publications on reinforced concrete (e.g. Scott et al, 1965; Park & Paulay, 1975) for complete details of concrete retaining wall design and detailing.

11.2 GENERAL NOTES

11.2.1 *Codes*

Reinforced concrete structural design should be in accordance with the appropriate standard currently used in Hong Kong; either the Building (Construction) Regulations Cap. 123 (Hong Kong Government, Buildings Ordinance), or Chapter 4, Volume V of the PWD Civil Engineering Manual (Public Works Department, Hong Kong, 1977).

11.2.2 *Ultimate Strength or Limit State Design*

The Code being used will specify the load factors or partial factors to be used. A serviceability limit state analysis should always be made to ensure that the limits given in Chapter 4, Volume V of the PWD Civil Engineering Manual are not exceeded.

11.2.3 *Cover to Reinforcement*

Particular attention should be given to the cover of reinforcement, both in the detailing and during construction. Blinding concrete should always be used on soil-like materials.

11.3 TOE DESIGN

Shear in a toe is usually the critical loading case. The critical section of the toe may be taken at distance 'd' out from the face of the support as shown in Figure 32. The detailing of the curtailment and anchorage of reinforcement is important (see Section 11.8).

11.4 STEM DESIGN

11.4.1 *Stem Loading*

For the stem design cantilever and counterfort walls, it is normal practice to take the earth pressure acting on the vertical plane through the rear of the heel as being projected onto the stem (see Figure 1). However, in nearly all walls, the earth pressure acting on the structural section of the wall is different from this, because of the lateral pressures that develop during the compacting of the backfill. Such lateral pressures are usually much higher than active and can be higher than at-rest pressures. The magnitude of such lateral pressures is discussed in Sections 3.10 & 3.11.

Therefore, in designing stem of a wall the earth pressures from compaction should always be calculated. In many cases, this will be the critical loading. There is little evidence to show that the deflection of cantilever walls will reduce the compaction pressures. (See Section 3.11).

11.4.2 *Bending Moments and Shear Forces in the Stems of Counterfort Walls*

The bottom of a stem, where it joins the heel, should be reinforced for vertical spanning action in addition to horizontal spanning action. Horizontal steel should be continuous in both faces. Horizontal bending moment variations with height should be catered for by varying the reinforcement spacing in preference to changing the bar sizes.

Shear forces should be calculated at the face of the counterforts. Shear stresses will usually govern the stem thickness.

The bending moments and shear forces in stems should be calculated by methods which properly take into account the fixity of each edge of the stem slab and the distribution of pressures on the slab. Huntington (1961) gives useful guidance on this based on work done by the US Portland Cement Association. Bowles (1977) gives similar information.

11.5 HEEL SLAB DESIGN

11.5.1 Loading

The design loading on the heel slab is shown in Figure 30. The bearing pressures for use in structural design are not the same as those used to check the factor of safety against ultimate bearing failure (Section 6.4). They are normally taken as the bearing pressures at working loads, as follows:

(a) If the resultant passes through the base within the middle third, the toe and heel pressures for structural design may be calculated from

$$P = \frac{V}{BL} \pm \frac{6Ve_b}{B^2L} \qquad \text{.....(23)}$$

where V is the normal component of the resultant loading on the base, B is the base width, and L is the length of wall for which the resultant earth pressure is calculated (usually unity), and e_b is the eccentricity of the load.

(b) If the resultant lies outside the middle third :

$$P_{max} = \frac{2V}{3(\frac{B}{2} - e_b)L} \qquad \text{.....(24)}$$

11.5.2 Heel Slabs for Counterfort Walls

The heel slab for counterfort walls should be designed as a slab spanning in two directions. The references given in Section 11.4.2 may be consulted for this purpose.

As in Section 11.4.2, the critical section for shear is at the face of the counterforts. Again, shear stresses usually govern the heel thickness.

11.6 COUNTERFORT DESIGN

Vertical steel in the counterfort is required to carry the net tensile load from each strip of the heel slab into the counterfort. The main moment reinforcement for the wall is usually concentrated at the back of the counterfort. Horizontal steel in the counterfort is required to carry the net load on each horizontal strip of stem. The detailing of this steel should be done so as to provide adequate anchorage between the stem slab and the counterfort (Figure 31). Consideration should be given to staggering the laps in these anchorage bars.

Cut-off positions for the main tensile steel in the counterforts are shown in Figure 31.

11.7 KEY DESIGN

In general the ratio of depth to thickness of the key should be less than 2.0. It is difficult to predict what the force acting on the key will be. Approximately :

$$\text{Design horizontal load on key} = \text{horizontal loads tending to cause sliding} - 0.4 \times \text{total vertical loads above blinding layer}$$

It may be assumed that this load acts at one-third of the key height from the bottom of key. The key should be detailed in accordance with Section 11.8 & 11.9. Note that tensile stresses are carried from the key into the bottom of the heel slab, and therefore some reinforcement is called for in that area.

11.8 CURTAILMENT AND ANCHORAGE OF REINFORCEMENT

The curtailment of reinforcement in retaining walls is critical. A bar must extend beyond the point where it is theoretically no longer required to allow for inaccuracies in loading and analysis, to allow for inaccuracies in placing bars, and to avoid large cracks at the curtailment section. Such cracks reduce the resistance to shear forces and introduce high peak stresses in the tension reinforcement. It is recommended that the following requirements from Clause 3.11.7.1 of the Code of Practice for the Structural Use of Concrete, CP110 (British Standards Institution, 1972) should be applied to all designs, regardless of the code or regulation being used.

"In any member subject to bending every bar should extend, except at end supports, beyond the point at which it is no longer needed for a distance equal to the effective depth of the member, or twelve times the size of the bar, whichever is greater. A point at which reinforcement is no longer required is where the resistance moment of the section, considering only the continuing bars, is equal to the required moment. In addition, reinforcement should not be stopped in a tension zone, unless one of the following conditions is satisfied :

(1) the bars extend an anchorage length appropriate to their design strength ($0.87f_y$) from the point at which they are no longer required to resist bending, or

(2) the shear capacity at the section where the reinforcement stops is greater than twice the shear force actually present, or

(3) the continuing bars at the section where the reinforcement stops provide double the area required to resist the moment at that section.

One or other of these conditions should be satisfied for all arrangements of ultimate load considered."

Although the above clause is worded in terms of ultimate load design its provisions can clearly be used for working stress design as well.

11.9 DETAILING OF REINFORCED CONCRETE CORNERS AND JOINTS

11.9.1 *Background*

Many reinforced concrete walls involve cantilevers that meet at right angles. At this junction, there is the combination of peak bending moments and peak shear forces. Such cantilevers and corners must be carefully detailed to avoid wide crack width, and so ensure the strength and serviceability of the structures. Some guidance on suitable detailing is given in this Chapter.

Research work by Nilsson & Losberg (1976) has shown that reinforcement details commonly used in cantilever walls have ultimate capacities significantly less than are usually assumed in calculations, and they result in excessively wide corner crack widths at what would normally be working loads. For example the commonly used detail shown in Figure 32a, even with the addition of diagonal stirrups, had a failure moment of less than 80% of the calculated ultimate capacity, and at a load of 55% of the calculated ultimate capacity, there was a corner crack 2.5mm wide. The detail shown in Figure 32b, while having sufficient ultimate moment capacity, had a corner crack 5.3mm wide at a load of 55% of the calculated ultimate capacity. Other commonly used details had an even worse performance. These tests were at relatively small steel percentages of 0.5 to 0.8%. Swann (1969) carried out a similar series of tests at the higher steel percentage of 3% and significantly worse moment capacities were obtained. Such joints should be capable of resisting a moment at least as large as the calculated failure moment in adjacent cross sections. The cracks that form in the inside of corners should have acceptable crack widths for loads in the working range. Also the reinforcement in corners should be easy to fabricate and position, and this should normally avoid the need for stirrups or ties.

For the reinforcement of corners subjected to an opening bending moment, Nilsson & Losberg (1976) recommended that the reinforcement loop from each adjacent part of the structure should be taken out into the corner region, as far as cover restrictions allow, and should then be brought back into the same cross-section adjacent to the inclined reinforcement (see Figures 32(c) and 32(d)). The main reinforcement should be designed on the basis of the moments in the adjacent sections (M1 & M2), ignoring the effect of reinforcement loop curtailment in the compression zone and the inclined reinforcement. The cross-sectional area of the inclined reinforcement should be approximately one-half the area of the largest main reinforcement. Bars should never be spliced in the corner region.

Normal code requirements regarding the least permissible bending radius and the spacing of reinforcing bars generally result in the dimensions of structural elements being limited. The dimensions of the cross-section should also be chosen in such a way that the following restrictions on the reinforcement percentage are satisfied in order to avoid failure in the corner :

(a) For Swedish deformed bars K_s40 (yield stress 390MPa), steel $\leq$ 1.25%.

(b) For Swedish deformed bars K_s60 (yield stress 590MPa), steel $\leq$ 0.8%.

Note that these steel percentage restrictions are for right angled corners; acute or obtuse corners have lower steel percentages. The restriction are based on tests using concrete with a cube strength of 30MPa.

These values for the maximum steel percentage may be interpolated or extrapolated with regard to the yield strength of other steel grades. For example, for reinforcing steels to BS 4449, which are commonly used in Hong Kong, the following percentages apply :

	Characteristic Strength	Maximum steel
Deformed high yield bars	410 MPa	1.20%
Mild steel	250 MPa	1.56%

11.9.2 Reinforcing Steel Detailing Recommendations

Based on the recommendations in Section 11.9.1, the corners in retaining walls should be reinforced according to the general solutions given in the following paragraphs.

When the length of the toe is less than the stem thickness, the joint should be reinforced as a corner subjected to an opening moment. The reinforcement in the base slab should be taken out into the toe as far as the cover requirement permits (see Figure 32(c)).

When the length of the toe is greater than the stem thickness, and the length of the toe is sufficient to provide adequate anchorage length, reinforcement can be as in Figure 32(d). The concrete Code or Regulation requirements regarding bending radius, spacing of bent bars and cover should be borne in mind. To limit corner crack widths, inclined reinforcement with cross-sectional area approximately one half the area of the largest main reinforcement should be used. The limitations on steel percentage given in Section 11.9.1 apply only to the main reinforcement, and the diagonal bars should not be included in this percentage.

Haunches in the re-entrant corner, accommodating substantial diagonal flexural bars, force the plastic hinge away from the face of the joint. This improves the anchorage of the main tensile steel where it enters the joint. The increased internal lever-arm within the joint, in turn, reduces the internal tensile force. Haunching would allow the use of higher steel percentages, but Nilsson & Losberg (1976) make no specific recommendations on allowable steel percentages for haunched right angled corners.

For large joints with up to 0.5% steel, Park & Paulay (1975) recommended the use of diagonal bars across the corner equal in area to 50% of the main reinforcement.

Above 0.5% of steel, they proposed that radial hoops (Figure 32(e)) be provided, the area of one radial hoop being given by :

$$a_{sj}* = \left(\frac{\rho - 0.005}{\rho} \right) \left[\frac{f_y}{f_{yj}} \sqrt{1 + \left(\frac{h_1}{h_2}\right)^2} \right] \frac{A_{s1}}{n} \qquad \text{.....(25)}$$

where $\rho = \frac{A_{s1}}{b.d_e}$ in the critical member,

n = no. of legs.

A_{s1} = area of steel limiting the magnitude of the moment that can be applied to the joint,

f_{yj} = yield stress of radial hoops.

It should be emphasised that problems of construction may arise because of steel congestion at such corners, and it is usually a better solution to thicken the concrete sections involved.

Where the backfilled faces of a retaining wall meet at an acute angle in plan, then similar considerations to those above should be given to the detailing of the reinforcing steel. Additional horizontal reinforcing steel will be required in the outside face of the wall.

11.10 JOINTS

11.10.1 Vertical Joints for Longitudinal Movement

Vertical joints are required in retaining walls to minimise the effects of temperature changes and shrinkage, and because of construction stages. In reinforced concrete walls, vertical construction joints with V-notches at the face should be provided at sections preferably not over 10m apart, together with reinforcement through the joints. Expansion joints with grooved shear keys should be provided not more than 30m apart, the reinforcement not being carried through such joints. In gravity concrete walls, similar expansion joints should be provided, preferably not more than 10m apart. Where the water table is high, water stops should be provided at all construction and expansion joints.

Where there are large temperature variations, expansion joints may require resilient jointing material to allow movement.

Sections where there is a substantial change in wall stiffness or wall type (e.g. counterfort to cantilever), or where the nature of the foundation changes (e.g. from fill to rock), require careful detailing. At such locations, it is usually possible to work out the direction of movement that may occur and to provide adequate clearance to accommodate the movements. It is usually best to provide a structural separation, rather than to attempt to reinforce the junction to take the bending moments and shears involved.

11.10.2 Horizontal Construction Joints

The standard of roughness and clean-up on horizontal construction joints should be clearly specified and controlled. Keys in such joints should be avoided, and water stops should be provided in joints below the water table.

The construction joint at the base of a cantilever stem should always be detailed as being at least 100mm above the heel slab, to enable the concrete formwork to be held during construction.

In the stem of a wall, the position of all construction joints should be carefully considered from the point of view of appearance as well as structural performance (see Section 11.12).

11.11 CONTROL OF CRACKING

To prevent unacceptable cracking of retaining structures the following steps should be taken, in addition to normal good quality concrete practice :

(a) Provide shrinkage and temperature reinforcement. This steel should be in accordance with Chapter 4 of the PWD Civil Engineering Manual to ensure that the crack widths given in that chapter are not exceeded. Note that there is a relationship between the reinforcing bar size, steel percentage and crack width involved. In no case should the steel percentage used be less 0.3% of the gross concrete area of the wall both horizontally and vertically. In the stem of the wall exposed to the air two thirds of this steel should be on the outside face and one third of the steel on the earth face.

(b) Specify that the concrete placing and temperature is to be kept as low as practical, especially in the summer period.

(c) Specify successive bay, not alternate bay, construction.

(d) Specify early curing for the purpose of cooling, so as to minimise the heat rise.

(e) Specify good quality concrete and, where appropriate, limit the cement content.

(f) Additional protection against cracking can be given by painting the earth face of a wall with, for instance, two coats of asbestos filled bituminous or asphaltic paint.

11.12 APPEARANCE OF RETAINING WALLS

Retaining structures are very dominant forms on the urban and rural landscape. Careful design can make a considerable improvement to the appearance of a retaining wall without, in many cases, influencing the cost of the wall.

The aspects and features of retaining walls that are important to their aesthetic impact are :

(a) wall height,

(b) front face slope of the wall,

(c) slope and surface treatment of backfill behind wall,

(d) longitudinal elevation of wall in relation to plan (poor design can give the appearance of the wall having a 'kink' in it),

(e) concrete surface textures, and the expression and position of vertical and horizontal construction joints (concrete textures can be cheaper than 'smooth' finishes), and

(f) the coping of the wall.

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APPENDIX A

SYMBOL

A	area of drainage material
A'	effective area of base
A_s, A_s1, A_s2, A_s3	area of cross-section of reinforcing steel
B	base width of wall
B'	effective base width
b	distance from crest of slope to foundation
c	cohesion of soil in terms of total stress
c_b	adhesion at base
c'	cohesion of soil in terms of effective stress
d_e	effective depth of wall stem
D	depth of foundation
e_l, e_b	eccentricity of load on base in the directions of length and breadth respectively
F_s	factor of safety
f	moment arm of vertical component of earth pressure force
f_y	characteristic strength of reinforcement
g	acceleration due to gravity
g_c, g_q, g_γ	foundation ground slope factors
H, H_1, etc.	height of plane on which earth pressure is calculated (from underside of base or bottom of key to ground surface)
H	tangential component of foundation loading
h	distance of resultant force from wall toe
h_c	critical depth of fill where compaction pressures equal active pressure
i	hydraulic gradient
i_w	waviness of rock joint
i_c, i_q, i_γ	bearing capacity inclination factors
K_o	coefficient of earth pressure at rest

K_a	coefficient of active earth pressure
K_p	coefficient of passive earth pressure
K_s	coefficient of subgrade reaction
k	coefficient of permeability
L	length of base
L'	effective length of base
L_h	length of wall heel
L_s	clear span between counterforts
L_t	length of wall toe
M, M_1, M_2, M_3	bending moments for reinforcement design
M_o	sum of moments causing overturning
M_r	sum of moments resisting overturning
N_b	stability factor relating to excavation base failure
N_c, N_q, N_γ	bearing capacity factors
n	moment arm of resultant water force on back of wall
p	equivalent line load due to roller
P_A	active earth pressure force
P_o	'at rest' earth pressure force
P_H	horizontal component of active earth pressure force
P_N	normal component of earth pressure force
P_p	passive earth pressure force
P_T	tangential component of earth pressure force
P_Q	lateral earth pressure due to line or point surcharge load (per unit length of wall)
P_V	vertical component of earth pressure force
P_W	water force due to water in tension crack
P, P_{max}, P_t	pressure for structural design
Q	total load
Q_L	line load

Q_p	point load
q	intensity of load on base or surcharge load
q_{all}	allowable bearing capacity
q_d	flow rate through drain
q_{ult}	ultimate bearing capacity
R, R_a, R_p, R_w	resultant forces
s	shear strength of soil
S	total shearing resistance at underside of base
s_c, s_q, s_γ	foundation shape correction factors
T	thickness of wall stem
t_c, t_q, t_γ	foundation tilt factors
U, U_1, U_2	resultant force due to water pressures
U_{1H}, U_{1V}	horizontal and vertical components of resultant water force
u	pore water pressure
V	normal component of foundation bearing pressure
V	shear force for reinforcement design
W, W_b	weight of backfill
W_t	weight of wall
X	resultant horizontal reaction
y	lateral deformation of retaining wall
Y_o	vertical depth of tension crack in cohesive soil
Z	depth below final fill level
Z_c	depth below final fill level of maximum residual compaction pressure
α	angle of inclination of foundation base
β	angle of inclination of the back of the retaining wall
γ	bulk unit weight of soil
γ'	effective unit weight of submerged soil
γ_w	unit weight of water
γ_{sat}	saturated unit weight of soil

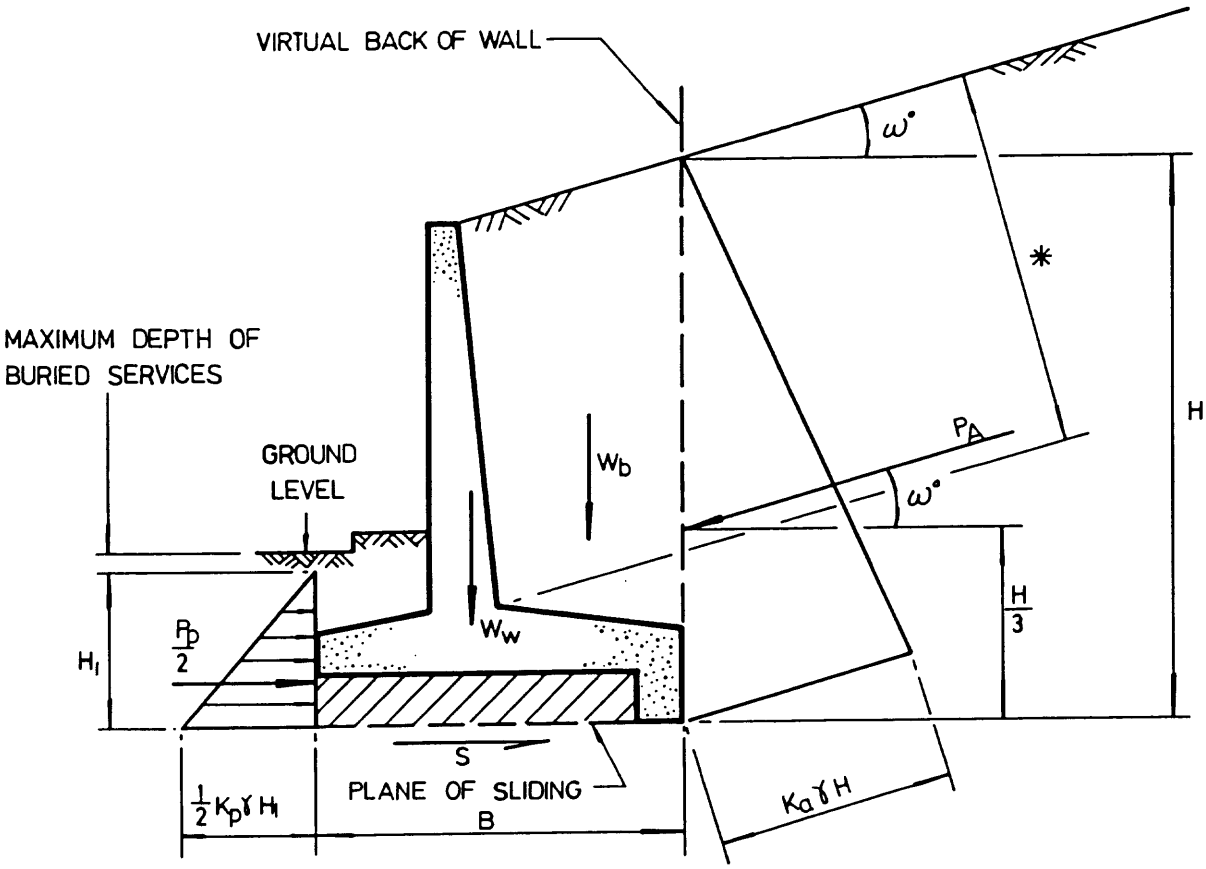
Δ	settlement of wall
δ	angle of wall friction
δ_b	angle of base friction
Θ, Θ_1, Θ_2	location angles for failure plane
Θ_b	angular rotation of foundation base
σ, σ'	total and effective normal stress
ϕ, ϕ'	angle of shearing resistance in terms of total and effective stress
ω	angle of ground slope
τ	shear stress

APPENDIX B

LIST OF TABLES

APPENDIX C

LIST OF FIGURES



SLIDING STABILITY

NOTES

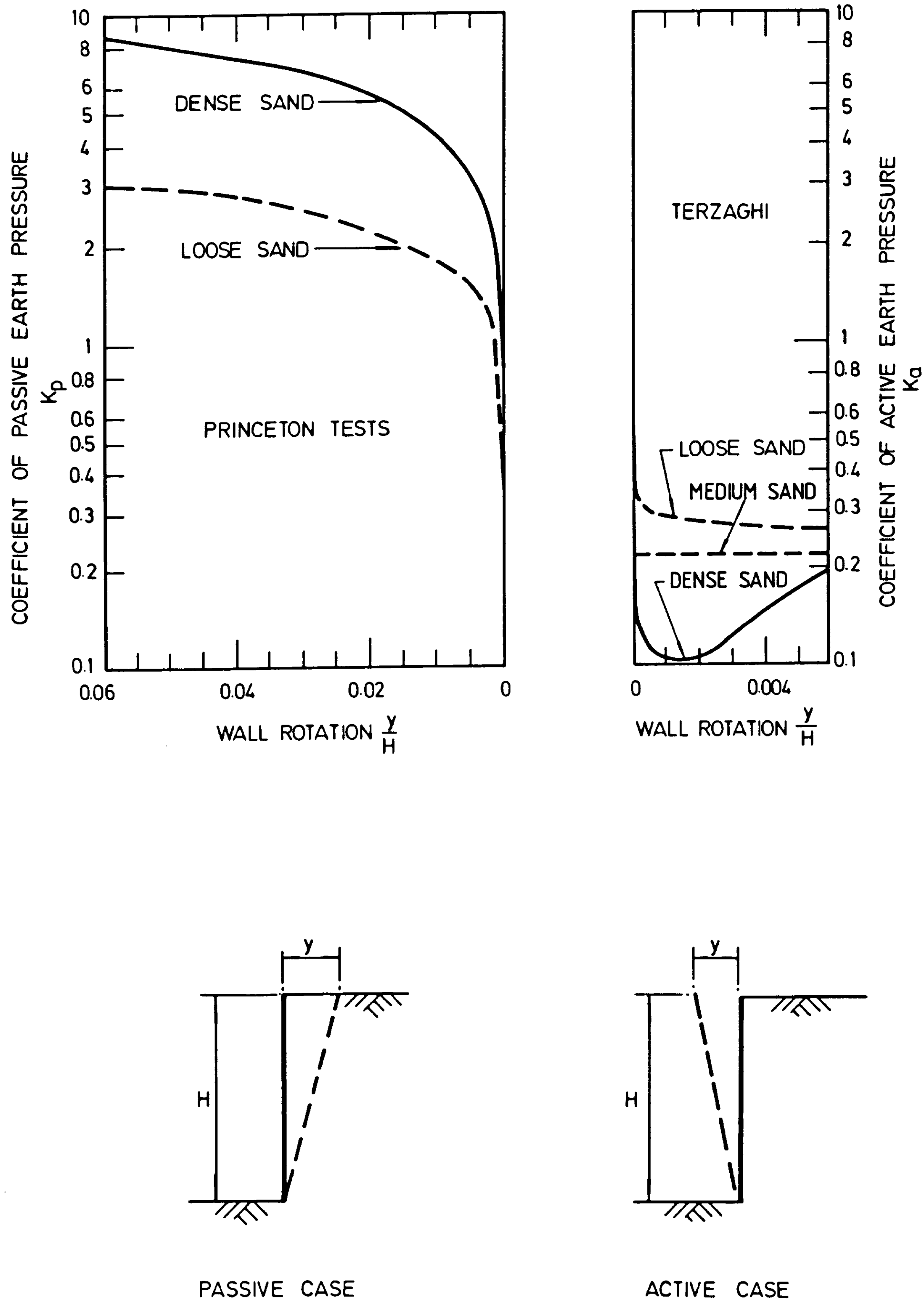
1. Material shaded [hatched] is included in the total weight for calculation of sliding stability.

2. Earth pressure denoted by * is used for the stem design. (See Section 11.4.1).

3. Water forces not shown.

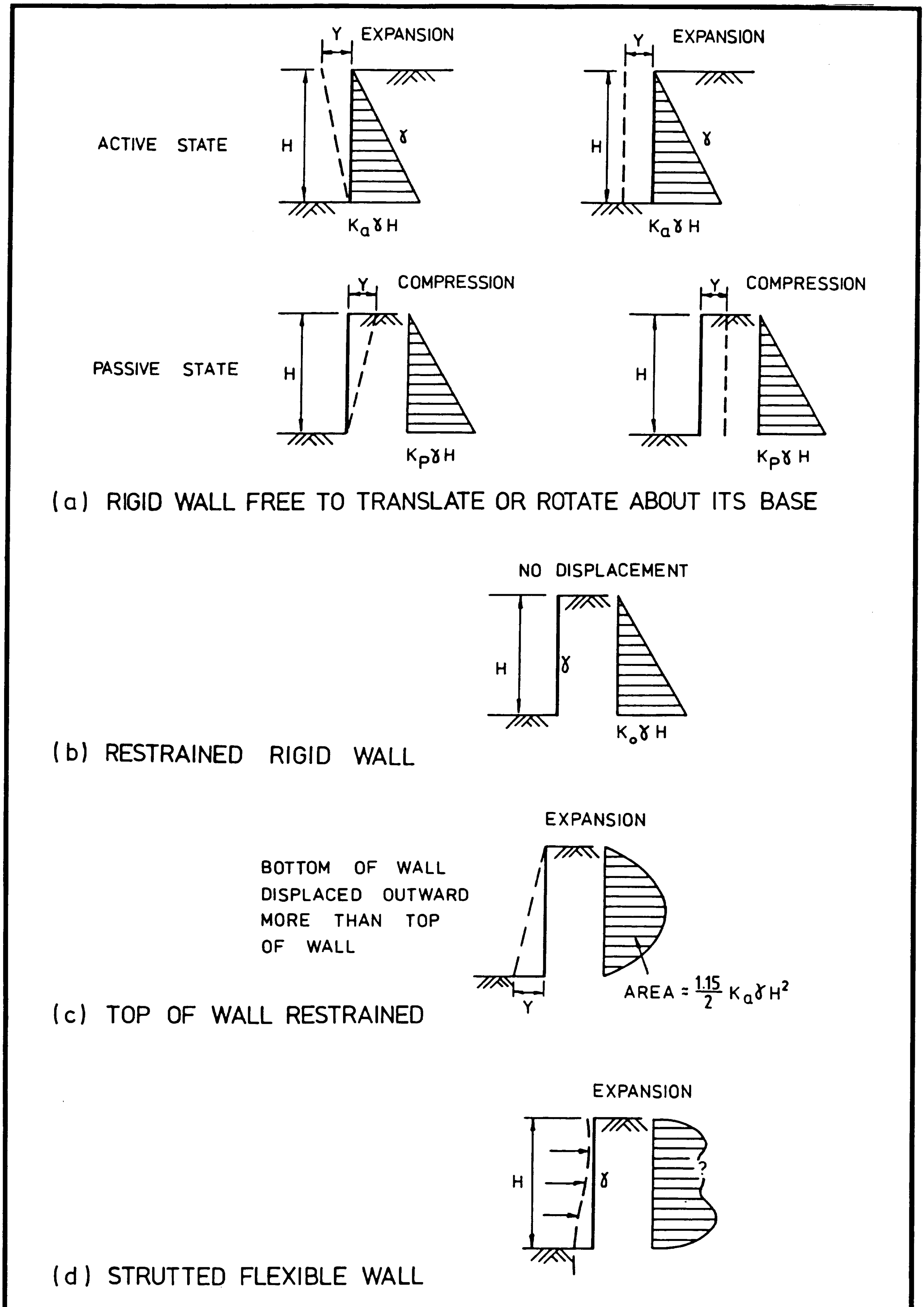
LOADING ON TYPICAL RETAINING WALL (DRAWN FOR RANKINE ASSUMPTION)

FIGURE 1



(after Canadian Geotechnical Society, 1978)

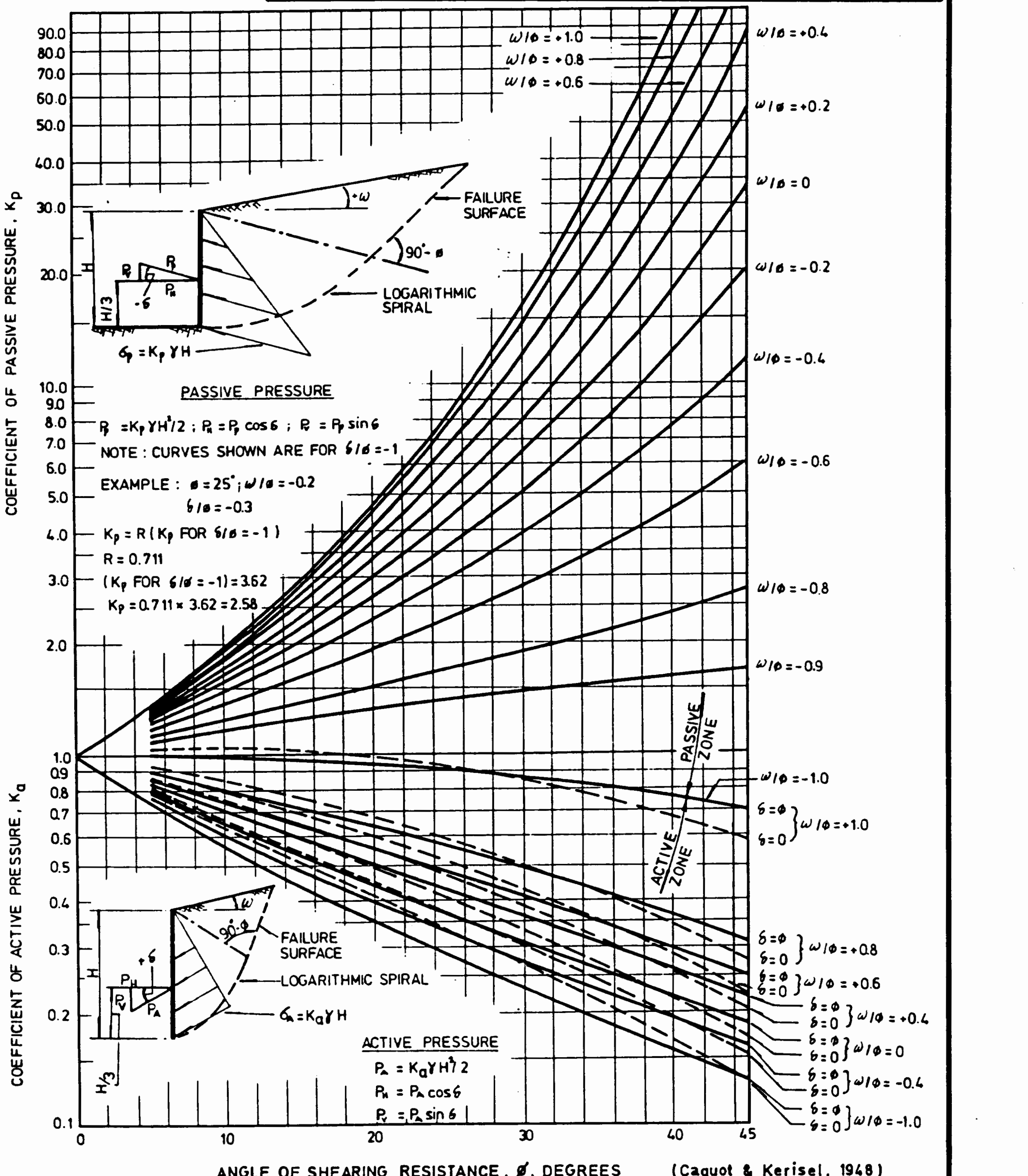
EFFECT OF WALL MOVEMENT ON WALL PRESSURE	FIGURE 2



EFFECT OF DEFORMATION ON EARTH PRESSURES — FIGURE 3

REDUCTION FACTOR, R, OF Kp FOR VARIOUS RATIOS OF δ/ϕ

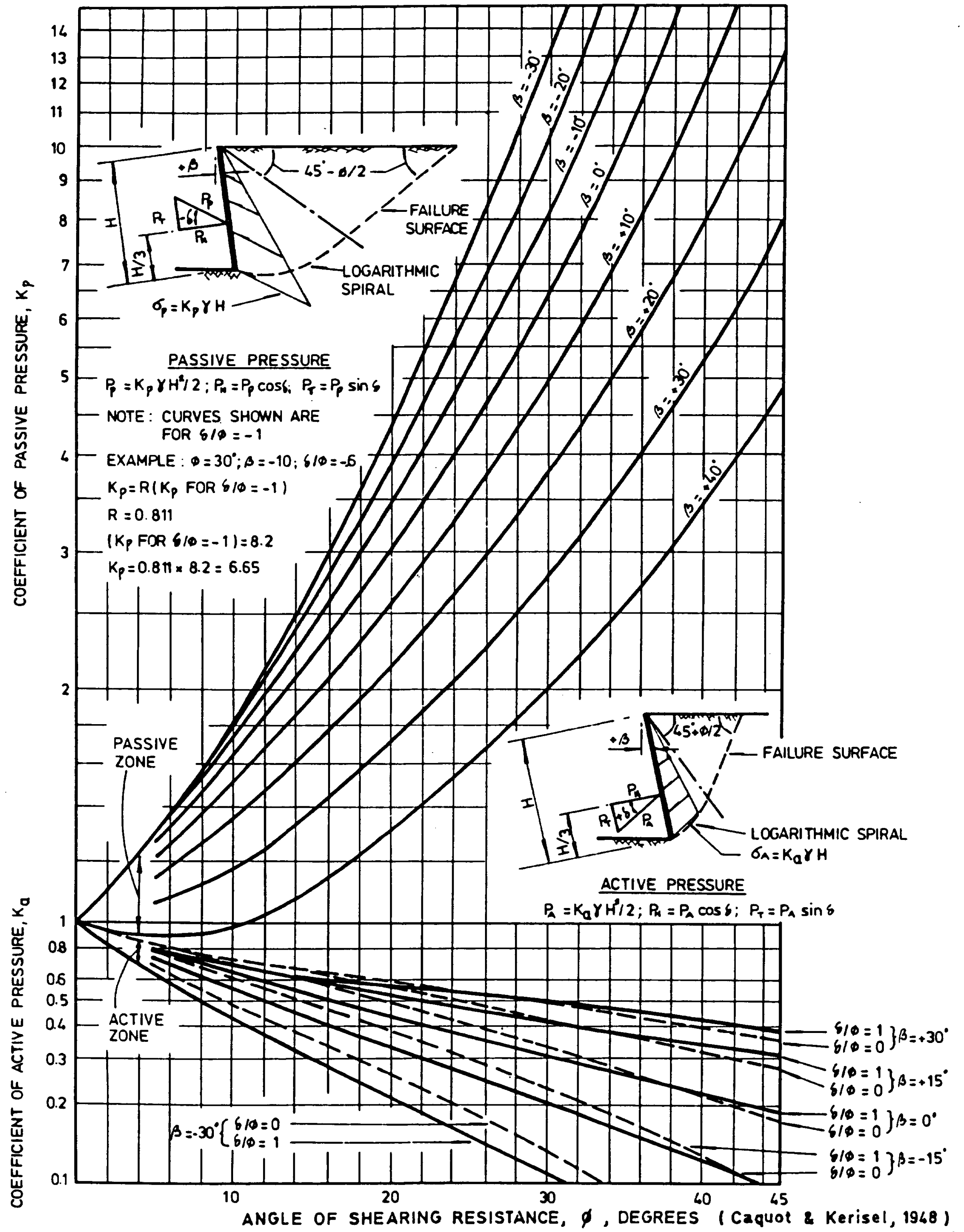
ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174



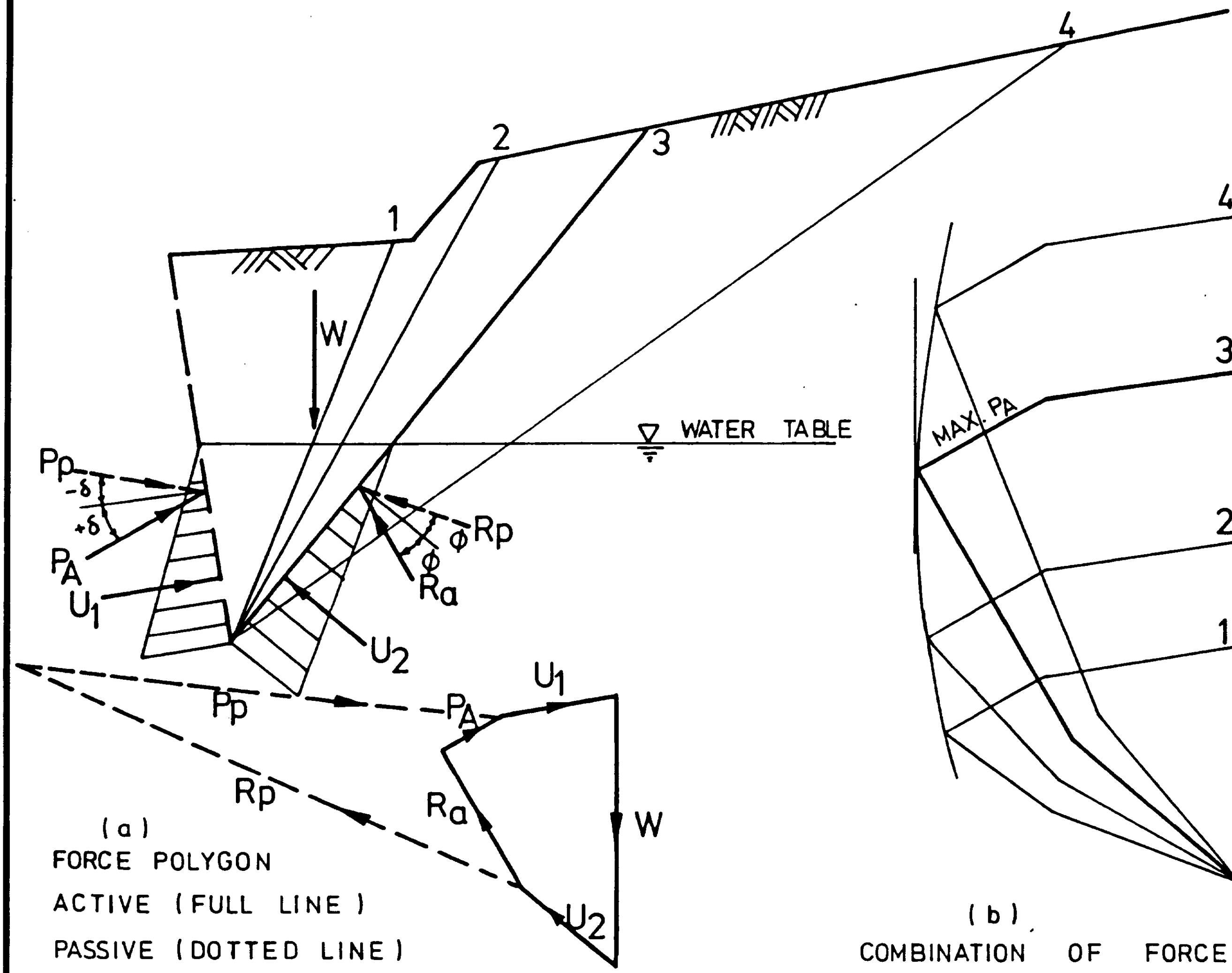
EARTH PRESSURE COEFFICENTS — SLOPING GROUND | FIGURE 4

REDUCTION FACTOR, R, OF K_p FOR VARIOUS RATIOS OF δ/ϕ

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.854
15	.961	.933	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.673
25	.912	.860	.803	.759	.711	.666	.620	.574
30	.878	.811	.746	.696	.637	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174



EARTH PRESSURE COEFFICIENTS — SLOPING WALL **FIGURE 5**



(a)
FORCE POLYGON
ACTIVE (FULL LINE)
PASSIVE (DOTTED LINE)

(b)
COMBINATION OF FORCE POLYGONS TO OBTAIN MAX. P_A
(ACTIVE CASE ONLY)

NOTES

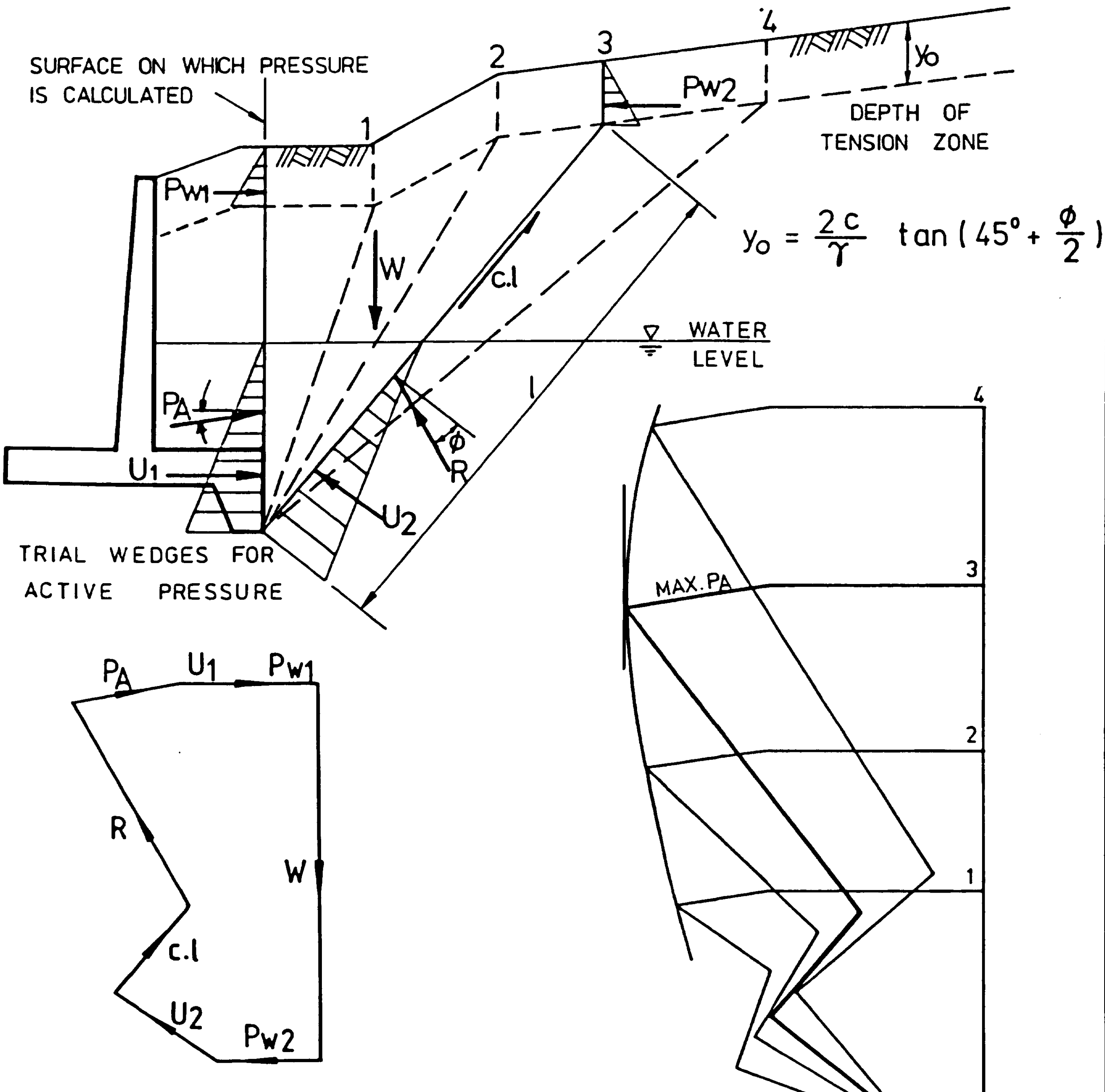
1. The lateral earth pressure is obtained by selecting a number of trial failure planes and determining corresponding values of P_A (or P_P) by drawing a force polygon – see (a). For the active pressure case, the maximum value of P_A is required and for the passive case, the minimum P_P is required. These limiting values are obtained by interpolating between the values for the wedges selected – see (b).

2. Lateral earth pressure may be calculated on any surface or plane through the soil.

3. See Figures 11 and 12 for the point of application of P_A.

4. The trial wedge method may also be used for a level or constantly sloping ground surface, in which case it should yield the same result as that given by Rankine's or Coulomb's equations (whichever is applicable).

TRIAL WEDGE METHOD — COHESIONLESS SOIL | **FIGURE 6**



TRIAL WEDGES FOR ACTIVE PRESSURE

FORCE POLYGON FOR TYPICAL WEDGE

COMBINATION OF FORCE POLYGONS TO OBTAIN MAX. P_A

NOTES

1. The above example shows Rankine's conditions but the same principle applies for Coulomb's conditions. (Adhesion on the back of the wall is ignored).

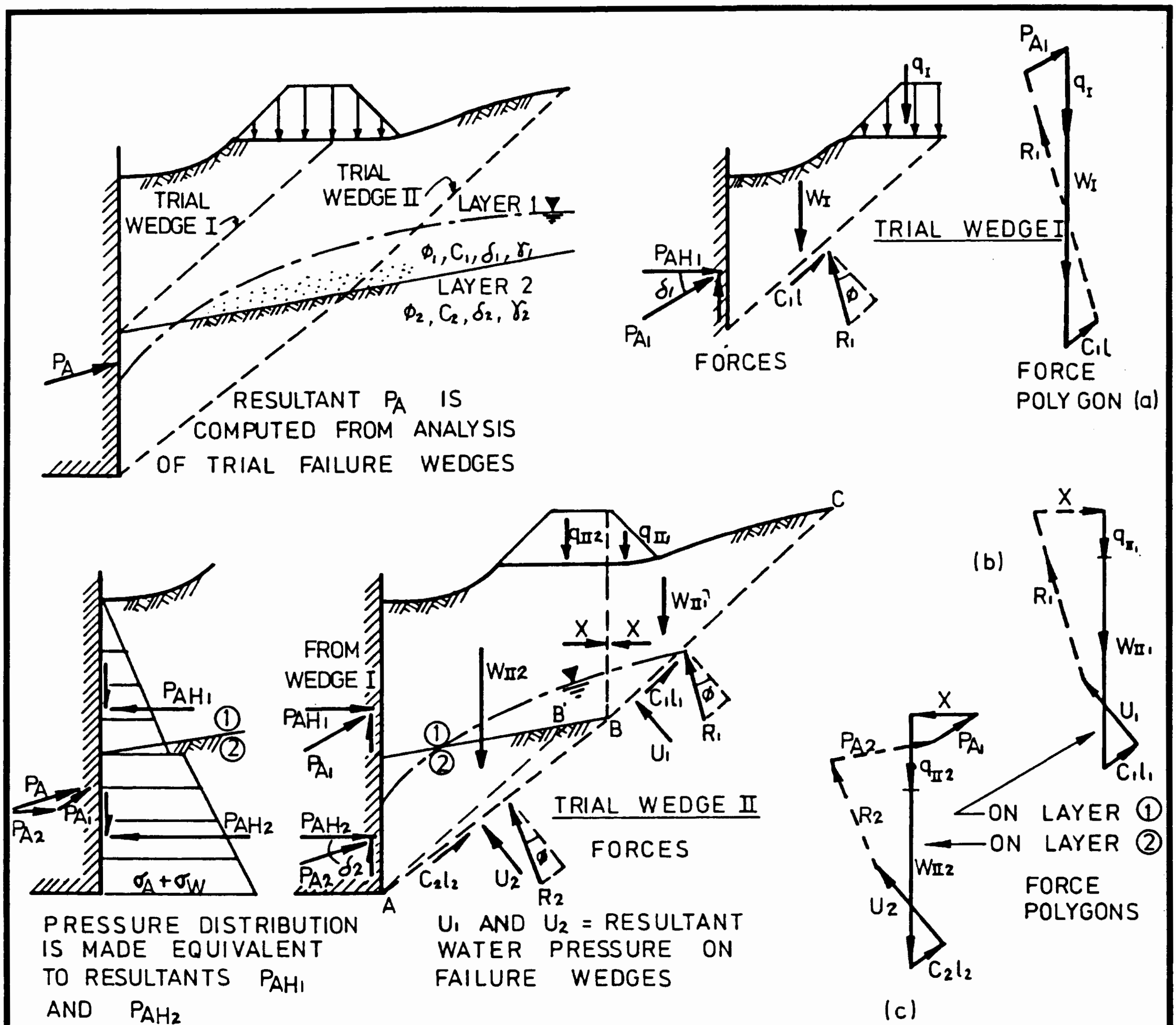
2. For direction P_A see Figure 10 (Rankine's conditions) or Figure 6 (Coulomb's conditions).

3. See Figures 11 and 12 for point of application.

4. See Figure 12 for resultant pressure diagram.

5. The trial wedge method may be used for a level or constantly sloping ground surface.

TRIAL WEDGE METHOD — COHESIVE SOIL | **FIGURE 7**



PROCEDURE

1. Draw trial wedge I in layer (1) (as shown) and obtain $P_{A1\ max}$ by varying the failure plane and drawing the force polygon (a).

2. Draw trial wedge II (as shown) by choosing failure plane AB in layer (2).

3. Find X_{max} by varying the inclination of plane BC from B and drawing the force polygon (b).

4. Using X_{max} draw force polygon (c) and find P_{A2}.

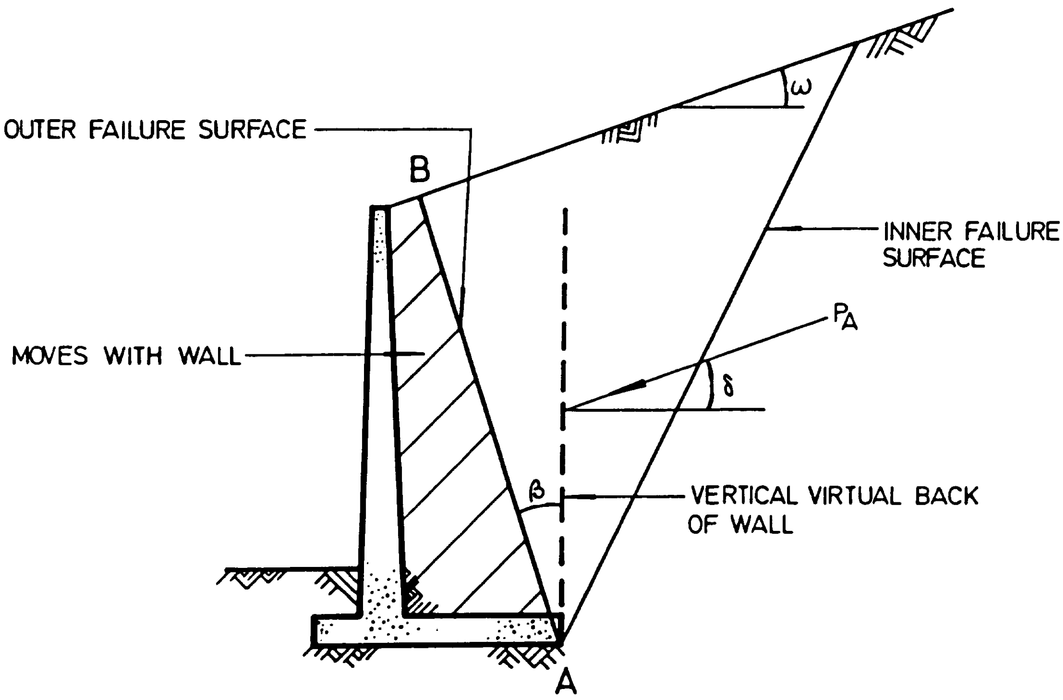
5. Repeat steps 2. to 5. using other trial failure planes AB', etc. until $P_{A2\ max}$ is determined.

NOTE

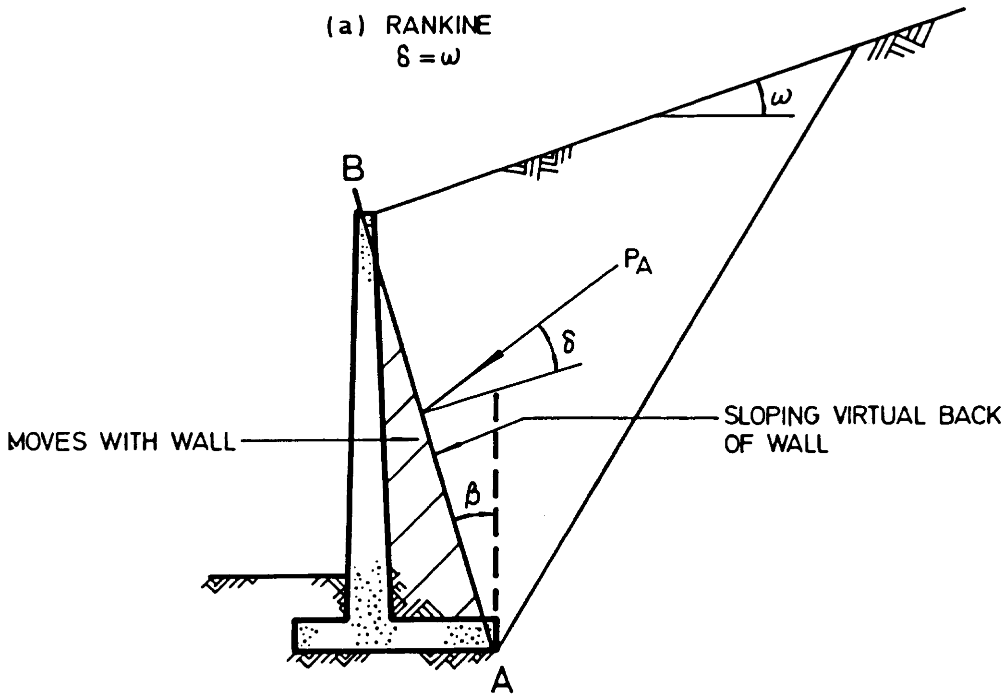
Where layer 2 is rock-like material, such that no earth pressures are exerted against the wall, due account should however be taken of water pressures and joint controlled failure modes.

TRIAL WEDGE METHOD — LAYERED SOIL AND POREWATER PRESSURE (ACTIVE CASE)

FIGURE 8



(a) RANKINE
$\delta = \omega$



(b) COULOMB
$\delta = \phi$

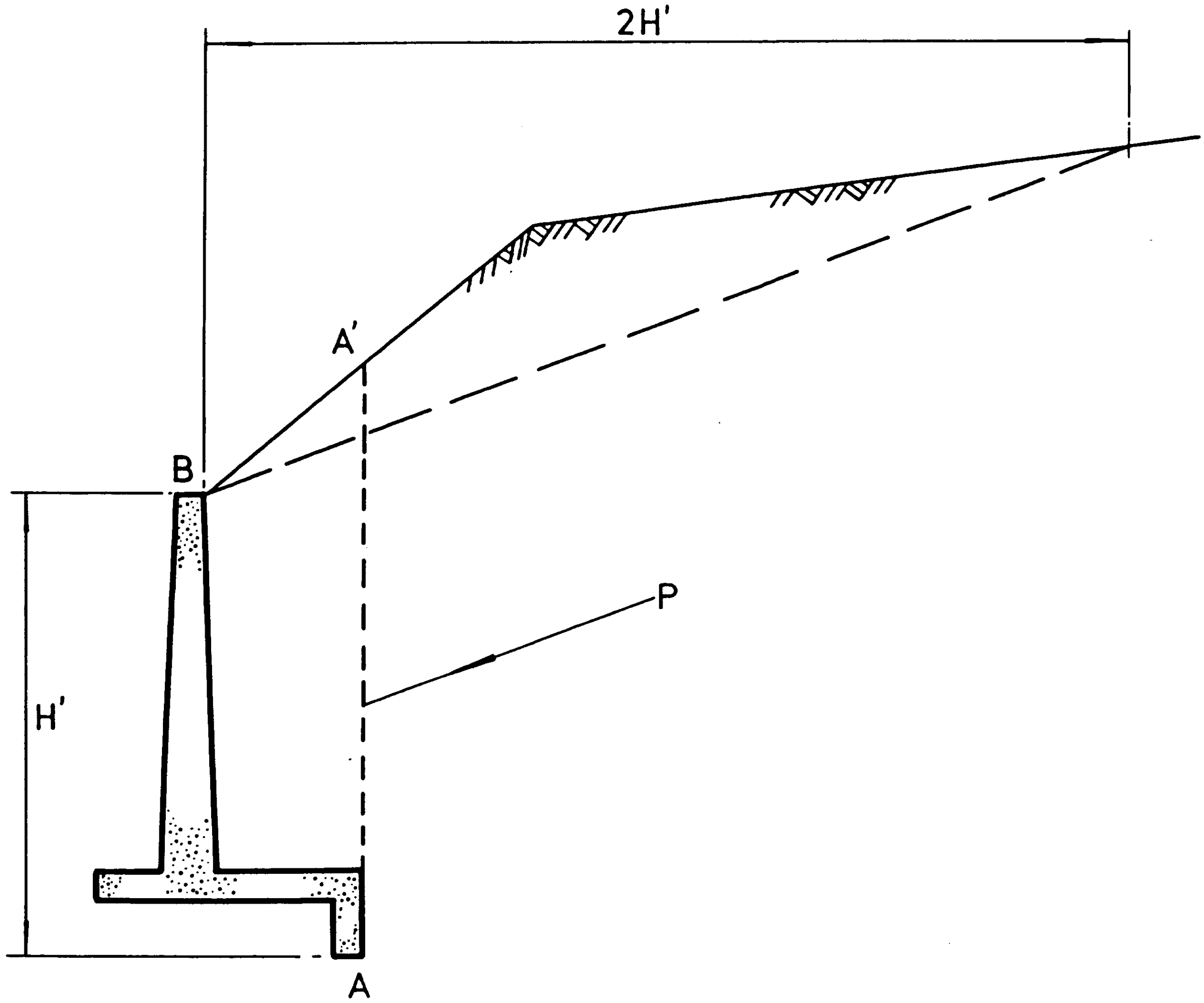
NOTE

(1) If line AB does not intersect the wall, Rankine's conditions apply.

If line AB does intersect the wall, Coulomb's conditions apply.

(2) $\beta = \frac{1}{2}(90 - \phi') - \frac{1}{2}(\varepsilon - \omega)$ where $\sin \varepsilon = \frac{\sin \omega}{\sin \phi'}$

INFLUENCE OF HEEL LENGTH ON ANALYSIS METHOD | **FIGURE 9**



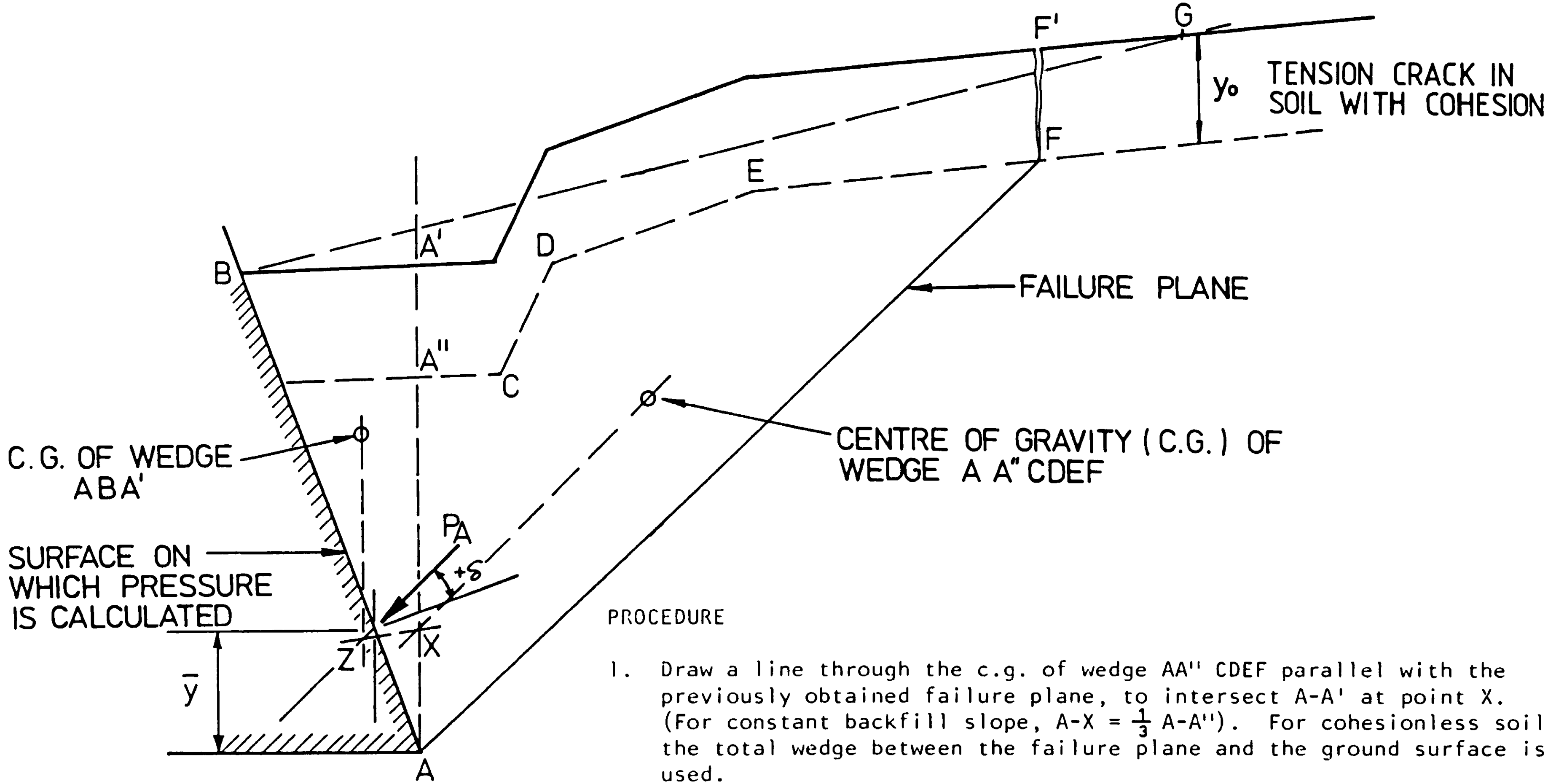
PROCEDURE

1. Draw a line from the point where the ground surface intersects the back of the wall (B) to a point on the ground surface located at a distance equal to 2H' from B.

2. The pressure on A-A' may be assumed to act parallel with this line.

APPROXIMATE METHOD FOR DETERMINATION OF DIRECTION OF RANKINE ACTIVE EARTH PRESSURE

FIGURE 10



PROCEDURE

1. Draw a line through the c.g. of wedge AA'' CDEF parallel with the previously obtained failure plane, to intersect A-A' at point X. (For constant backfill slope, $A\text{-}X = \frac{1}{3}\, A\text{-}A''$). For cohesionless soils the total wedge between the failure plane and the ground surface is used.

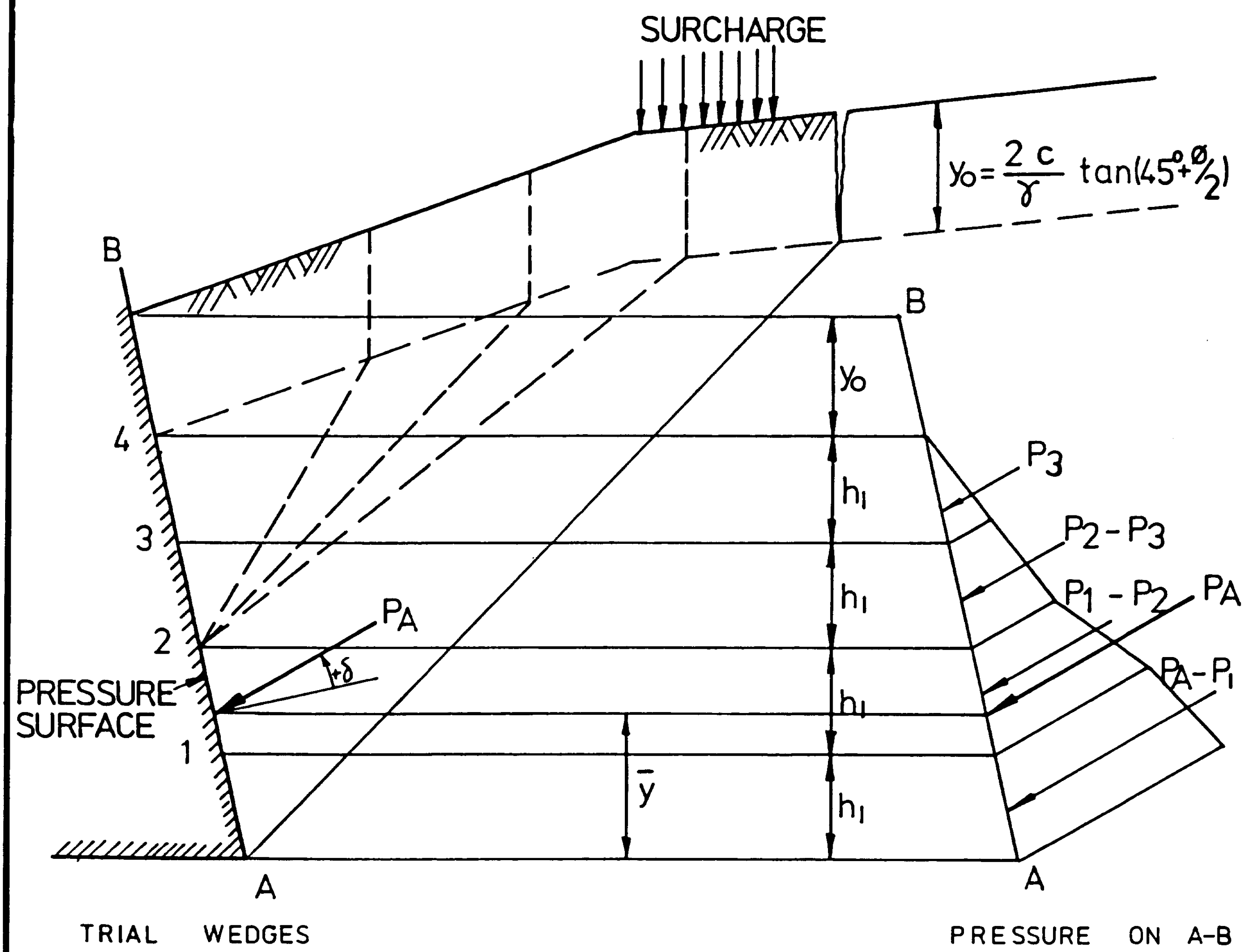
2. Draw a line through point X parallel to BG (see figure 10) and a vertical line through the c.g. of wedge ABA' to intersect at point Z.

3. P_A acts through point Z at an angle of $\delta°$ to the normal to the surface on which the pressure is calculated.

NOTE : a) If the pressure is calculated on a vertical plane steps 2 and 3 are unnecessary as P_A acts through point X.

b) Water forces must be considered separately.

POINT OF APPLICATION OF ACTIVE FORCE | **FIGURE 11**



Use when the ground surface is very irregular or when a non-uniform surcharge is carried.

PROCEDURE

1. Subdivide the line A-4 into about 4 equal parts h_1 (below the depth y_o of tension crack).

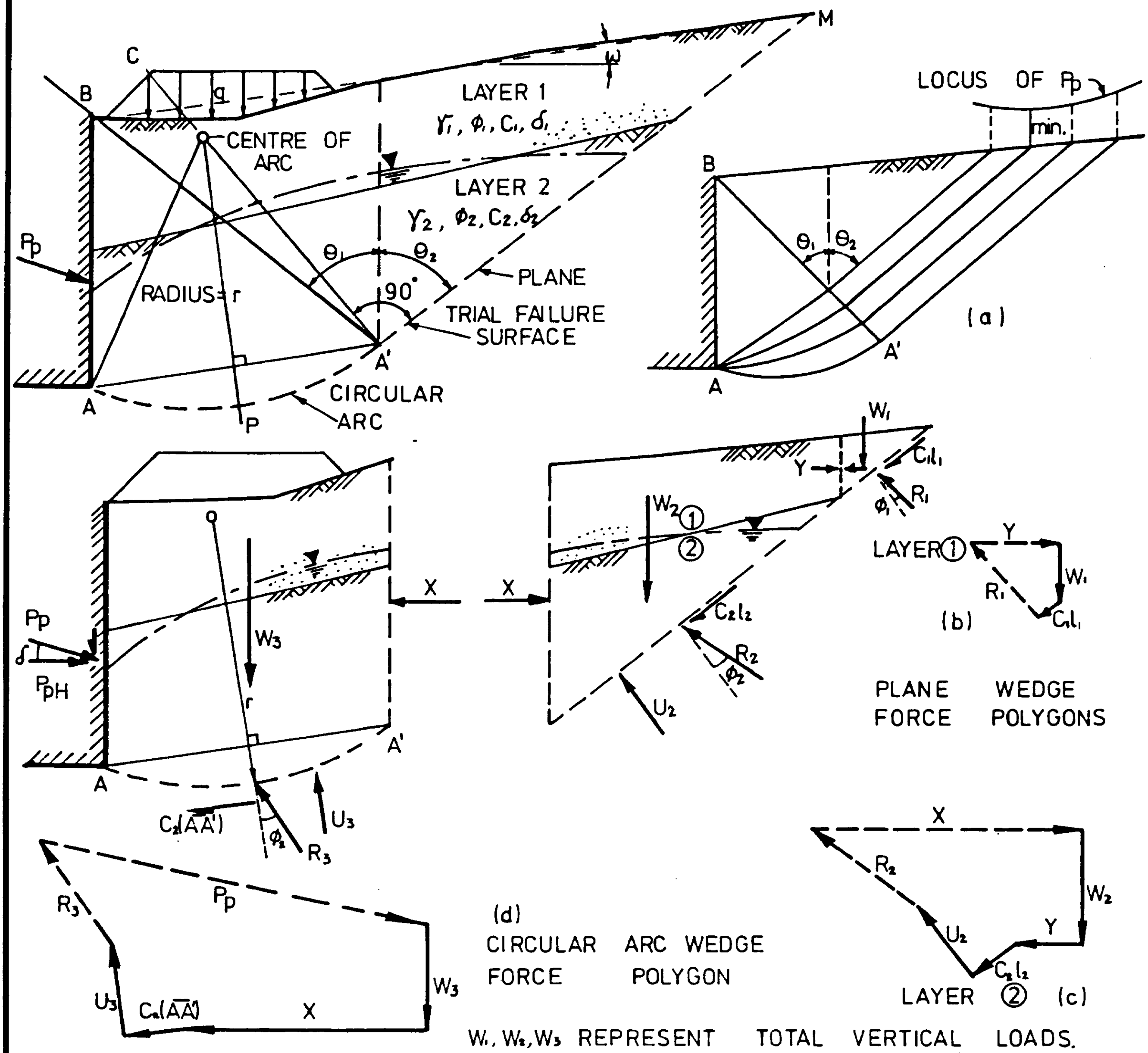
2. Compute the active earth pressures P_1, P_2, P_3, etc., as if each of the points 1, 2, 3, etc., were the base of the wall. The trial wedge method is used for each computation.

3. Determine the pressure distribution by working down from point 4. A linear variation of pressure may be assumed between the points where pressure has been calculated.

4. Determine the elevation of the centroid of the pressure diagram, $\bar{y}$. This is the approximate elevation of the point of application of the resultant earth pressure, P_A.

NOTE : Water forces must be considered separately.

POINT OF APPLICATION OF RESULTANT FORCE AND PRESSURE DISTRIBUTION	FIGURE 12



GENERAL PROCEDURE

1. Determine the directions of surface of sliding BA' and the plane portion A'M of the surface of rupture from the following formulae :

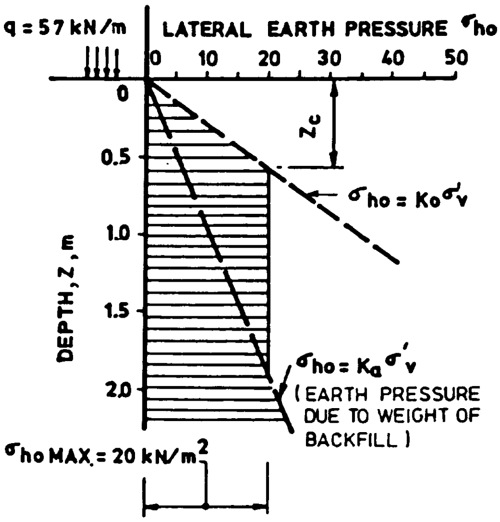
   $\theta_2 = \frac{1}{2}(90° + \emptyset) - \frac{1}{2}(\varepsilon + \omega)$ where, ω = mean ground slope

   $\theta_1 = \frac{1}{2}(90° + \emptyset) + \frac{1}{2}(\varepsilon + \omega)$ and $\sin\varepsilon = \sin\omega/\sin\emptyset$

2. Select a reasonable position for A' and join A'M with a straight line.
3. Construct A'C perpendicular to A'M at A'. Produce a perpendicular bisector OP cutting A'C at O, draw arc AA' with O as centre.
4. Determine U_3 & U_2, resultant of water pressure on each portion of wedge.
5. Compute W_1, W_2 & W_3 and construct force polygons b, c & d in order to obtain Pp.
6. Draw the pressure locus of Pp in (a) for various trial positions of A'.
7. Repeat steps 2-6 with different locations of A' until the min. value of Pp is found.

PASSIVE FORCE BY CIRCULAR ARC METHOD
LAYERED SOIL AND POREWATER PRESSURE

FIGURE 13



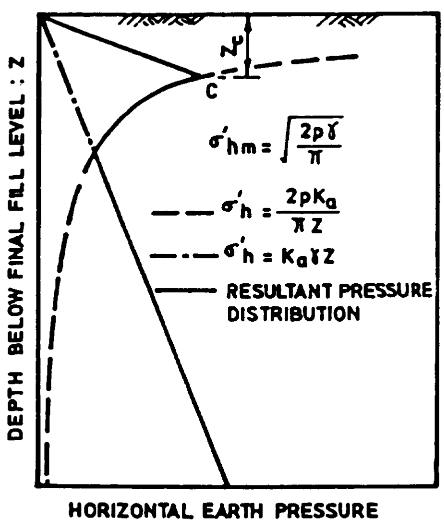
CRITICAL DEPTHS AND EARTH PRESSURE VALUES

COMPACTING MACHINE		CRITICAL DEPTH Z_c (m)	MAXIMUM HORIZONTAL EARTH PRESSURE σ_{ho} MAX. (kN/m²)
10.2 t	SMOOTH WHEEL ROLLER	0.58	20.0
*3.3 t	VIBRATORY ROLLER	0.52	19.0
*1.4 t	VIBRATORY ROLLER	0.35	12.5
400 kg	VIBRATORY PLATE COMPACTOR	0.45	16.0
120 kg	VIBRATORY PLATE COMPACTOR	0.32	11.5

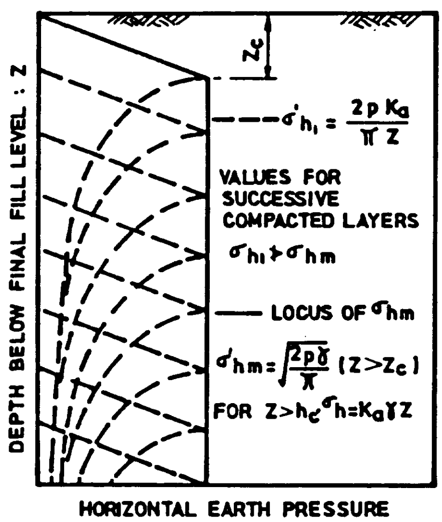
NOTE. DIAGRAM DRAWN FOR 10.2 t SMOOTH WHEEL ROLLER ON FILL, $\phi' = 30°$, $\gamma = 18$ kN/m³

* EFFECTIVE WEIGHT OF VIBRATORY ROLLERS ASSUMED TO BE TWICE TOTAL STATIC WEIGHT.

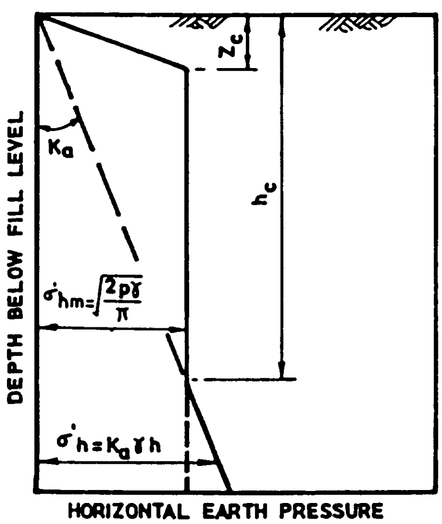
(i) COMPACTION AGAINST UNYIELDING WALLS (BROMS, 1971)



(a) SHOWS INFLUENCE OF COMPACTING SURFACE LAYER OF FILL WHICH WAS PLACED WITHOUT COMPACTION.



(b) SHOWS INFLUENCE OF SUCCESSIVELY COMPACTING LAYERS OF SOIL BEGINNING AT BASE OF WALL.



(C) SHOWS PROPOSED DESIGN PRESSURE DIAGRAM.

σ'_{hm} – MAXIMUM VALUE OF HORIZONTAL STRESS SUSTAINED AFTER COMPACTION.

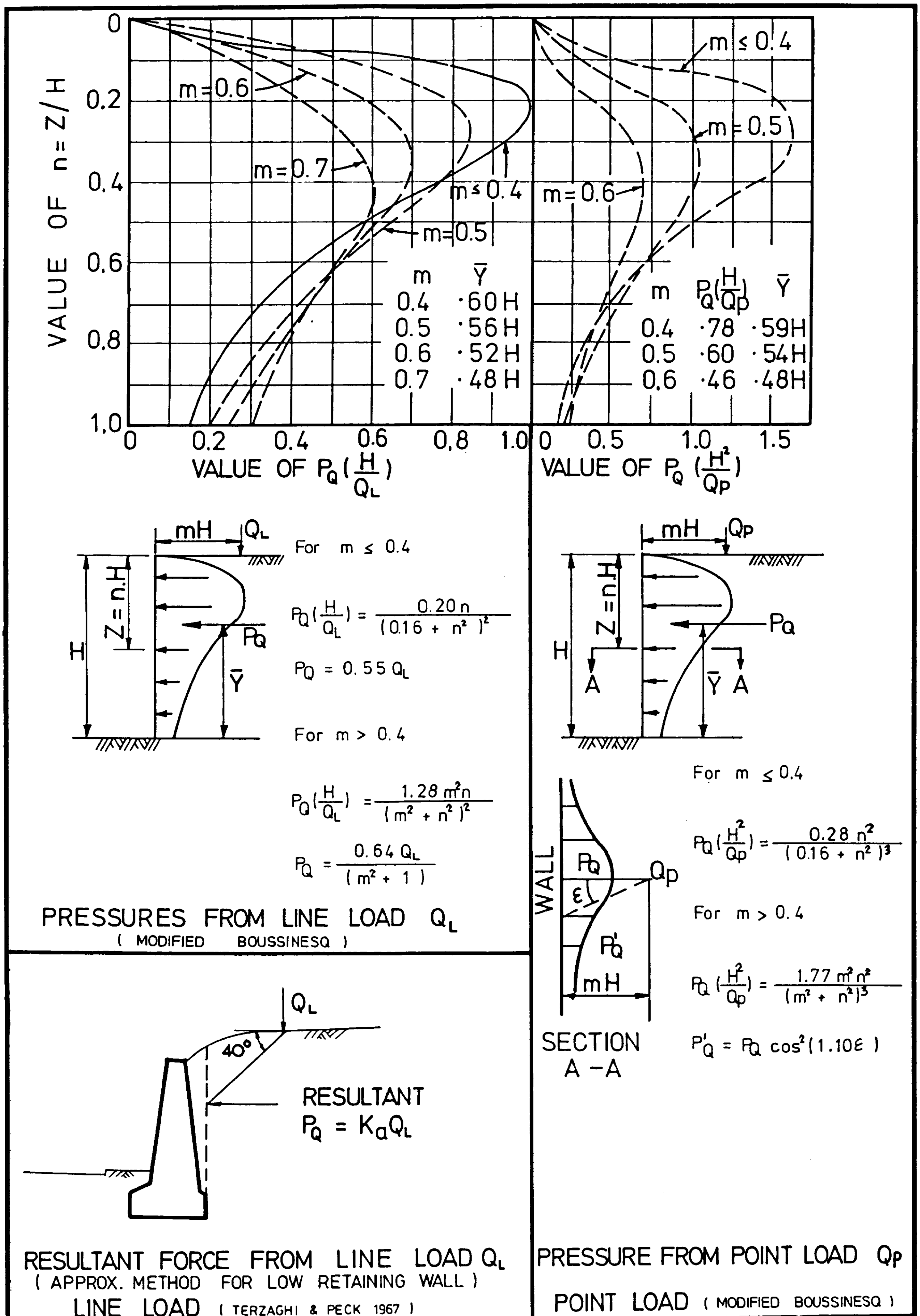
$$Z_c = K_a\sqrt{\frac{2p}{\pi\gamma}}$$

$$h_c = \frac{\sqrt{\frac{2p}{\pi\gamma}}}{K_a}$$

WHERE p = EQUIVALENT LINE LOAD DUE TO ROLLER. FOR VIBRATORY ROLLERS CALCULATE p USING AN EQUIVALENT WEIGHT EQUAL TO DEADWEIGHT OF ROLLER PLUS CENTRIFUGAL FORCE INDUCED BY ROLLER VIBRATING MECHANISM.

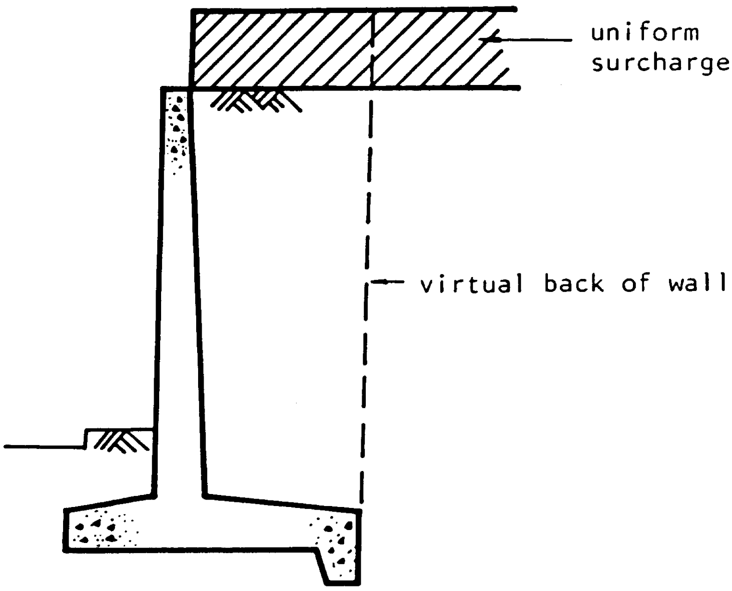
(ii) COMPACTION PRESSURES – DESIGN DATA (INGOLD, 1979)

EARTH PRESSURE DUE TO COMPACTION	FIGURE 14



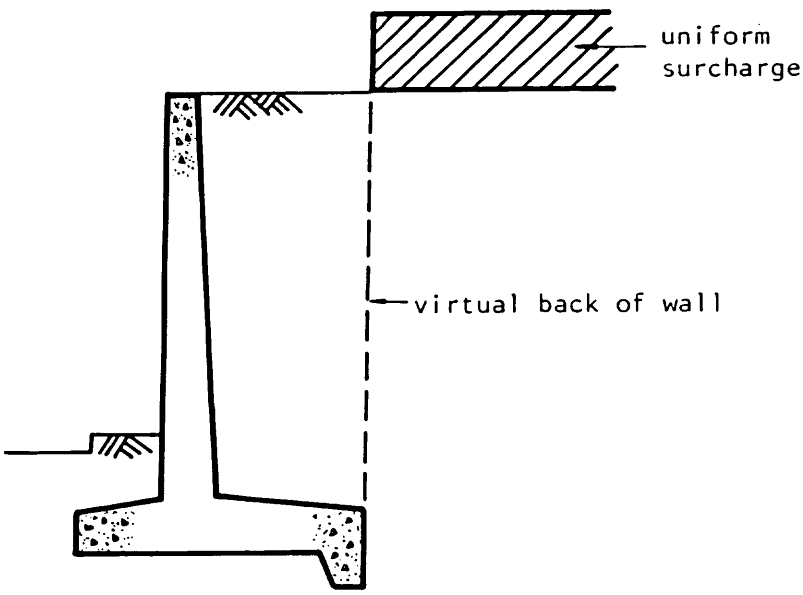
LATERAL LOADS ON WALL DUE TO POINT AND LINE LOAD SURCHARGES

FIGURE 15



LOADING 1

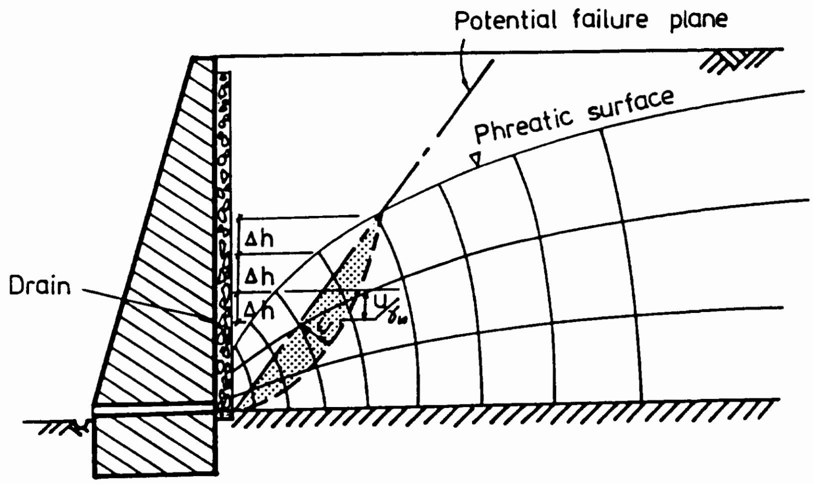
CRITICAL FOR BEARING PRESSURES AND
WALL REINFORCEMENT



LOADING 2

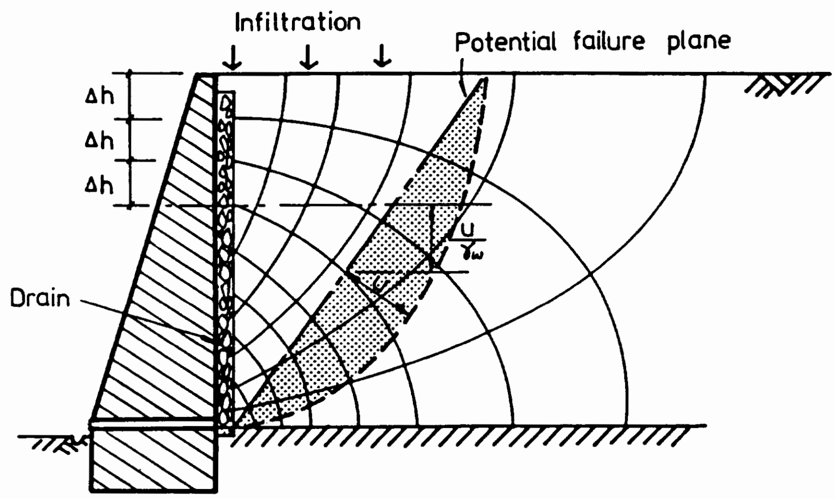
CRITICAL FOR STABILITY

SURCHARGE LOAD CASES	FIGURE 16



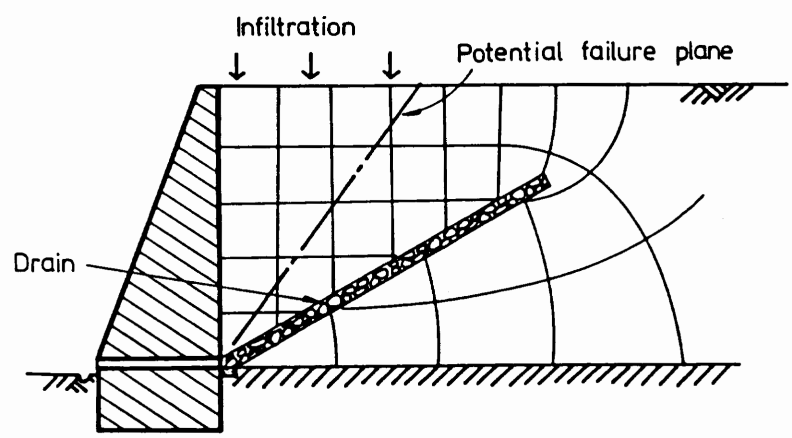
Water pressure distribution on potential failure plane due to steady seepage.

(a) NORMAL STEADY STATE SEEPAGE CONDITION (VERTICAL DRAIN)



Note increase in water pressure on potential failure plane due to surface infiltration.

(b) SURFACE INFILTRATION (VERTICAL DRAIN)



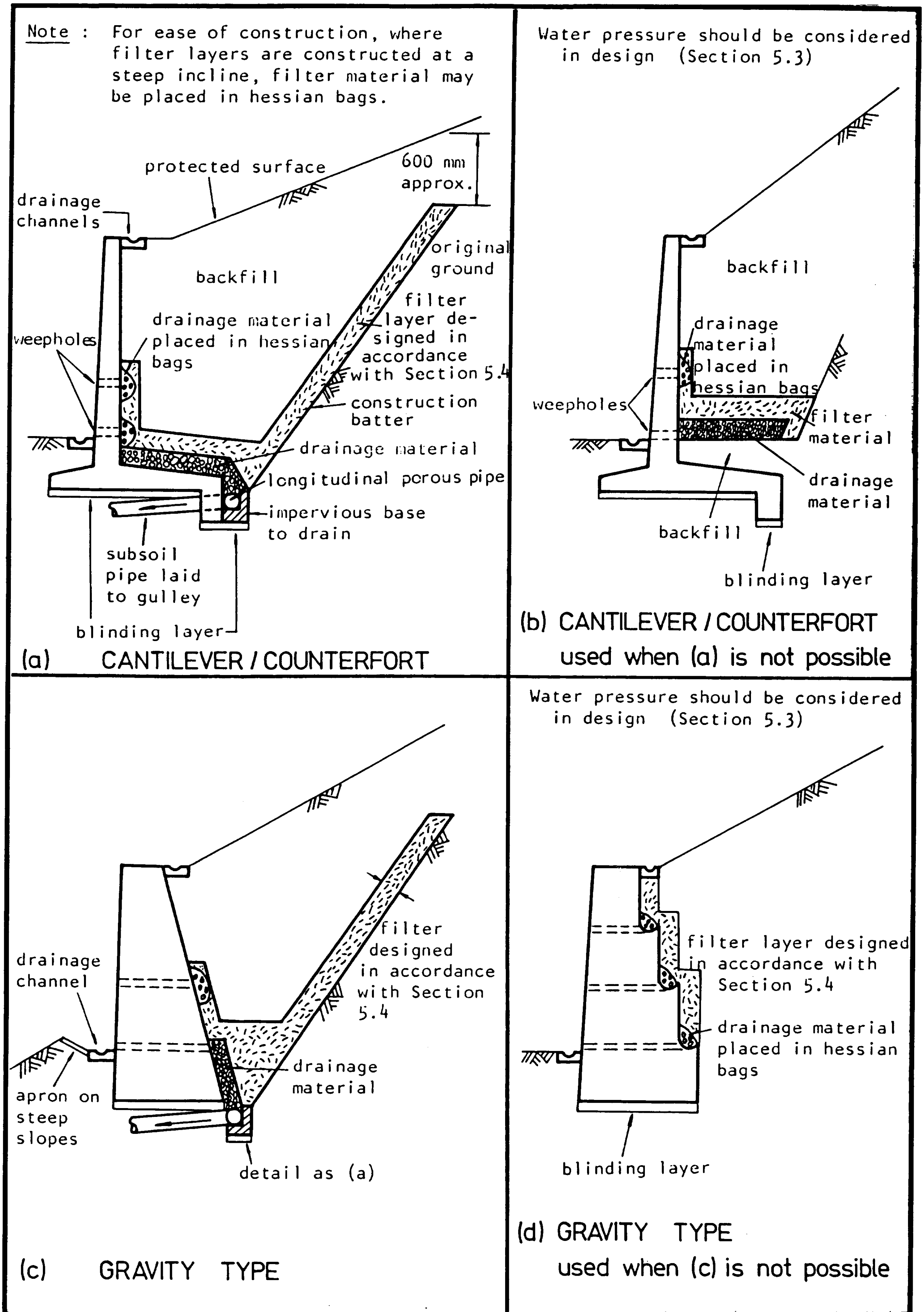
Note water pressure is zero on potential failure plane.

(c) SURFACE INFILTRATION (INCLINED DRAIN)

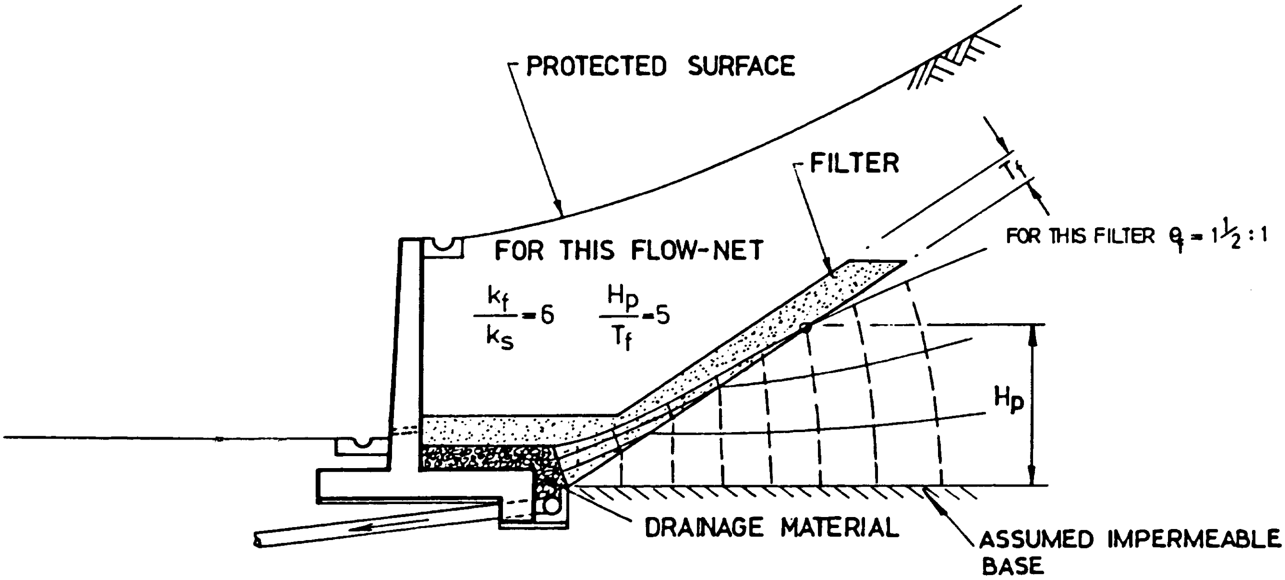
(FLOW NETS ASSUME HOMOGENEOUS, ISOTROPIC SOIL)

EFFECT OF SURFACE INFILTRATION AND DRAIN LOCATION ON WATER PRESSURES

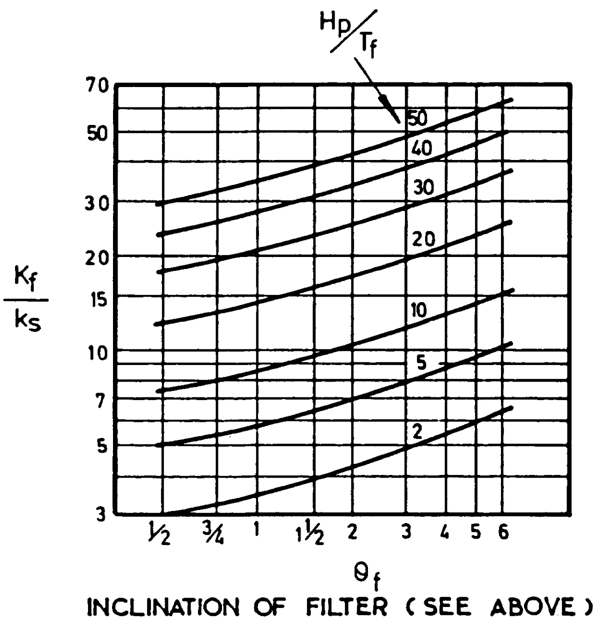
FIGURE 17



DRAINAGE DETAILS FOR RETAINING WALLS | **FIGURE 18**



(a) TYPICAL FLOW NET FOR SEEPAGE INTO INCLINED FILTER



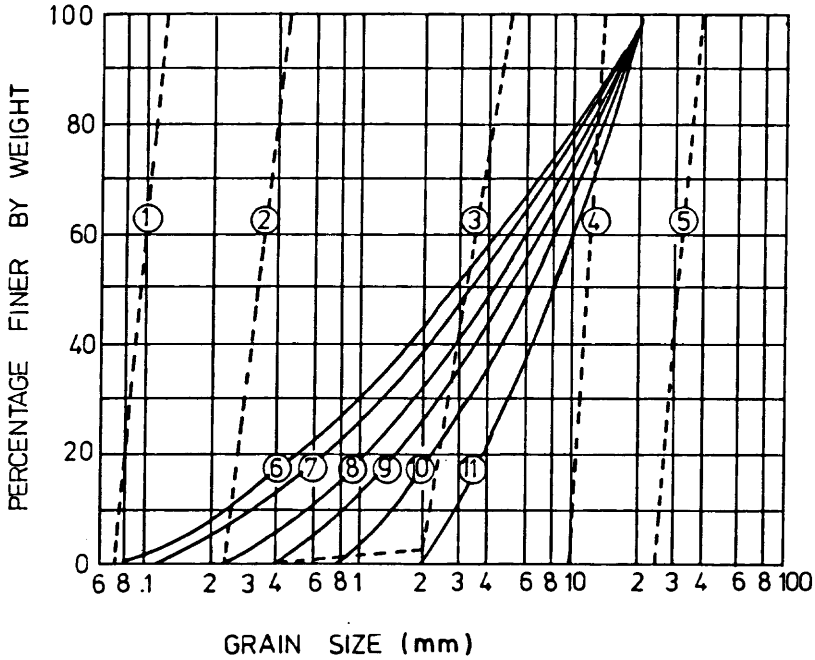
K_f = PERMEABILITY OF FILTER
K_s = PERMEABILITY OF SOIL

(b) CHART DEVELOPED FROM FAMILY OF FLOW-NETS

(after Cedergren, 1977)

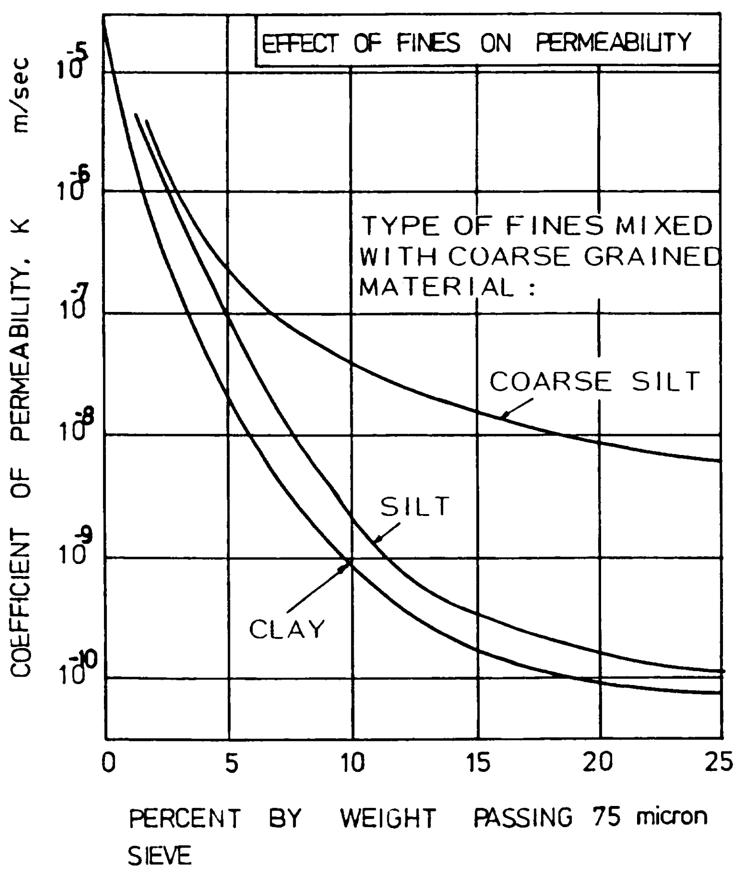
DESIGN OF INCLINED DRAINS

FIGURE 19



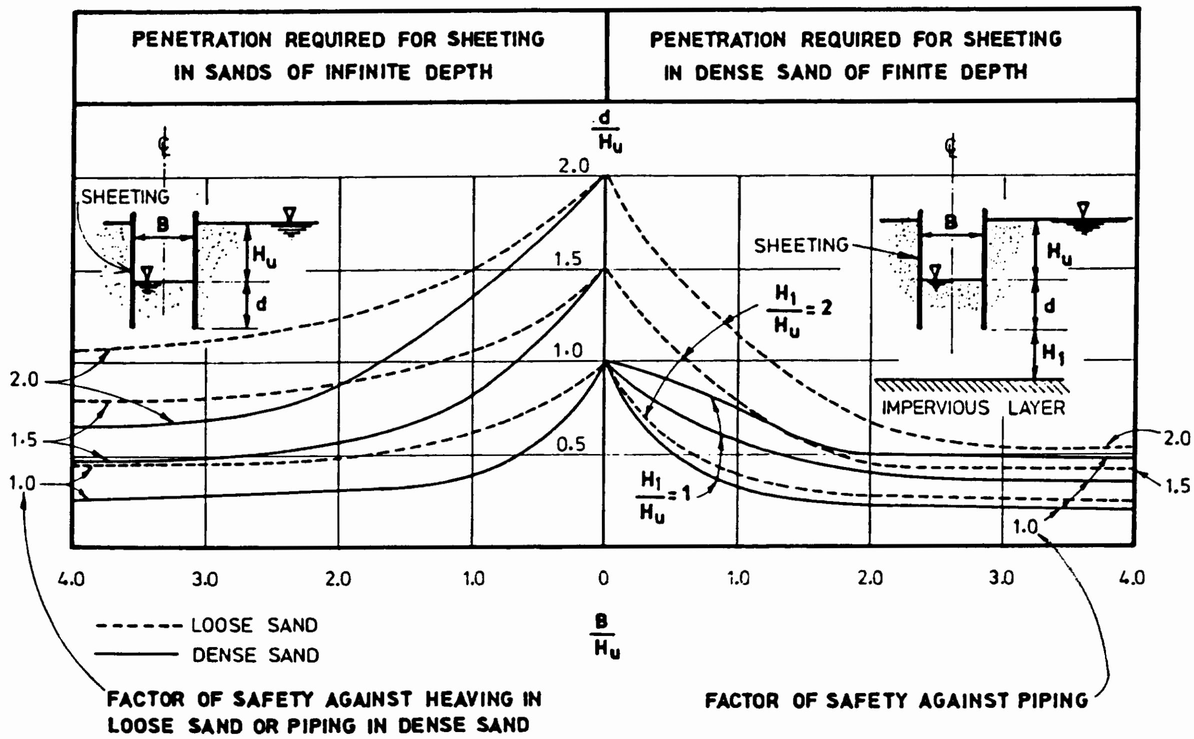
COEFFICIENT OF PERMEABILITY FOR CLEAN COARSE-GRAINED DRAINAGE MATERIAL

CURVE	m/sec
1	0.5×10^{-4}
2	6.6×10^{-4}
3	2.7×10^{-2}
4	2.9×10^{-1}
5	3.7×10^{-1}
6	0.5×10^{-4}
7	4.1×10^{-4}
8	1.1×10^{-3}
9	3.6×10^{-3}
10	9.2×10^{-3}
11	1.1×10^{-2}

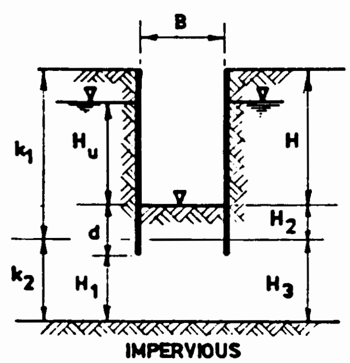


(after NAVFAC DM-7, 1971)

PERMEABILITY OF DRAINAGE MATERIALS **FIGURE 20**



a) SHEET PENETRATION IN GRANULAR SOILS



Let $k_1 < k_2$

If $H_1 < H_3$ there generally is more flow than given in graph (a)(infinite) above.

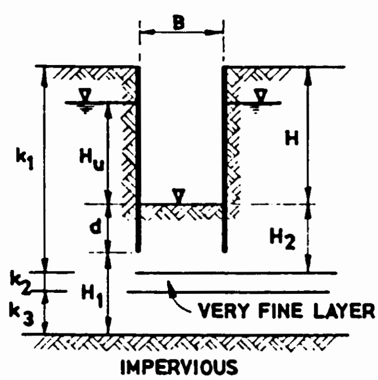
If $(H_1 - H_3) > B$ use graph (a)(infinite).

If $(H_1 - H_3) < B$ there is more flow than given in graph (a)(infinite). If $k_2 > 10K_1$ failure head H_u is equal to H_2.

Let $k_1 > k_2$

If $H_1 < H_3$ safety factors are intermediate between those for graph (a)(finite).

If $H_1 > H_3$ graph (a)(finite) is conservative.



Let $k_1 = k_3$ and $k_1 \gg k_2$

If $(H_2 - d) > B$ use graph (a)(finite) above.

If $(H_2 - d) < B$ pressure relief required so that unbalanced head on fine layer does not exceed weight of H_2.

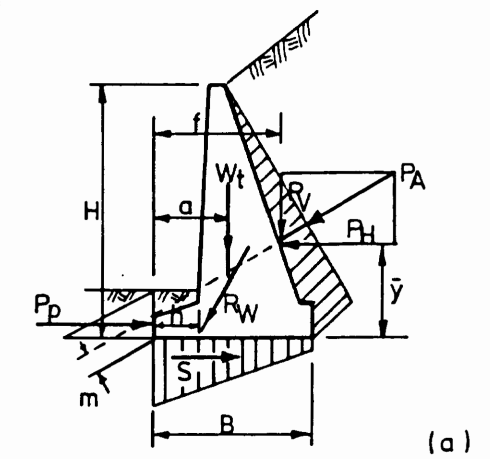
If fine layer is higher than bottom of excavation the completed excavation is safe, but during construction a blow in may occur - pressure relief then required.

b) PILING PENETRATION TO PREVENT PIPING (after NAVFAC DM-7, 1971)

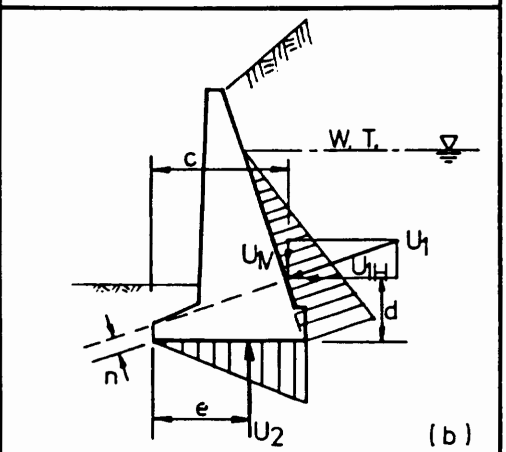
STABILITY AGAINST PIPING IN COHESIONLESS SOILS | FIGURE 21

WALL TYPE	LOAD DIAGRAM	STABILITY CRITERIA

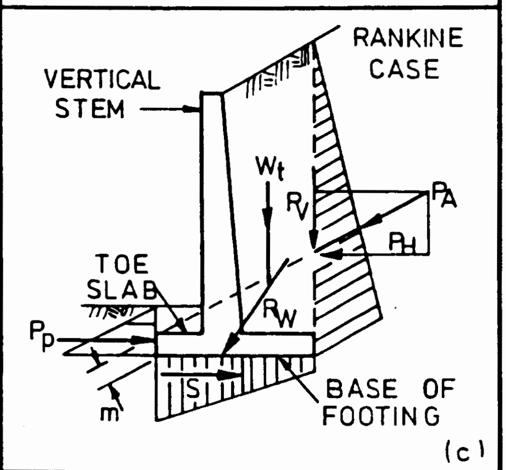
GRAVITY



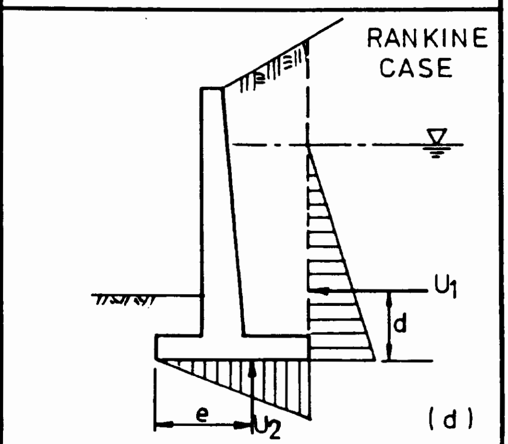
GRAVITY (water forces)



CANTILEVER



CANTILEVER (water forces)



STABILITY CRITERIA

SLIDING

$S = (W_t + P_v + U_{1V} - U_2)\tan\delta_b + c_bB$

$$Fs\ (\text{sliding}) = \frac{S + 0.5P_p}{P_H + U_{1H}} \geq 1.5$$

i.e. F.S. on any included ultimate passive ≥ 3.0

OVERTURNING

Moments about the toe of the base

$$Fs\ (\text{overturning}) = \frac{\text{Moments resisting overturning}}{\text{Moments causing overturning}} = \frac{Mr}{Mo}$$

$Mr = W_t a$ (Passive Resistance P_p ignored)

$Mo = P_A m + U_1 n + U_2 e$

$$Fs = \frac{W_t a}{(P_H\bar{y} - P_v f) + (U_{1H} d - U_{1V} c) + U_2 e} \geq 2.0$$

N.B. It is illogical to take vertical components of the disturbing forces and use them as restoring moments in the expression for F.S. See section 6.3.2

Overturning may be ignored if R_w lies within middle third (soil), middle half (rock). For gravity type walls, overturning must be checked at selected horizontal planes, the resultant must remain within the middle third.

LOCATION OF RESULTANT

Point where R_w intersects base, from toe,

$$h = \frac{W_t a + P_v f - P_H\bar{y} + U_{1V} c - U_{1H} d - U_2 e}{W_t + P_v + U_{1V} - U_2}$$

(Passive resistance Pp ignored)

For soil foundation material, R_w should lie within middle third of the base

For a rock foundation, R_w should lie within middle half of the base

BEARING PRESSURE

See section 6.4 for calculation of factor of safety for bearing Fs (bearing) ≥ 3.0

W_t = total weight of the wall including soil on toe plus soil above heel (for cantilever walls only)

R_w = resultant of W_t, P_A, U_1 & U_2

SLOPE FAILURE IN SURROUNDING SOIL

With shear surfaces passing under the wall, the factors of safety should comply with the requirements of Table 5.2 of the Geotechnical Manual for Slopes.

WATER FORCES

Reference should be made to Chapter 5 for cases other than those shown here.

STABILITY CRITERIA FOR RETAINING WALLS — **FIGURE 22**

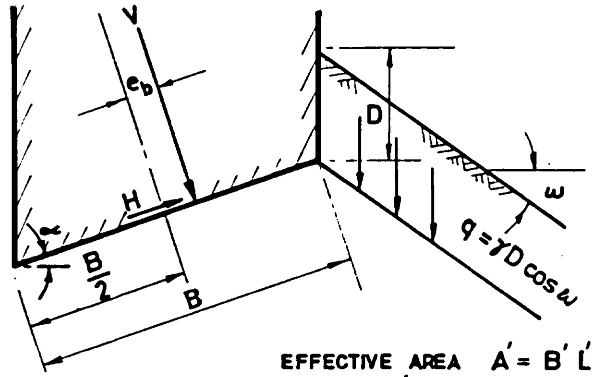
$$q_{ult} = cN_c S_c i_c t_c g_c + \tfrac{1}{2}\gamma B N_\gamma S_\gamma i_\gamma t_\gamma g_\gamma + q N_q S_q i_q t_q g_q$$

SHAPE FACTORS

$$S_c = 1 + \frac{B}{L}\,\frac{N_q}{N_c}$$

$$S_\gamma = 1 - 0.4\frac{B}{L}$$

$$S_q = 1 + \frac{B}{L}\tan\phi'$$



EFFECTIVE AREA $A' = B'L'$

WHERE $B' = B - 2e_b$

$L' = L - 2e_l$

(SECTION 6.4.3)

INCLINATION FACTORS

$$i_c = i_q - \frac{1 - i_q}{N_c \tan\phi'}$$

$$i_\gamma = \left[1 - \frac{H}{V + B'L'\,c\cot\phi'}\right]^{m+1}$$

$$i_q = i_\gamma^{\frac{m}{m+1}}$$

WHERE $m = \dfrac{2 + \frac{B}{L}}{1 + \frac{B}{L}}$ PROVIDED THE INCLINATION OF LOAD IS IN THE DIRECTION OF B

NOTE : $H_{max} = V\tan\phi' + A\,c$

TILT FACTORS

$$t_c = t_q - \frac{1 - t_q}{N_c \tan\phi'}$$

$$t_\gamma = t_q = [1 - \alpha\tan\phi']^2$$

WHERE α IS IN RADIANS

GROUND SLOPE FACTORS

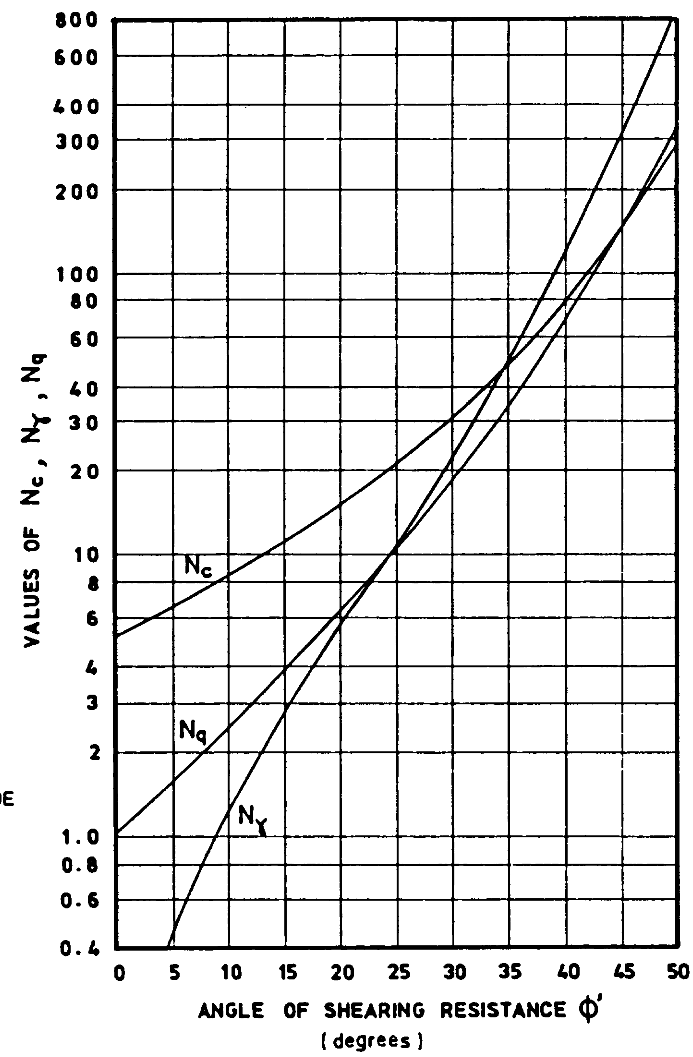
$$g_c = g_q - \frac{1 - g_q}{N_c \tan\phi'}$$

$$g_\gamma = g_q = [1 - \tan\omega]^2$$

WHERE $\alpha < 45°$, $\omega < 45°$ AND $\omega < \phi$

q = SURCHARGE EFFECT

$= \gamma D \cos\omega$

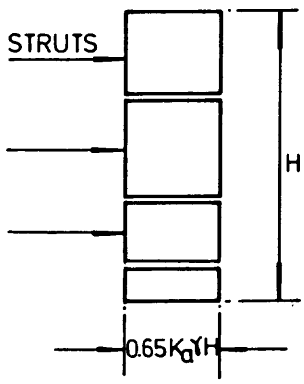


BEARING CAPACITY FACTORS

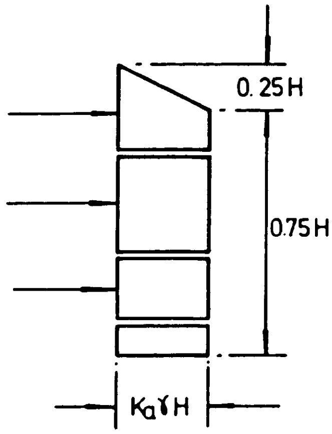
NOTES

1. DATA APPLIES TO SHALLOW FOUNDATIONS ONLY $D < B$.
2. FOR $\omega \geq \frac{\phi'}{2}$ A CHECK SHOULD ALSO BE MADE FOR OVERALL SLOPE STABILITY.
3. FOR THE EFFECTS OF NONHOMOGENEOUS SOIL AND SOIL COMPRESSIBILITY AND SCALE EFFECTS REFERENCE SHOULD BE TO VESIC.
4. WHERE THE FOUNDATION IS SET BACK FROM THE CREST OF THE SLOPE, REFER TO SECTION 6.6

BEARING CAPACITY DATA (VESIC, 1975)	FIGURE 23



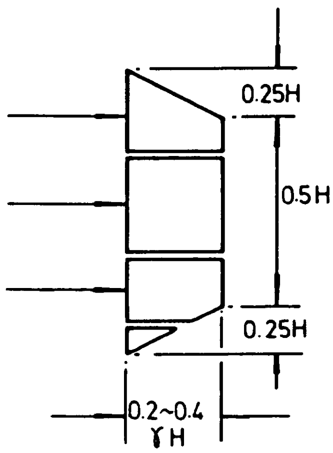
SANDS



N.C. CLAYS

WHERE

$$K_a = 1 - \frac{4C}{\gamma H}$$

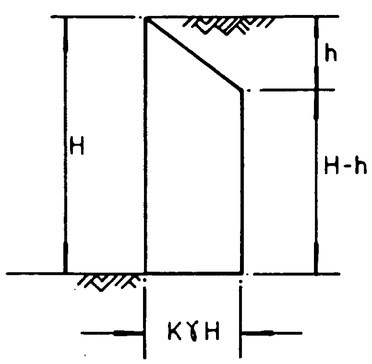


O.C. CLAYS

K_a = RANKINE COEFFICIENT OF ACTIVE EARTH PRESSURE

NOTE: WATER AND SURCHARGE LOADINGS SHOULD BE CONSIDERED.

(a) after Peck, 1969



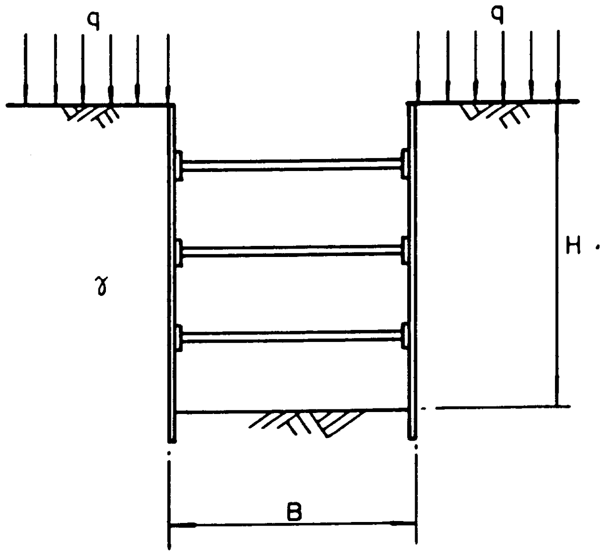
	h	K
SAND	0.4 H	0.2~0.3
HARD CLAY (N > 4)	0.4 H	0.2~0.4
SOFT CLAY (N ≤ 4)	0.4 H	0.4~0.5

K = COEFFICIENT OF EARTH PRESSURE
N = STANDARD PENETRATION TEST VALUE

(b) after Japan Society of Civil Engineers, 1977

PRESSURE ENVELOPES FOR INTERNALLY BRACED EXCAVATION

FIGURE 24

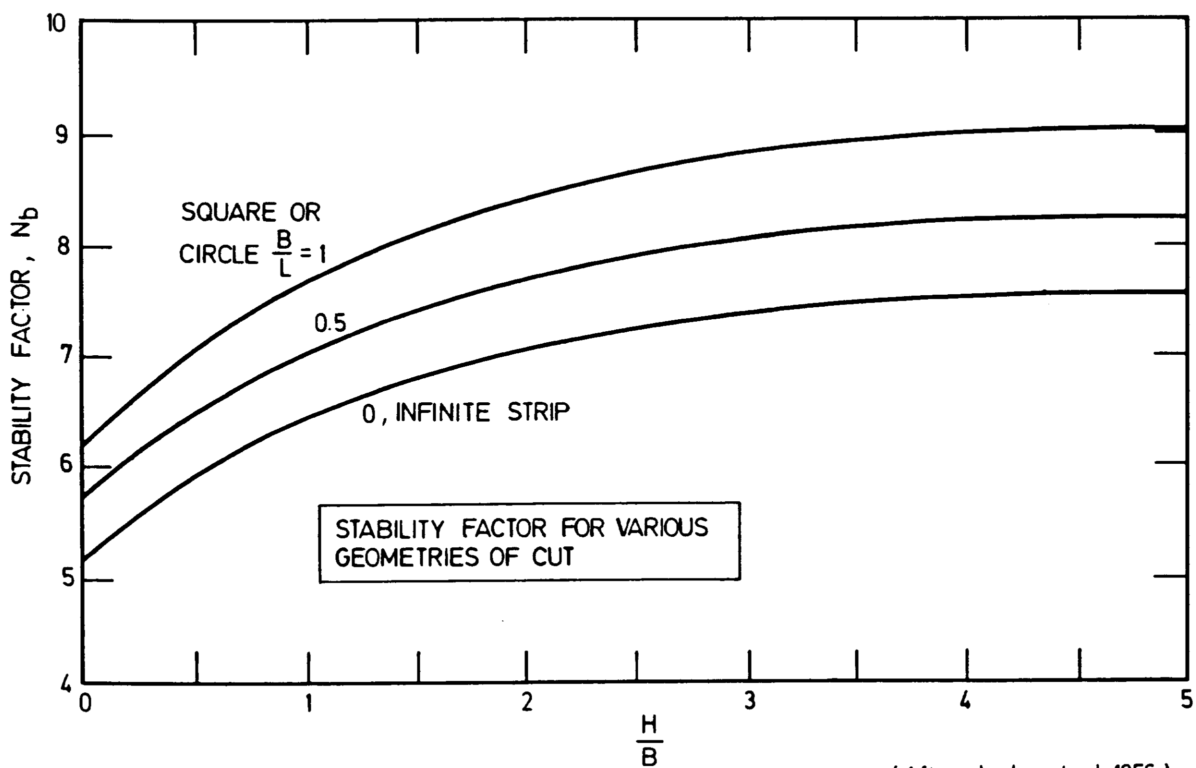


$$F_s(\text{base}) = \frac{N_b c}{\gamma H + q}$$

c = AVERAGE UNDRAINED SHEAR STRENGTH OF THE SOIL FROM BASE LEVEL TO A DEPTH OF 0.25H BELOW THE BASE

N_b = STABILITY FACTOR

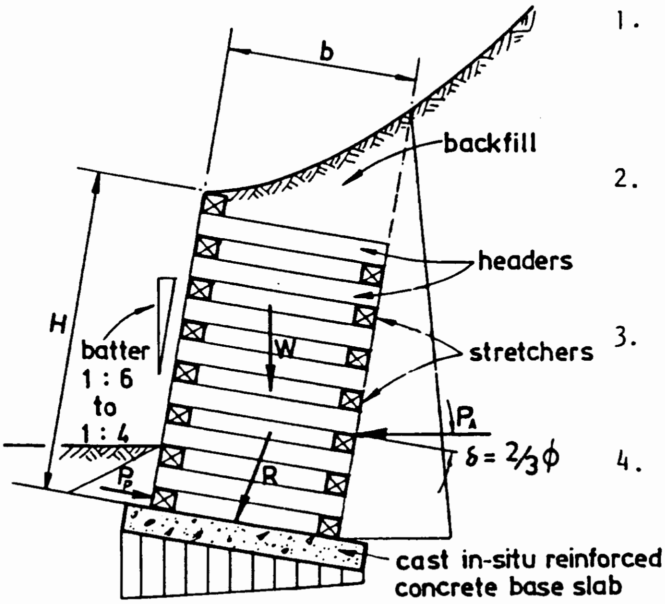
L = EXCAVATION LENGTH



(After Janbu et al, 1956)

FACTOR OF SAFETY WITH RESPECT TO BASE HEAVE

FIGURE 25



(a) TYPICAL SECTION
(diagrammatic)

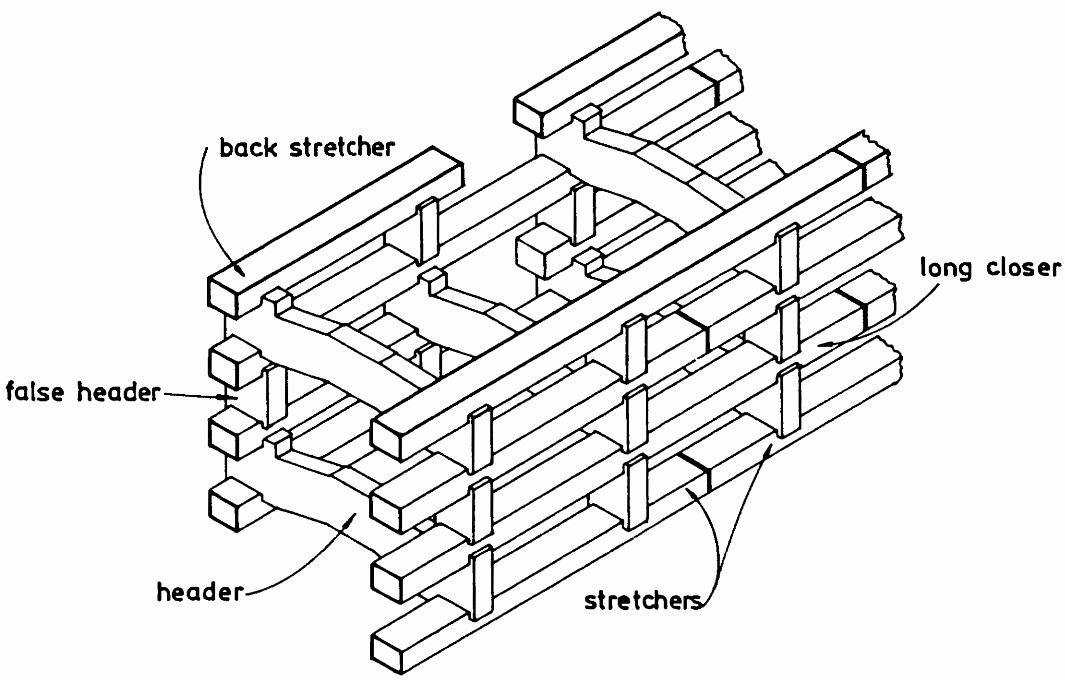
NOTES

1. Crib wall units to be infilled with free draining material, well compacted in layers. Care should be taken to avoid disturbing the units.

2. Design criteria for gravity walls apply to crib walls. Wall section resisting overturning is taken as a rectangle of dimension (H x b).

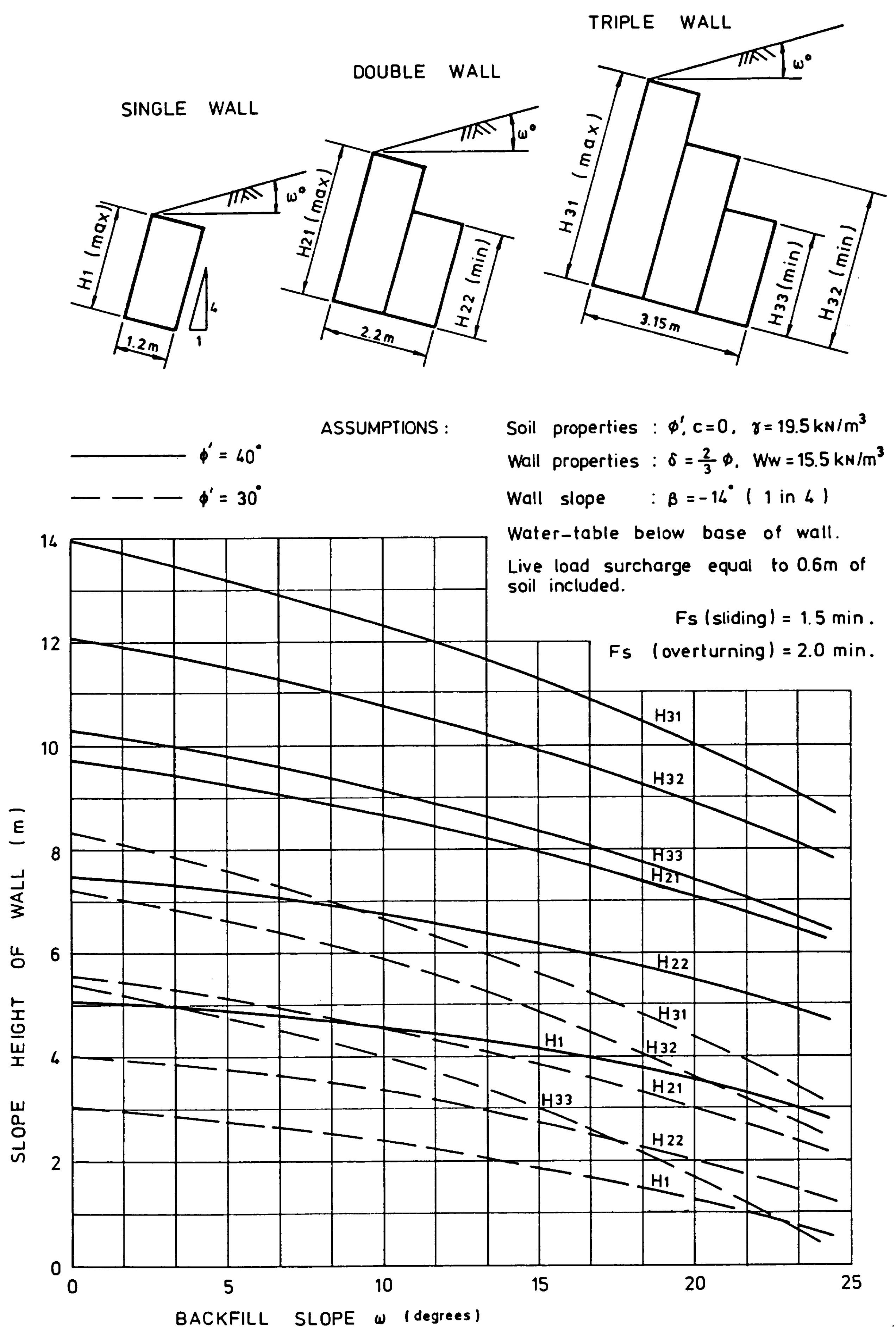
3. Low walls (under 1.5m high) may be made with a plumb face. Higher walls should be battered as shown.

4. For high walls (4m high and over) the batter is increased or supplementary cribs are added at the back.

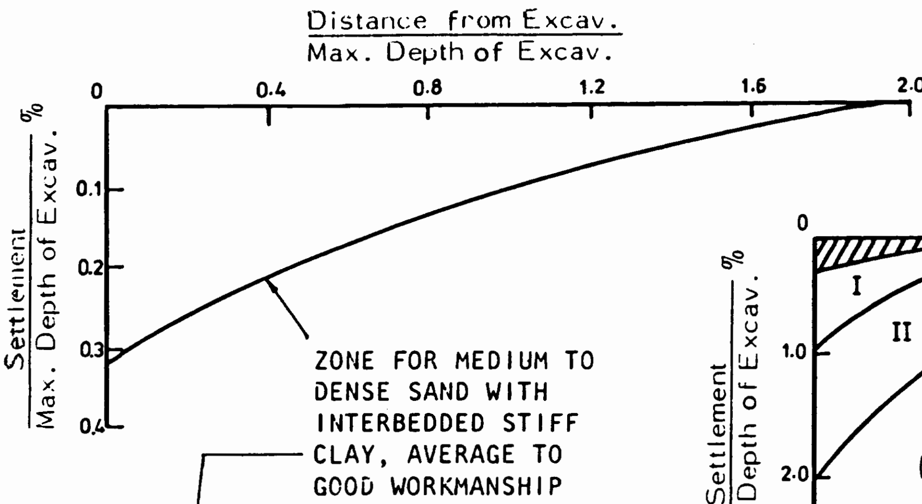


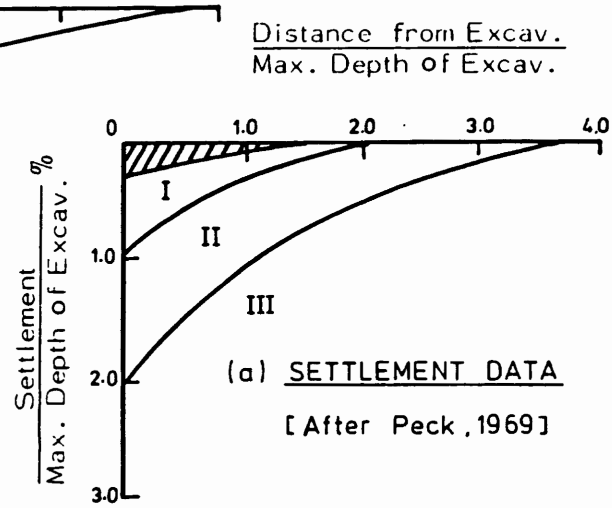
(b) TYPICAL FORM OF CRIB WALLING

CRIB WALL DETAILS	FIGURE 26



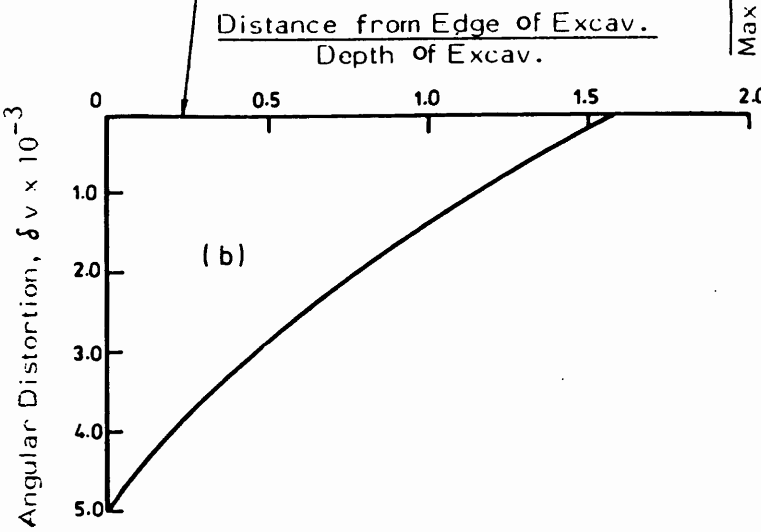
CRIBWALL DESIGN CURVES	**FIGURE 27**





(a) SETTLEMENT DATA

[After Peck, 1969]



(b) SETTLEMENT DATA

[after O' Rourke et al, 1976]

Note

May be used for approximate guidance only for residual soils.

Zone I

Sand and soft to hard clay average workmanship

Zone II

a) Very soft to soft clay

1) Limited depth of clay below bottom of excavation.

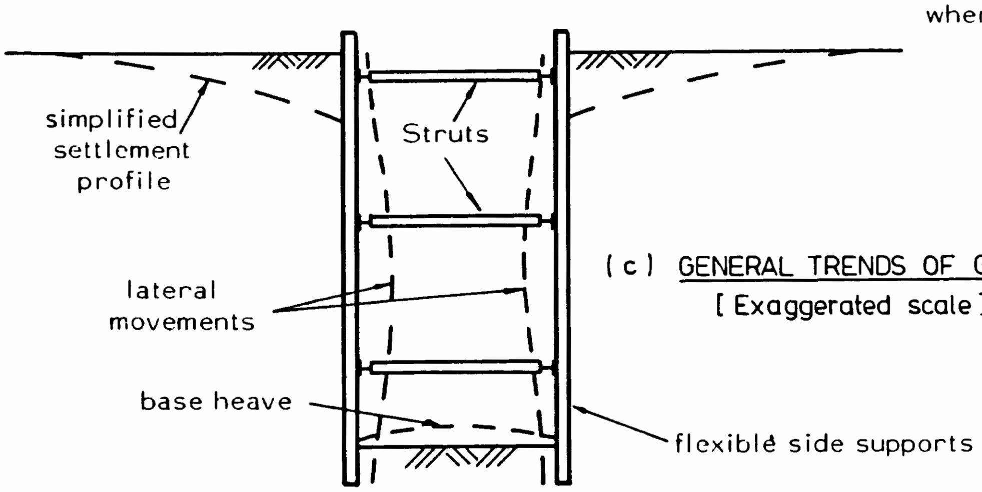
2) Significant depth of clay below bottom of excavation but $N_b < 5.14$.

b) Settlements affected by construction difficulties.

Zone III

Very soft to soft clay to a significant depth below bottom of excavation and with $N_b > 5.14$,

where $N_b = \frac{\gamma H}{c_b}$



(c) GENERAL TRENDS OF GROUND MOVEMENTS

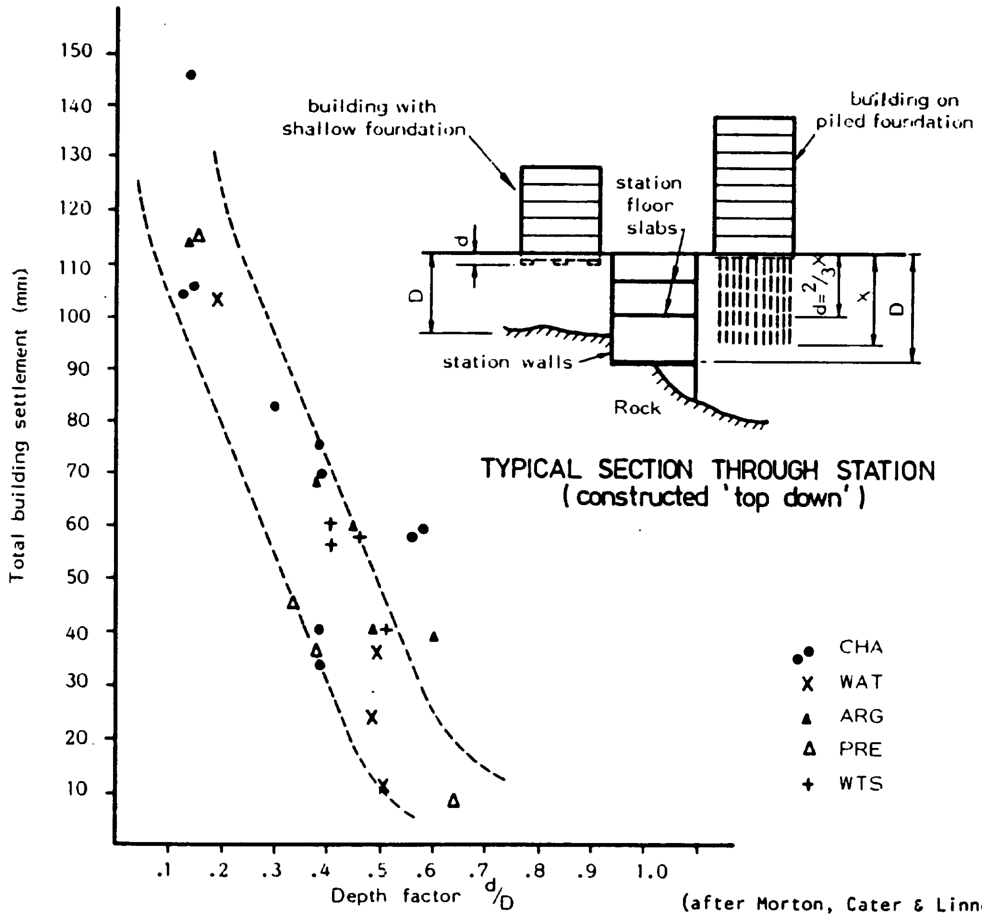
[Exaggerated scale]

LARGE EXCAVATIONS — SETTLEMENT GUIDE

FIGURE 28

Station	Building Notation	Effective Foundation Depth (d) m	Depth Factor d/D	Settlement During Wall Installation mm	Settlement Due To Dewatering mm	Settlement During Excavation mm	Total Settlement mm	Lateral Wall Movement mm
Chater (CHA)	Prewar I	4	0.16	63	46	37	146	58
	Prewar II	4	0.16	36	32	37	105	-
	1960 I	8	0.31	46	20	16	82	-
	1960 II	10	0.39	28	6	6	40	-
	1960 III	10	0.39	35	14	20	69	-
	1960 IV	10	0.39	15	9	9	33	-
	1960 V	10	0.39	40	22	13	75	-
	1970 I	15	0.59	29	15	15	59	-
Waterloo (WAT)	1960 I	5	0.20	30	40	33	103	-
	1960 II	13	0.52	0	6	5	11	-
	1960 III	3	0.50*	0	15	21	36	-
	1970 I	12	0.48	6	4	13	23	-
Argyle (ARG)	1950 I	4	0.15	-	108	5	113	-
	1950 II	12	0.46	-	56	4	60	43
	1950 III	13	0.50	-	36	4	40	-
	1950 IV	10	0.38	-	50	18+	68	31
	1960 I	16	0.61	-	35	4	39	32
	1970 I	17	0.65	-	4	4	8	22
Prince Edward (PRE)	Prewar I	4	0.17	-	115		115	10
	1960 I	8	0.34	-	42	3	45	9
	1960 II	9	0.38	-	31	6	37	-
Wong Tai Sin (WTS)	1950 I	9	0.47	14	42	2	58	18/28
	1950 II	8	0.42	5	39	12	56	-
	1950 III	8	0.42	10	35	15	60	-
	1950 IV	10	0.52	1	39	0	40	-

* High rock at this location D = 6 metres
\+ Also influenced by adjacent sheet pile excavation

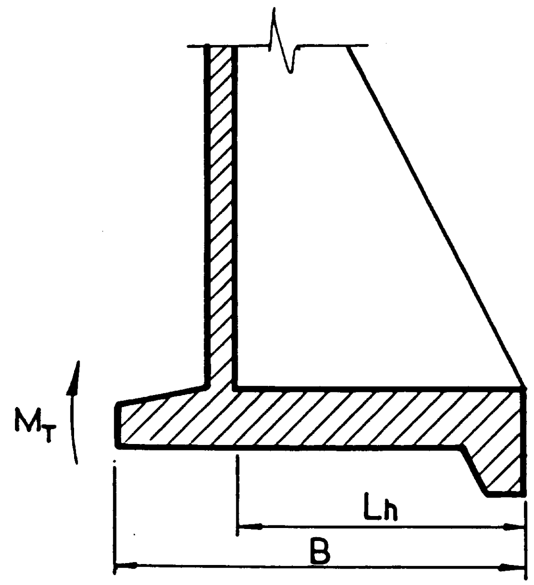


TYPICAL SECTION THROUGH STATION (constructed 'top down')

(after Morton, Cater & Linney, 1980)

RELATIONSHIP BETWEEN TOTAL BUILDING SETTLEMENT AND DEPTH FACTOR

BUILDING SETTLEMENT DATA | **MTR CONSTRUCTION HONG KONG** | **FIGURE 29**

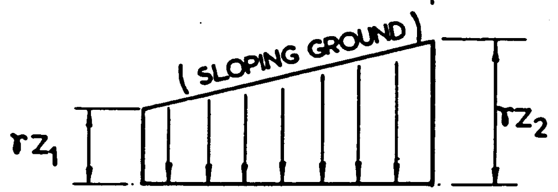


TOE MOMENT EFFECT ON HEEL

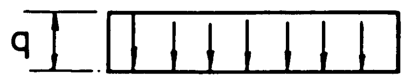
The toe support moment produces a loading on the heel. If it is assumed that no moment is transmitted into the stem, an equivalent parabolic heel loading is as shown below, with the maximum ordinate given by

$$P_t = 2.4 \frac{M_T}{L_h^2}$$

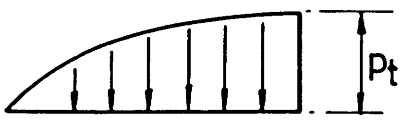
where M_T is the toe support moment.



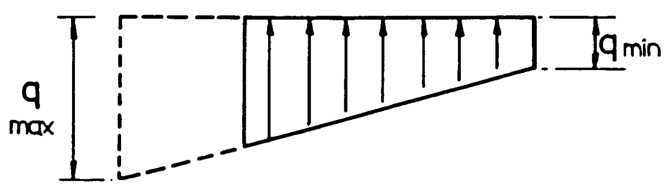
WEIGHT OF BACKFILL ABOVE HEEL



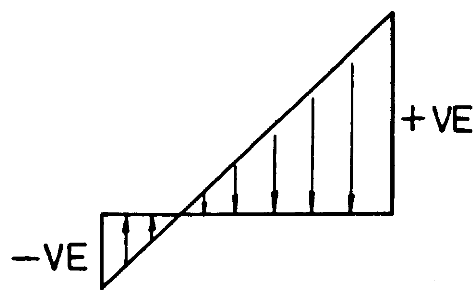
SELF WEIGHT OF HEEL



LOADING FROM TOE MOMENT



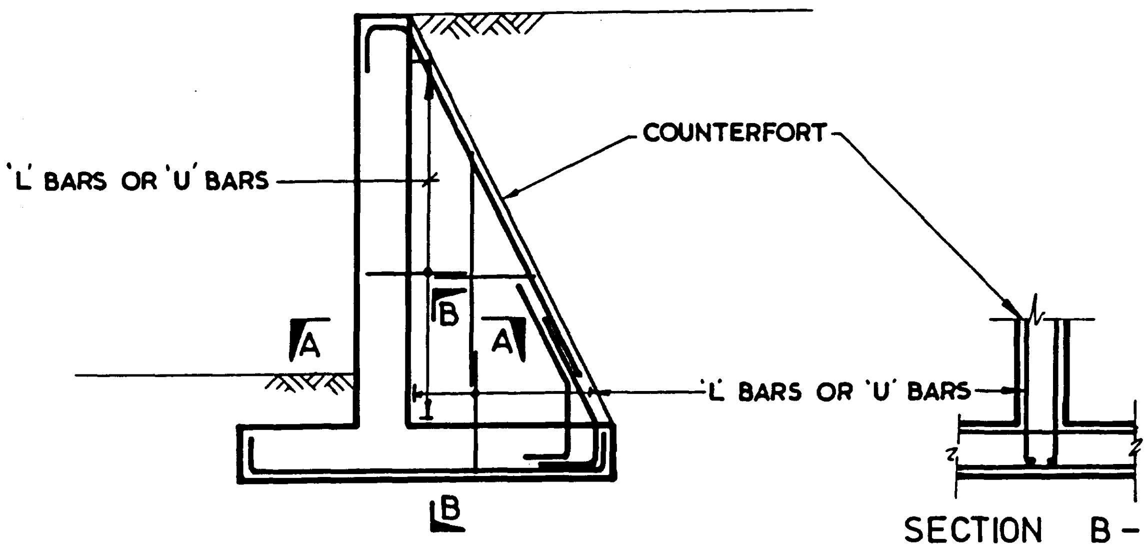
ASSUMED FOUNDATION BEARING PRESSURES



RESULTANT LOADING ON HEEL (MAY BE FULLY POSITIVE)

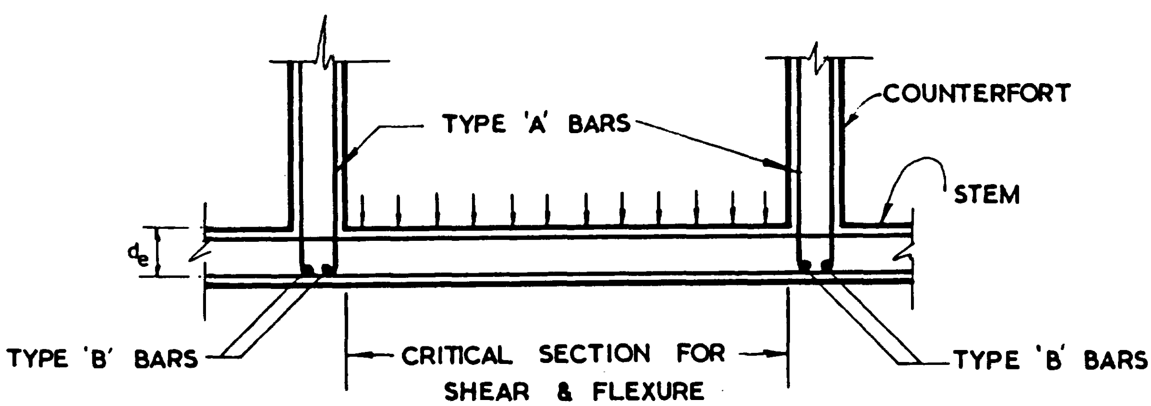
NOTE : PRESSURE DIAGRAMS NOT TO SCALE

DESIGN LOADING ON HEEL SLAB — **FIGURE 30**



SECTION B-B

COUNTERFORT WALL
(DIAGRAMMATIC)



SECTION A-A

NOTES

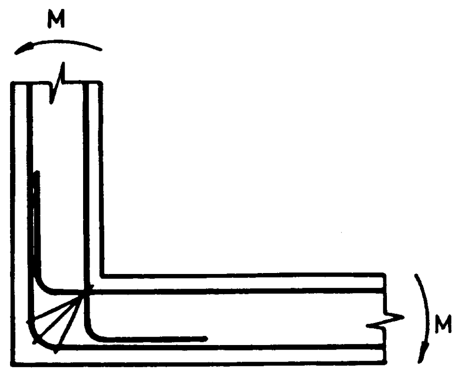
1. Type A bars, or 'hangers', must be designed to take the full reaction from the wall spanning between the counterforts.

2. Type B bars provide additional mechanical anchorage for the hanger bars.

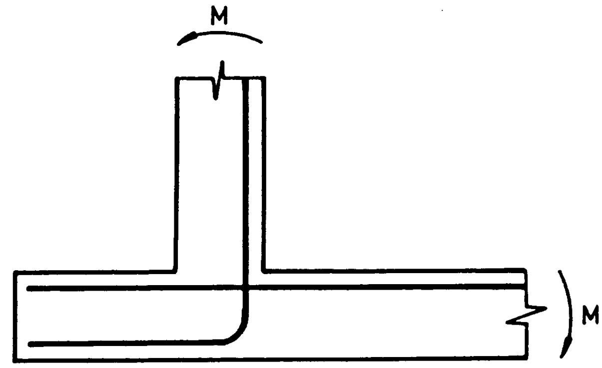
3. Note that the critical section for shear is at the face of the counterfort not at a distance d_e equal to the effective depth from the face.

4. For clarity, only limited steel is shown on the sketches.

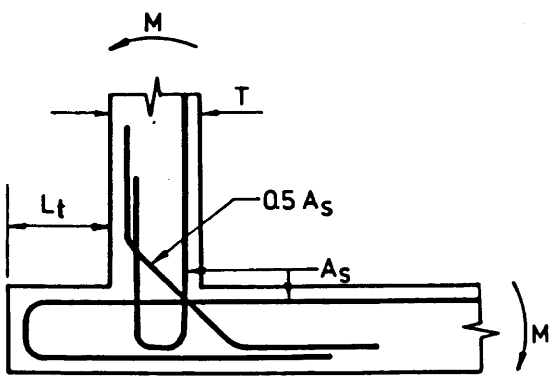
COUNTERFORT WALLS — DETAILING AT JUNCTION OF COUNTERFORT WITH HEEL & STEM	FIGURE 31



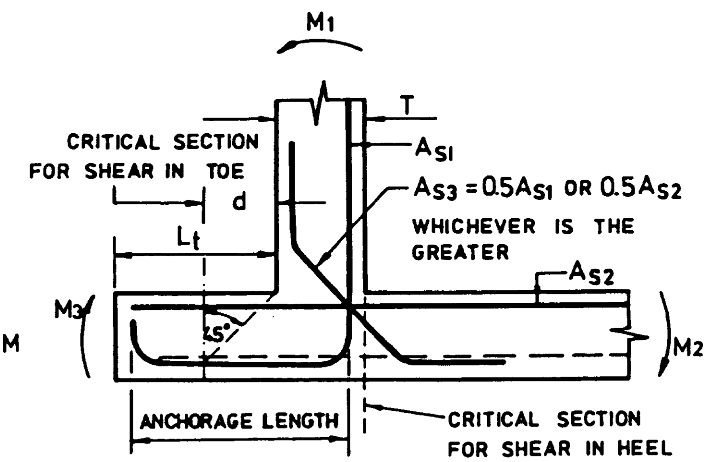
a) UNSATISFACTORY DETAIL



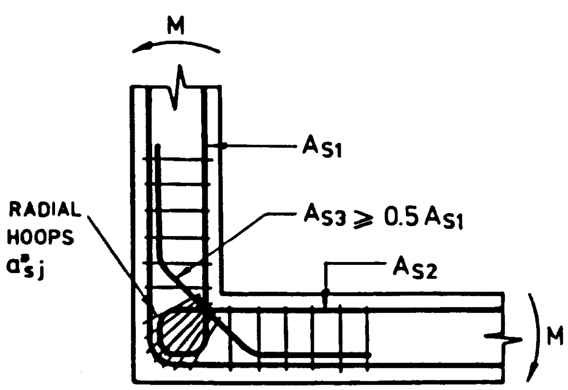
b) UNSATISFACTORY DETAIL



c) RECOMMENDED DETAIL FOR $L_t < T$



d) RECOMMENDED DETAIL FOR $L_t > T$ & L_t GREAT ENOUGH TO PROVIDE ANCHORAGE LENGTH



e) RECOMMENDED DETAIL FOR LARGE JOINTS ($A_{S1} > 0.5\%$)

NOTES

1. Refer to Sections 11.8 & 11.9 for discussion, including limitations on steel percentage.

2. For clarity, not all steel is shown in these sketches. Additional steel for toe moment M3 is shown dotted. No shrinkage, temperature or distribution steel is shown.

3. If desired, a fillet may be included.

DETAILING OF CANTILEVER WALL REINFORCEMENT | **FIGURE 32**