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DESIGN OF EMBEDDED RETAINING WALLS IN STIFF CLAYS

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ABSTRACT

Increasing use is being made of embedded retaining walls for the construction of roads below ground level in urban areas. This report considers the design requirements of these types of structure in heavily overconsolidated sedimentary clays on which many urban areas in England are founded. The Report reviews methods of determining wall stability and of assessing the structural loading in service. Results from simple limit equilibrium calculations are compared with predictions from numerical analyses. Attention is drawn to the importance of wall flexibility, the initial in situ stress state in the ground and the effects of excavation on the equilibrium pressure distributions likely to act on embedded walls in stiff clays in the longer term.

1. INTRODUCTION

In urban areas of the United Kingdom the restricted space available, combined with environmental considerations, means that, increasingly, roads have to be located beneath existing ground level. Many of the road improvement schemes currently underway or planned in urban areas therefore involve significant lengths of retained cutting or cut-and-cover tunnel. The cost of the retaining walls on such schemes is typically about 20-30 per cent of the total.

Embedded retaining walls, in which the wall is installed first and then excavation carried out to formation level on one side, are frequently used for below ground construction. This is because with this type of structure less land take is required and reduced disturbance occurs to the natural ground than would be the case with construction of a conventional cantilever or gravity wall in open excavation. Embedded walls can be free standing and act as cantilevers, or supported by anchoring back into the retained ground or by provision of structural props. For permanent works, bored pile or slurry trench (diaphragm) techniques are usually used to form walls of reinforced concrete. These types of construction have only been introduced into roadworks on a significant scale over the past 10 to 15 years. For cut-and-cover tunnels the roofing slab usually acts as the prop while, for retained cuttings, support can be provided by a structural slab located below excavation level and spanning between the walls on each side. This latter form of construction has the advantage that a smaller depth of embedment is generally required to prevent overturning of the wall. However, this advantage has to be offset against the higher prop forces and the need to install temporary and permanent propping at an early stage to prevent excessive movements during construction.

Many urban areas in the United Kingdom are founded on heavily overconsolidated clays which present particular problems in the design of earthworks and earth retaining structures. Among the features of these deposits which give rise to uncertainties in design are the presence of high in-situ lateral stresses, which render them susceptible to swelling and softening following excavation. In addition these clays often contain discontinuities within their mass which makes difficult the determination of representative strength and stiffness properties for use in design. Finally, they are normally of relatively low permeability so that it may take many years or decades for them to reach equilibrium under the new stress regime caused by construction.

The purpose of this Report is to outline and comment on the various approaches currently available for the design of these types of structure.

2. DESIGN REQUIREMENTS

CIRIA Report 104 (1984) gives guidance for the design of embedded walls in overconsolidated clay which rely in whole or in part on the passive resistance of the ground below excavation level for their support. It is therefore applicable for cantilever and singly propped walls. For these types of structure, failure of the soil will result in overturning of the wall about a point of support. The determination of the depth of embedment necessary to prevent overturning therefore constitutes a key design requirement. The usual approach adopted is to use a limit equilibrium calculation in which conditions at collapse are postulated and a 'lumped' factor of safety applied to ensure that such a failure will not take place.

A second requirement is that the wall and any support system are structurally capable of withstanding the most adverse loading which will develop during construction and in service. A simple hand calculation approach is given in CIRIA Report 104 for calculating the maximum bending moments, shear forces and prop loads based on the assumption of limiting active and passive earth pressures acting over a reduced depth of embedment. The loading effects are then multiplied by enhancement factors for design of the structural elements. However simple calculations of this type take no account of the relative stiffness of the soil and the structure, or of the initial in situ stress state in the ground. Increasing use is therefore being made of numerical analyses which cater for these effects and enable the proposed construction sequence to be simulated. For structural design purposes adequate information can often be obtained from the use of computational methods which incorporate relatively simple soil continuum models (Wood 1981, Pappin et al 1986).

Assessments are also needed of the deformation of the structure and of the local and global ground movements which will occur both during construction and in service. The magnitude and distribution of ground movement is particularly relevant for schemes in urban areas where other structures and buried services are located in close proximity. Methods of assessing ground movements can range from the use of simplified empirical correlations based on previous observations (Peck 1969, Clough and O'Rourke 1990) to the adoption of finite element methods incorporating complex non-linear soil models (Higgins, Potts and Symons 1989); the level of sophistication required depends upon the potential consequences for adjacent structures. For preliminary design purposes Bolton, Powrie and Symons (1990) have suggested an approach for assessing the movement of relatively rigid walls using idealised strain fields linked to relevant stress-strain relations for the soil.

3. LIMIT STATE APPROACH

There is an increasing trend to formulate design guidance in geotechnical codes of practice and standards in terms of the requirement to consider specific limit states. The advantage of this is that it introduces a more logical approach to the design process, simplifies checking and provides greater compatibility with existing structural codes, such as BS 5400 (1988).

From the preceding section the four principal limit states that need to be considered in the design of an embedded retaining wall are:-

(i) Ultimate limit states of the soil - involving shear failure and accompanying large strain in the ground within which the structure is located.

(ii) Serviceability limit state of the soil - representing the limit on soil deformation which will not adversely affect the performance in service of the retaining wall nor induce an ultimate or serviceability failure in any adjacent structures.

(iii) Ultimate limit state of the structure - involving structural failure of the retaining wall or of any component part of the structure in direct load, bending or shear.

(iv) Serviceability limit state of the structure - comprising limits on the deflection and movement of the structure in service for visual acceptability, limitation of structural cracking, and which will not adversely affect the performance of associated components or adjacent structures and services.

The success or otherwise of any design is likely to depend more on limitations in the designer's ability to envisage all relevant loading incidences and potential failure mechanisms, than on inadequacies of the calculation method or of any associated partial or global factors of safety. A wide perspective of the structure and its

environment is therefore necessary in considering each limit state. Finally, it should be recognised that the lateral soil loading acting on a retaining wall in service may well be greater than at a condition of ultimate failure; this is a result of the strain dependency of soil strength, and contrasts with gravitational loading on non-geotechnical structures.

4. OVERCONSOLIDATED CLAYS

Many urban areas in the United Kingdom are founded on stiff clays. Such deposits include sedimentary clays of high plasticity, such as the London, Oxford, Lias, Weald and Gault Clays, found predominantly in southern England as well as the more widely distributed glacial clays of generally lower plasticity. All these clays are overconsolidated so that the in-situ lateral effective stress in the ground is likely to exceed the vertical effective stress giving coefficients of earth pressure at rest, K_o, of greater than unity. Many are also highly fissured, which hampers the determination of strength and stiffness properties for use in design. Most high plasticity sedimentary clays have a relatively low mass permeability so that the swelling and softening effects, consequent on excavation, can continue for long periods after construction.

The problems of long term behaviour of retaining walls and slopes in overconsolidated clays are well illustrated by case records of failures which have taken place many years after construction (Civil Engineering Code of Practice No 2, 1951).

4.1 IN-SITU STRESS CONDITIONS

High lateral stresses present in overconsolidated clay deposits are a result of their geological history which has involved removal of overburden after their initial deposition and consolidation. As an example, estimates of the overburden removed from the London Clay suggest that the thickness involved generally decreases from west to east, with quoted values of about 380m at Ashford Common (Bishop, Webb and Lewin 1965); 180m in Central London (Skempton and Henkel 1957); 150m at Bradwell (Skempton 1961) and 60m at the Bell Common Tunnel (Hellings, Burland and Symons 1990).

Fig 1a shows the likely form of the relations between effective horizontal and vertical stress in the ground during deposition of the clay (OA, OA') and removal of overburden (AB, A'B') for two elements, one located near to the final ground surface and the other at a greater depth. During deposition both elements follow stress paths for normally consolidated deposits with an earth pressure coefficient K_{onc} taken as equal to 1-$\sin\phi'$ (Jaky 1944). For the element at shallow depth the reduction in vertical stress (σ_v') caused by the removal of overburden (AB) is of sufficient magnitude for the earth pressure coefficient to approach the lower bound Rankine passive value ($K_p = 1 + \sin\phi'/1-\sin\phi'$). At greater depth the
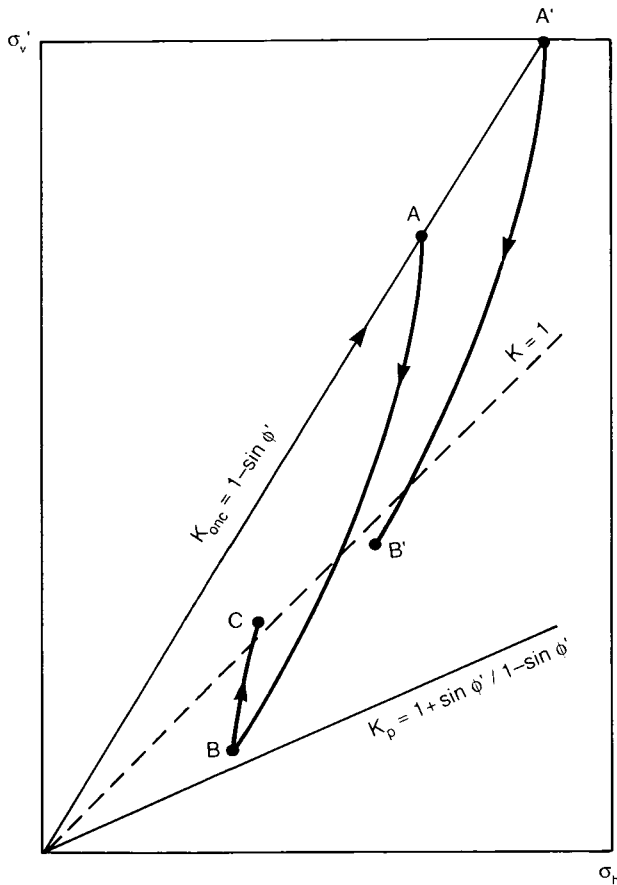
(a) Likely form of
unloading–reloading relation
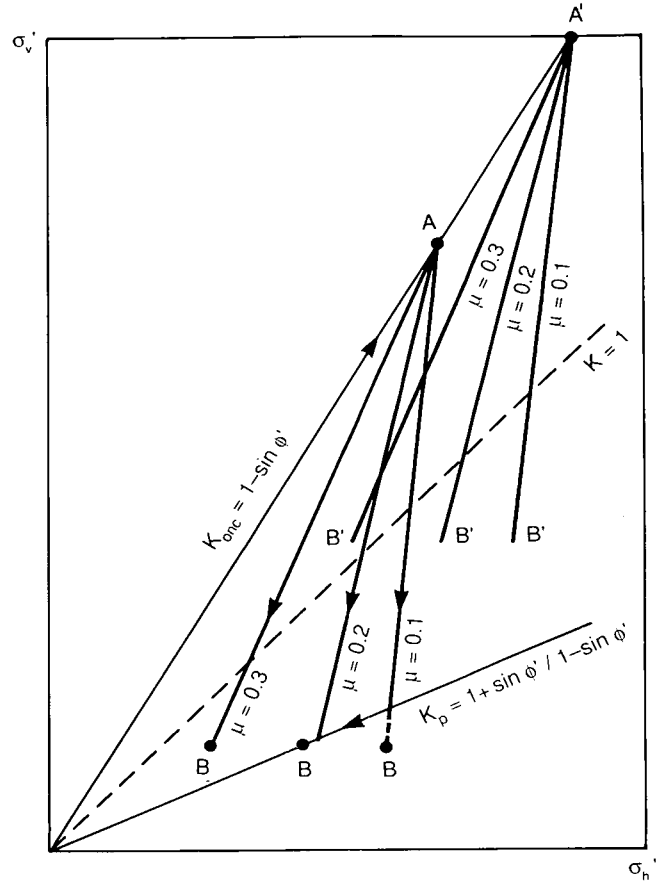
(b) Unloading relation
assuming elastic behaviour

Fig.1 Variations of effective stresses during unloading

reduction in vertical stress serves only to increase the earth pressure coefficient above the normally consolidated value.

Also illustrated in Fig 1a is the effect of a small reloading (BC) due, for example, to deposition of more recent alluvium or a reduction in the long term ground water level. Such a reversal in stress path is likely to lead to a markedly stiffer response from the clay, resulting in a relatively small increase in horizontal effective stress with a consequent large reduction in the magnitude of the earth pressure coefficient at shallow depth. For this reason, considerable variation in the profiles of in-situ earth pressure coefficient at rest (K_o) can be expected at different locations within an overconsolidated clay deposit, as discussed by Burland, Simpson and St John (1979).

Approximations to the unloading curve can be determined assuming elastic-plastic behaviour of the soil as described in Appendix A and these are also shown in Fig 1b for different values of Poisson's Ratio (μ). The approach can also be used to assess the K_o profile for an overconsolidated clay if the maximum preconsolidation

pressure is known. This is illustrated in Fig 2(a) and (b) assuming a water table at ground level throughout and the removal of depths of overburden (h) of 150m and 380m respectively. Over the upper part of the profile the earth pressure coefficient at rest K_o is assumed equal to the Rankine passive value K_p. Below this failure zone the K_o value will decrease progressively towards the normally consolidated value K_{onc} at infinite depth.

The results are in reasonable accord with the values of the K_o at various depths in London Clay determined from laboratory tests at Bradwell by Skempton (1961) and at Ashford Common by Bishop, Webb and Lewin (1965). In this latter profile the high K_o value at about 9m depth was derived from tests on samples of the London Clay of markedly lower plasticity and higher strength. The effective horizontal pressure at this depth would correspond to the passive pressure calculated using a ϕ' of 28° together with a cohesion component c' of about 15 kN/m² which is close to the lower limit of the peak strength reported from conventional laboratory triaxial compression tests. In-situ and laboratory measurements at other sites (Powell and Uglow 1986) have also indicated K_o values at shallow depth which are significantly
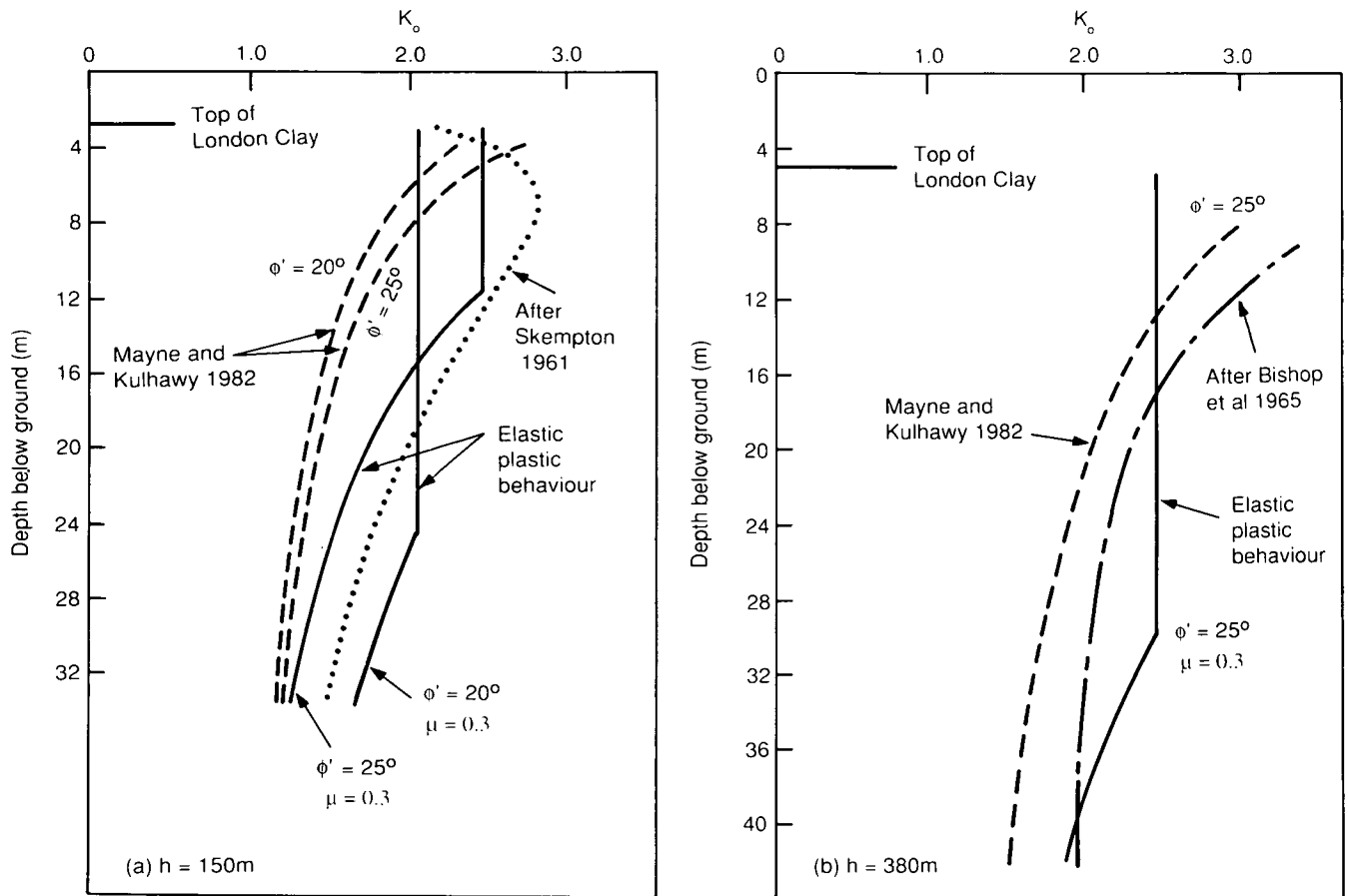
Fig.2 K_o – Depth relations

greater than the calculated lower bound Rankine passive coefficients (based on the angle of shearing resistance alone).

Also shown in Fig 2 are the corresponding relations obtained using the formula proposed by Mayne and Kulhawy (1982) based on an analysis of laboratory test data:

$$K_o = K_{onc} \cdot OCR^{\sin\phi'} \qquad (2)$$

where OCR is the overconsolidation ratio.

For the input parameters selected this equation appears to underestimate the K_o values except at shallow depth within the London Clay.

4.2 SHEAR STRENGTH PROPERTIES

The shear strength parameters of stiff clays are usually assessed from results of conventional drained or undrained triaxial compression tests carried out as part of the routine site investigation. Such tests are still frequently conducted on small 38mm diameter specimens, obtained from U100 samples, which are not large enough

to represent the bulk properties of a fissured material. Skempton (1977) has shown, for example, that the weathered brown London Clay gives an apparent cohesion intercept, c', of about 14 kN/m² from tests on 38mm diameter samples, while tests on large 250mm diameter samples give a c' value of about half this value. These apparent cohesion values are from linear relations between the peak shear strength and effective normal stress and can be compared with a c' value of about 1kN/m² for the bulk strength of the clay from back analysis of first time failures in the slopes of cuttings.

The stress paths and stress levels in conventional triaxial compression tests are likely to differ from those prevailing in the vicinity of an embedded wall as it approaches long term failure. In the conventional triaxial compression test, failure is induced under axi-symmetric conditions with vertical stress increasing and radial stress maintained constant. The level of radial stresses usually applied is such that failure occurs in drained tests at normal effective stresses in the range 100 to 1000 kN/m². In the vicinity of an embedded wall, failure of the soil would normally take place under plane strain conditions. Behind the wall this is likely to occur with reducing lateral stresses and approximately constant vertical stresses, at

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normal stresses for a typical height of retaining wall, in the range 0 to 150 kN/m². In front of the wall, soil failure is likely to occur under decreasing vertical stresses with smaller reductions in lateral stresses, at effective normal stresses typically in the range 0 to 100 kN/m².

The effect on the strength parameters of stress paths more representative of those in the vicinity of a retaining wall is illustrated by the work of Hellings et al 1990. Relatively small differences in strength parameters were obtained from triaxial compression and extension tests on samples of London Clay from Bell Common, Essex. However, much higher strengths were measured in passive stress relief tests simulating failure on the excavated side of a retaining wall, confirming earlier work by Burland and Fourie (1985). Helling's tests were conducted on small specimens and it is not known if these enhanced strengths could be developed and sustained in the field where the clay is fissured and subject to effects such as cyclic stress changes caused by seasonal fluctuations in ground water level.

At low normal stress levels the failure envelope at peak strength may be markedly non linear with curvature towards the origin as illustrated in Fig 3. In this region the peak strength parameters c' and ϕ' determined assuming a linear relation between shear strength and normal stress from tests at higher stress levels will overestimate the strength. A better estimate of the peak strength is to determine a value of ϕ'_p taken as a secant to the peak failure envelope over the maximum stress range likely to be encountered in the field situation, as illustrated in Fig 3.

The peak strength of an intact sample of overconsolidated clay occurs at relatively small strain and the strength then decreases towards the critical state value, as concentration of strain develops in discrete failure zones (Bolton 1979). Beyond this point the strength decreases further to reach the residual value at very large strains. This form of stress-strain relation means that in the field, where variations in shear strain can be expected within a large failing mass of overconsolidated and fissured clay, the peak strength that can be mobilised is likely to lie below the value measured under uniform strain conditions in laboratory tests on small intact samples.

Burland (1990) has suggested that the post-rupture strength determined at small strains immediately after the peak, may be relevant to many stability problems including retaining walls in stiff clay. He has also shown that for the London Clay the post rupture failure envelope at low normal stresses lies close to and slightly above the critical state strength. The critical state strength therefore represents a safe and conservative approach for retaining wall design in stiff fissured clays (Bolton 1979). The critical state angle of shearing resistance ϕ'_c can be determined from tests on normally consolidated remoulded samples and corresponds with the 'worst credible' angle of friction referred to by Padfield and Mair in CIRIA Report 104 (1984).

4.3 STIFFNESS PROPERTIES

To predict the behaviour of embedded retaining walls and the adjoining ground using numerical techniques, requires information on the stiffness properties of the soil.
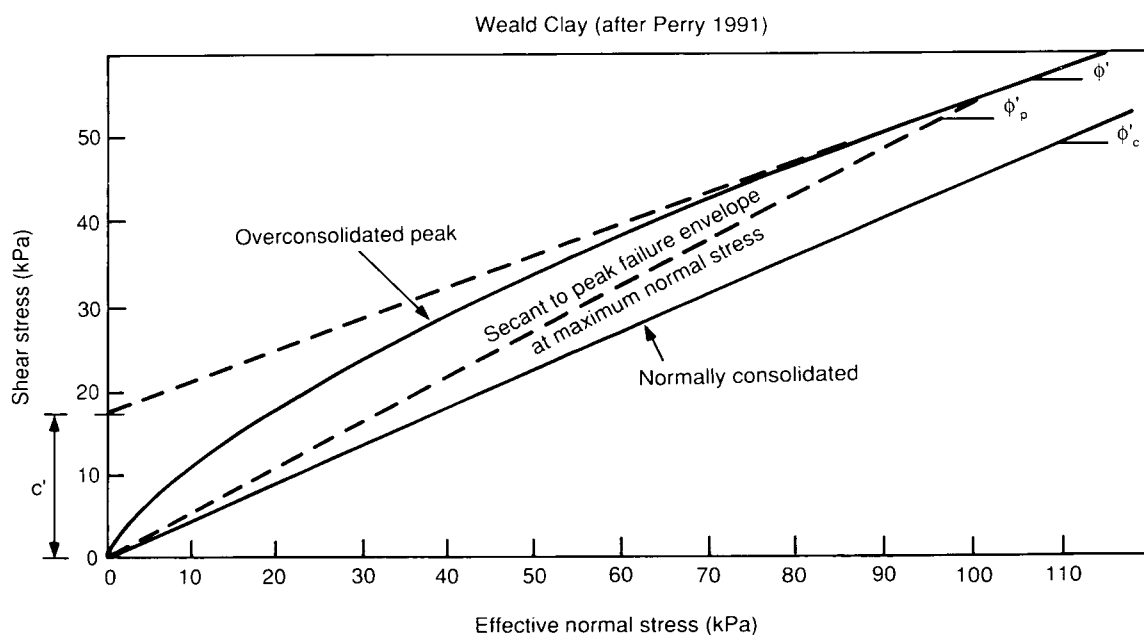
Weald Clay (after Perry 1991)

Fig.3 Failure envelopes for overconsolidated stiff clay

Recent research has shown that the stress strain behaviour of soil before failure is highly non-linear so that the soil moduli can decrease by up to an order of magnitude as the level of imposed strain increases (Atkinson and Salifors 1991). Although laboratory and field techniques are now available for the determination of the variation of soil stiffness with strain level (Jardine et al 1984, Jardine 1991) such methods are not normally used as part of the routine soils investigation.

At present most analyses for design purposes model the soil behaviour as linear-elastic perfectly-plastic. For the London Clay typical linear elastic stiffness profiles for use in this type of analysis (Potts and Burland 1983a) have been derived from back analyses of retaining wall behaviour (St John 1975, Burland and Kalra 1986). A limitation of this approach is that it is likely to overestimate the extent and to underestimate the gradient of ground movements (O'Brien 1991). Where adjacent structures are founded in the retained ground and are likely to be affected by the retaining wall construction, use of non- linear finite element analyses, which account for the variation in soil stiffness with strain level, may therefore be required for predicting the magnitude and distribution of ground movements. Higgins et al (1989) give comparisons of the measured ground movements behind the retaining walls of the Bell Common tunnel with predictions obtained from both linear and non-linear finite element analyses. The non-linear analyses incorporated the variation in shear modulus with strain level determined from a programme of laboratory testing on high quality samples from the site (Hellings et al 1990).

5. LIMIT EQUILIBRIUM CALCULATIONS

For unpropped and singly propped walls, limit equilibrium methods are normally used in design to determine both the required depth of embedment and to assess the loading on the wall for dimensioning of the structural components.

5.1 WALL STABILITY

The required depth of embedment is determined by considering the likely mode of wall collapse for failure in the surrounding soil, as illustrated in Fig 4, for both an unpropped wall and a wall propped at the top. The method considers bodily rotation of the wall about a point of support and is not therefore applicable where the prop is built into the wall structure, as in the case of a portal frame. At collapse, limiting active and passive earth pressures will act on the wall and, to prevent such a failure, factors of safety are normally introduced into the calculations and the design depth of wall penetration then determined for moment equilibrium about the point of support. Among the simplifications usually made in the calculations are the assumptions of linear distributions of earth and water pressure and the neglect of horizontal shear at the base of the wall and in the soil at the level of the support.

A single 'lumped' factor of safety is normally used which is intended to cater for the uncertainties in the design parameters and calculation method and also to ensure that deformation in service will be within acceptable limits. Depending on which method is employed, the calculations may consider the nett or gross pressures acting on the structure. The factor of safety can also be applied as a reduction factor on the nett or gross passive resistance or the soil shear strength parameters; or alternatively as an enhancement factor on the depth of wall penetration at collapse. In this context, since limiting active and passive thrusts can act in concert on a structure only at collapse, all methods give the same factor of safety of unity for this condition.

Fig 5 illustrates the pressure distributions at equilibrium considered in a number of common methods used to determine the depth of embedment for the simple case of a stable wall in dry cohesionless soil. The CP2 Method is based on that given in the Code of Practice (1951) in which the factor of safety F_p is applied as a reduction factor on the passive resistance of the soil. It has been shown (Burland et al 1981) that use of this method in total stress analyses can yield two different depths of penetration for the same value of factor of safety in clays of constant undrained strength.

The Nett Pressure Method, which has been widely used for the design of sheet pile walls in predominantly granular soils is described in the British Steel Corporation's Piling Handbook (British Steel Corporation 1988). With this method, the factor of safety F_{np} is applied as a reduction factor on the nett passive resistance, obtained as the difference between total soil and water pressures acting on the front and back of the wall below excavation level. It can be demonstrated that this will always give a smaller depth of penetration than the CP2 Method if the same value of factor of safety is used with both approaches.

The Strength Method is similar to that used in slope stability calculations. The factor of safety F_s is applied as a reduction factor on the soil strength parameters and these reduced strength parameters used to calculate 'mobilised' active and passive soil pressures. This method has the merit that it applies the factor of safety to the parameters over which there is normally the greatest uncertainly (Section 4.2). With this approach, the depth of penetration obtained is relatively sensitive to the value of factor of safety employed, with large changes in depth of penetration resulting from a small variation in the magnitude of F_s, as illustrated in Fig 6.

The Burland-Potts Method (Potts and Burland 1983b) is a more recent formulation, developed in an attempt to overcome some of the inconsistencies which can arise in certain design situations with the other methods. It has some similarity with the approach followed in calculating the bearing capacity of a strip footing, and considers only the active pressures in the soil generated by the retained ground above excavation level. The factor of safety F_r is applied as a reduction factor to the nett passive
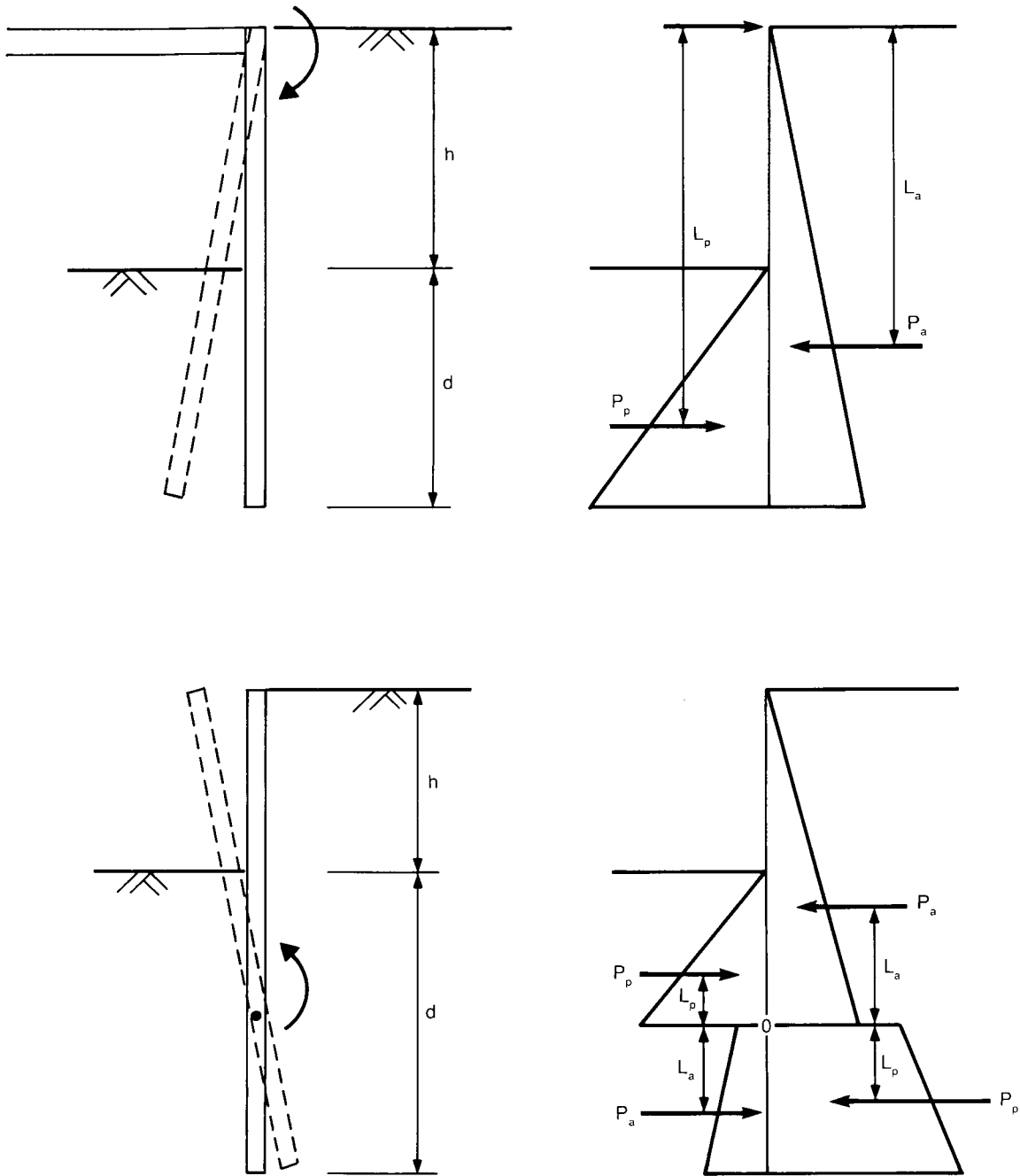
Fig.4 Failure mechanism for a propped and unpropped
embedded wall due to soil failure
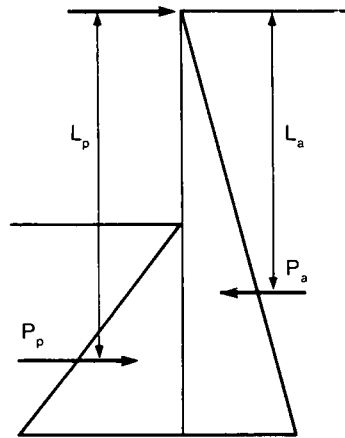
(a) Code of practice CP2

$$F_p = \frac{P_p L_p}{P_a L_a}$$
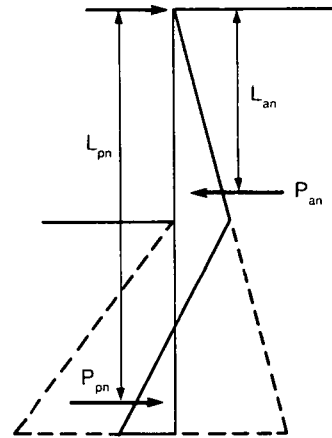
(b) Nett total pressure method

$$F_{np} = \frac{P_{pn} L_{pn}}{P_{an} L_{an}}$$

(c) Strength method

$$P_{pm} L_{pm} = P_{am} L_{am}$$

$$F_s = \frac{\tan\phi'}{\tan\phi_m'}$$
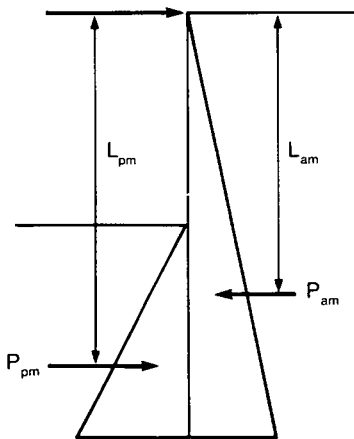
(d) Burland-Potts method
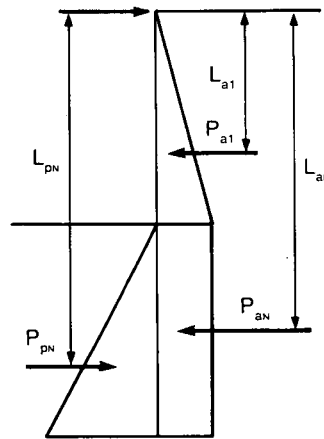
$$F_r = \frac{P_{pN} L_{pN}}{P_{a1} L_{a1} + P_{aN} L_{aN}}$$
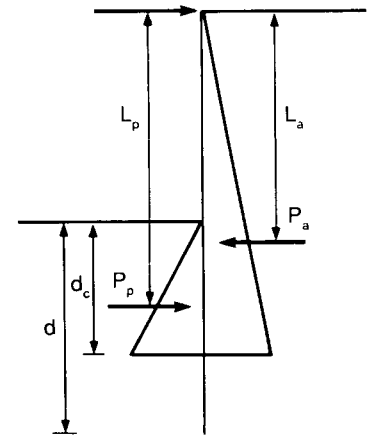
(e) Embedment method

$$P_a L_a = P_p L_p$$

$$F_d = \frac{d}{d_c}$$

Fig.5 Methods of determining the required depth of embedment (d)

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Fig.6 Variation in factors of safety with embedment (after Potts and Burland 1983)

resistance of the soil obtained as a difference between the passive and active thrusts of the ground below excavation level.

Parametric studies of wall stability have been carried out for both unpropped walls and walls propped at the top using these four design methods in conjunction with a wide range of input parameters (Potts and Burland 1983, Symons and Kotera 1987). These have shown that, to achieve consistency in the 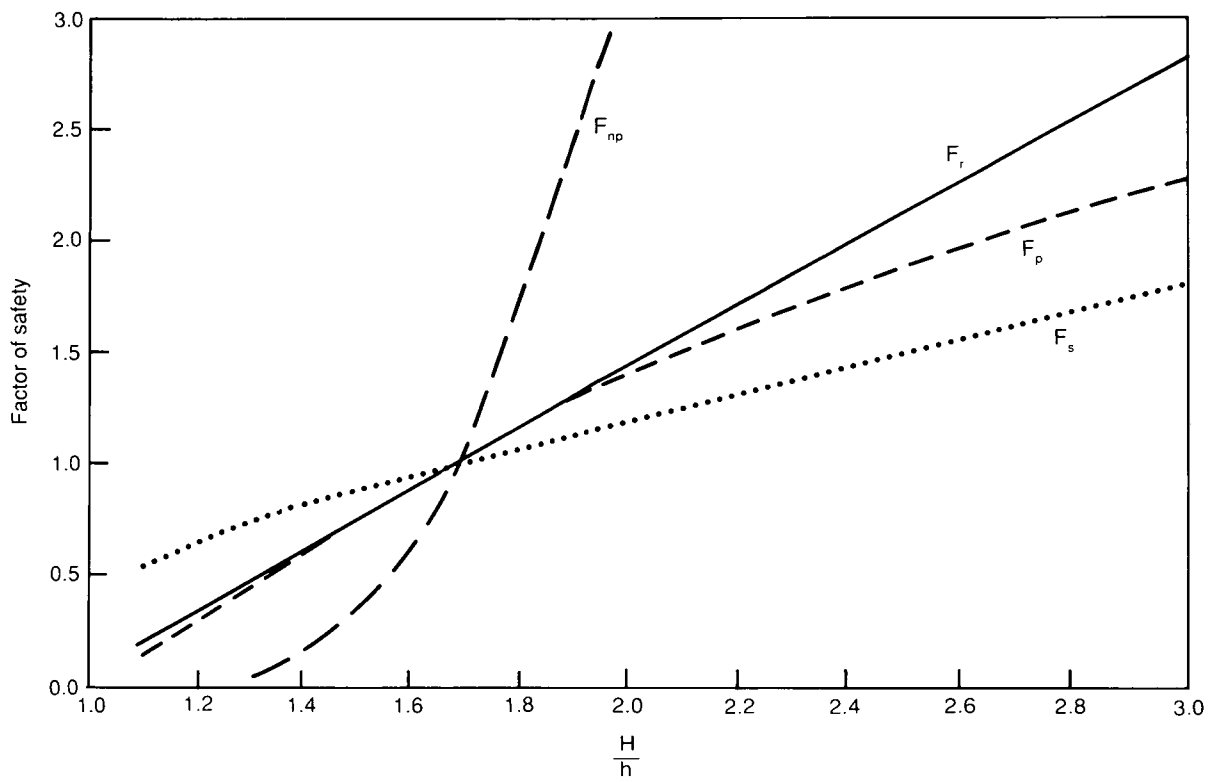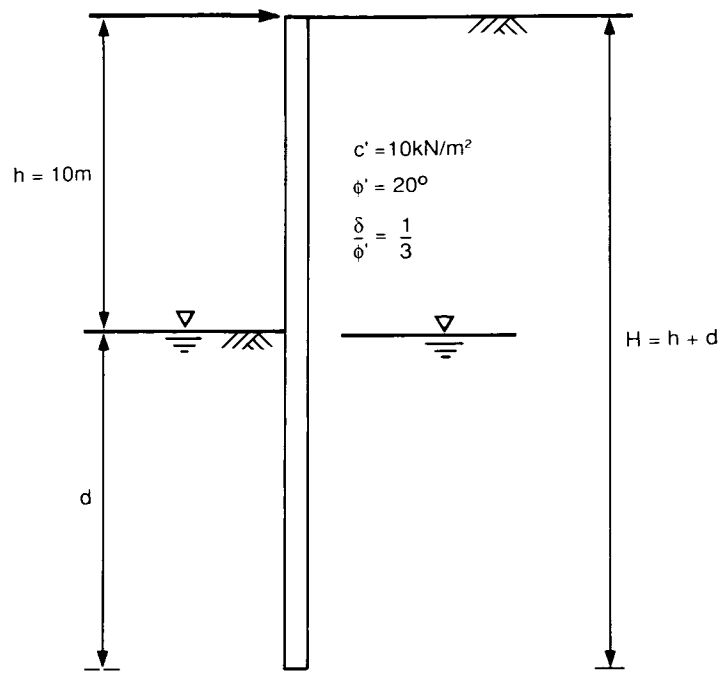design depth of embedment, it is necessary to specify separate ranges of factor of safety with each method (CIRIA Report 104 1984). The choice of method can then be left to the designer although it would clearly be good practice to ensure that the depth of embedment is always independently checked using an alternative method.

Fig 5 also shows the pressure distributions used in the Embedment Method (Tschebotarioff 1973). This is an empirical approach in which the factor, F_d, is applied as an enhancement factor on the depth of wall penetration below excavation level, required at equilibrium, with fully active and passive soil pressures acting on the wall.

The parametric studies referred to above have provided useful information on the relative magnitude of the factors of safety required with different methods, and on the sensitivity of the calculations to the selected input parameters (Symons 1983). However, such studies cannot indicate the absolute values of safety factor required to produce an economic design, whereby ground and wall movements in service will be within acceptable limits. To investigate this, information is required on the factors of safety from back analyses of walls which have performed satisfactorily in service (Carder and Symons 1989, 1990, Symons and Carder 1990, 1991) in conjunction with model studies of wall behaviour prior to and at collapse (Bolton and Powrie 1987, 1988).

In considering the various alternative design approaches, as outlined above, the factor of safety applied must be related to the selection of the design parameters, the type of structure and the ground conditions. For example, the apparent success of the Nett Pressure Method, based on relatively low factors of safety, for the design of sheet pile walls in granular soils, may well stem in part from the use of conservatively low strength parameters, making this a 'de facto' Strength Method. Moreover, for more flexible structures of the sheet pile type, non-linearity in the pressure distributions resulting from enhanced wall deflections, may serve to improve the actual wall stability compared with the calculated values based on linear pressure distributions.

The ground water conditions in the vicinity of an embedded cantilever or propped retaining wall and the friction acting at the wall-soil interface have a major influence on the required depth of penetration. This is illustrated in Fig 7 which gives generalised relations between the factor of safety F_s by the Strength Method and normalised depth of penetration below excavation level for both unpropped walls and walls propped at the top. Results are shown for a range of angles of shearing resistance

assuming zero and full wall friction (δ) for both dry conditions and for linear seepage between water tables located at ground level on both sides of the wall.

For walls supported by a prop located at, or close to, final excavation level, two modes of wall movement are theoretically possible; these are backward and forward rotation of the wall about the prop position. The relations between the factor of safety by the Strength Method (F_s) and normalised depth of penetration below excavation level for a wall propped at excavation level are shown in Fig 8 for both forward and backward rotation. Fig 8(a) is for dry conditions while Figure 8(b) is for linear seepage between water tables located at ground surface level on both sides of the wall. Wall friction (δ) equal to zero has been assumed for both cases. At small depths of embedment ($d/h < 0.5$) forward rotation of the wall is the more likely mode of behaviour. As the depth of penetration increases the calculated factor of safety against backward rotation decreases to a minimum value and then gradually rises. Compared with unpropped walls and walls propped at the top, those propped at or close to formation level require the smallest depth of embedment (Figs.7 and 8). This advantage is offset by higher imposed loading on the structural elements and the need to install supports at an early stage if the risk of excessive ground movement during excavation is to be avoided.

5.2 BENDING MOMENTS AND PROP LOADS

To assess the bending moments in the wall and the maximum loading in any supports for their structural design, use is often made of simple calculation methods given in Codes of Practice and design guides such as CIRIA Report 104. These are generally based on the assumption of linear distributions of earth and water pressures, similar to those employed in assessing wall stability. They therefore take no account of the relative stiffness of the soil and structural elements and only require information on the density and shear strength properties of the soil and the ground water levels. However such simple calculations may result in unrealistic assessments of bending moments and prop forces acting in service. For major structures, increasing use is therefore being made in design of analytical techniques, such as the boundary or finite element methods which attempt to model the behaviour of the soil-structure system. These methods require additional information on the stiffness properties of the soil and structural elements, the insitu stresses in the ground and the proposed construction sequence. The difficulty of determining reliable values for these input parameters often limits the accuracy of predictions obtained from such analyses at the design stage. In this Section the factors likely to influence the soil pressure distribution acting on an embedded wall in service are considered and the structural loading effects, calculated using simplified methods, are compared with results from finite element studies.

For a given wall geometry there are numerous linear distributions of earth and water pressure which will satisfy the criteria that the wall and its support system are in

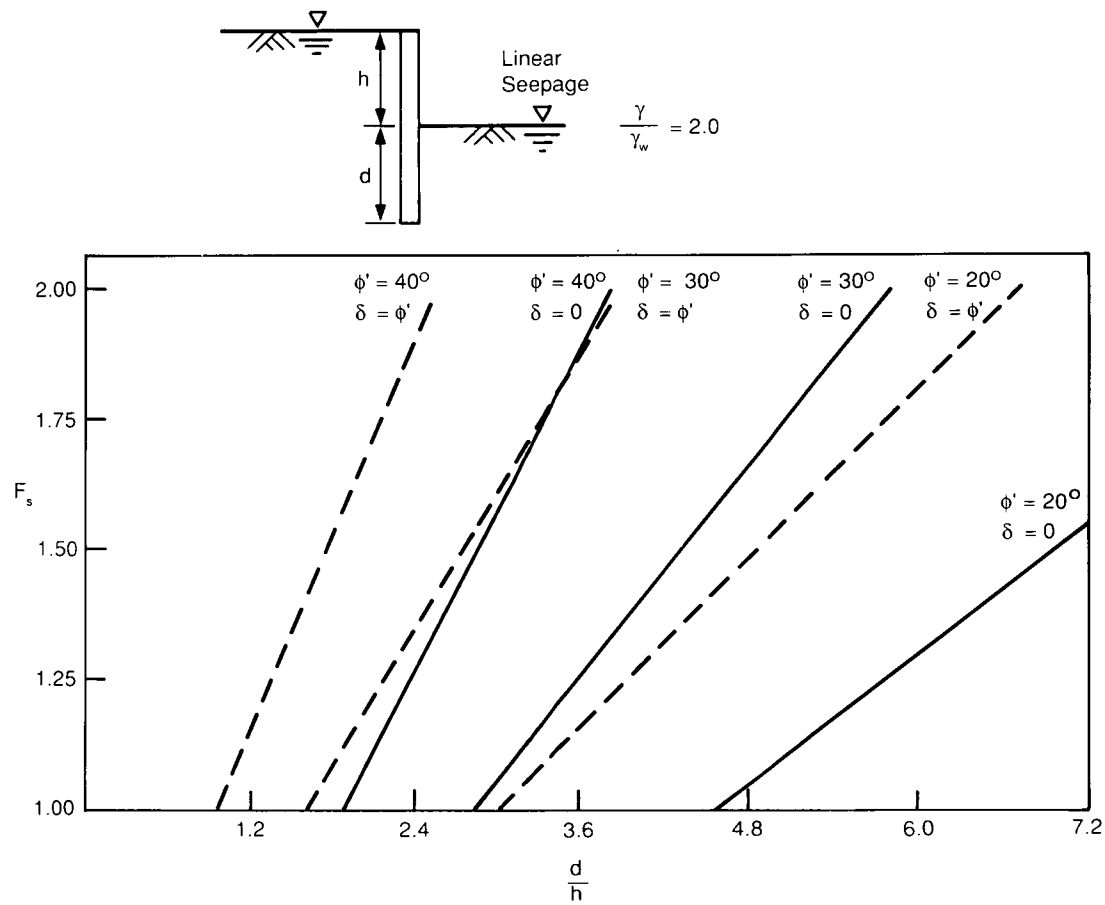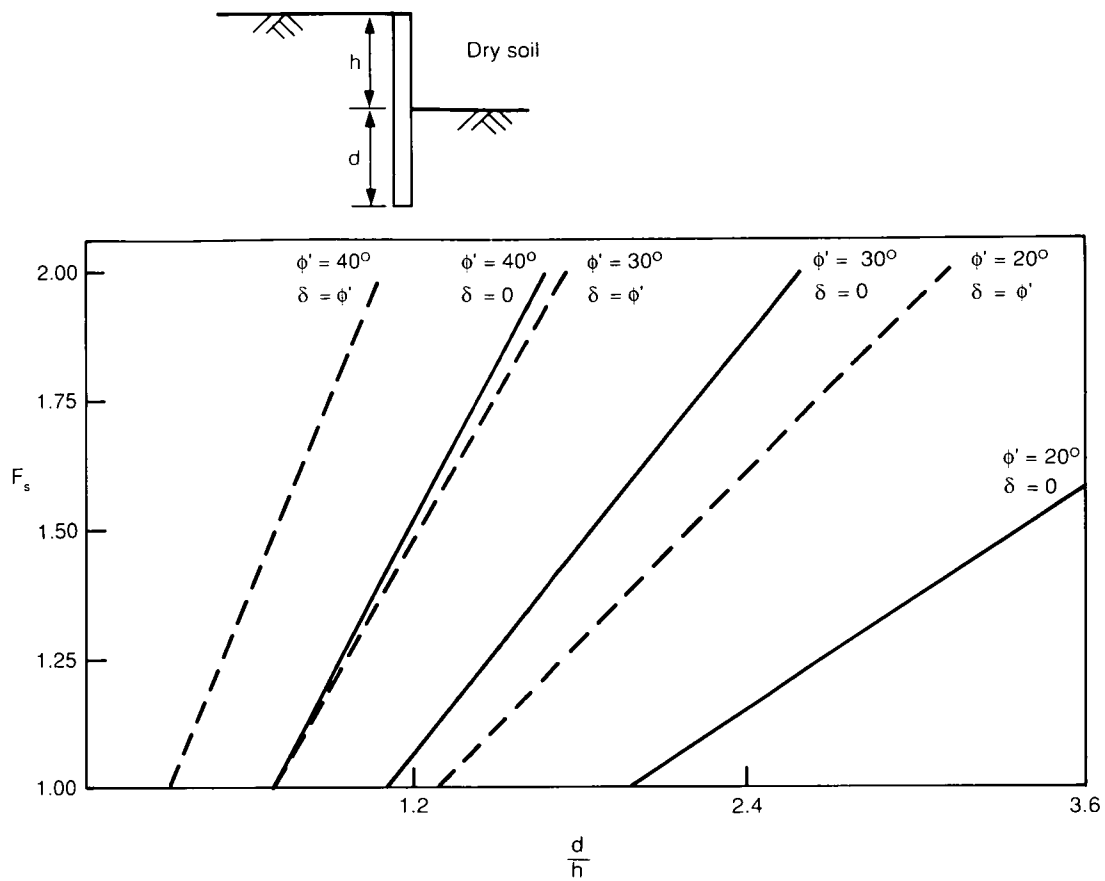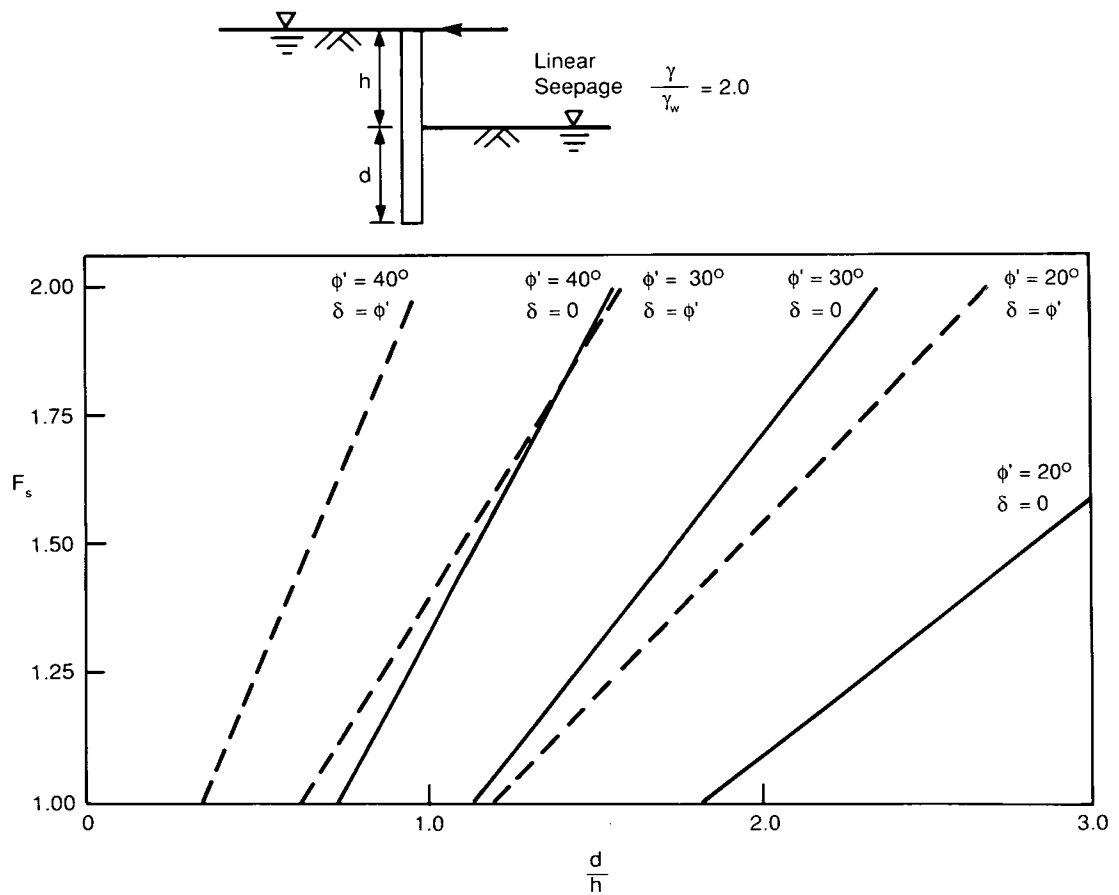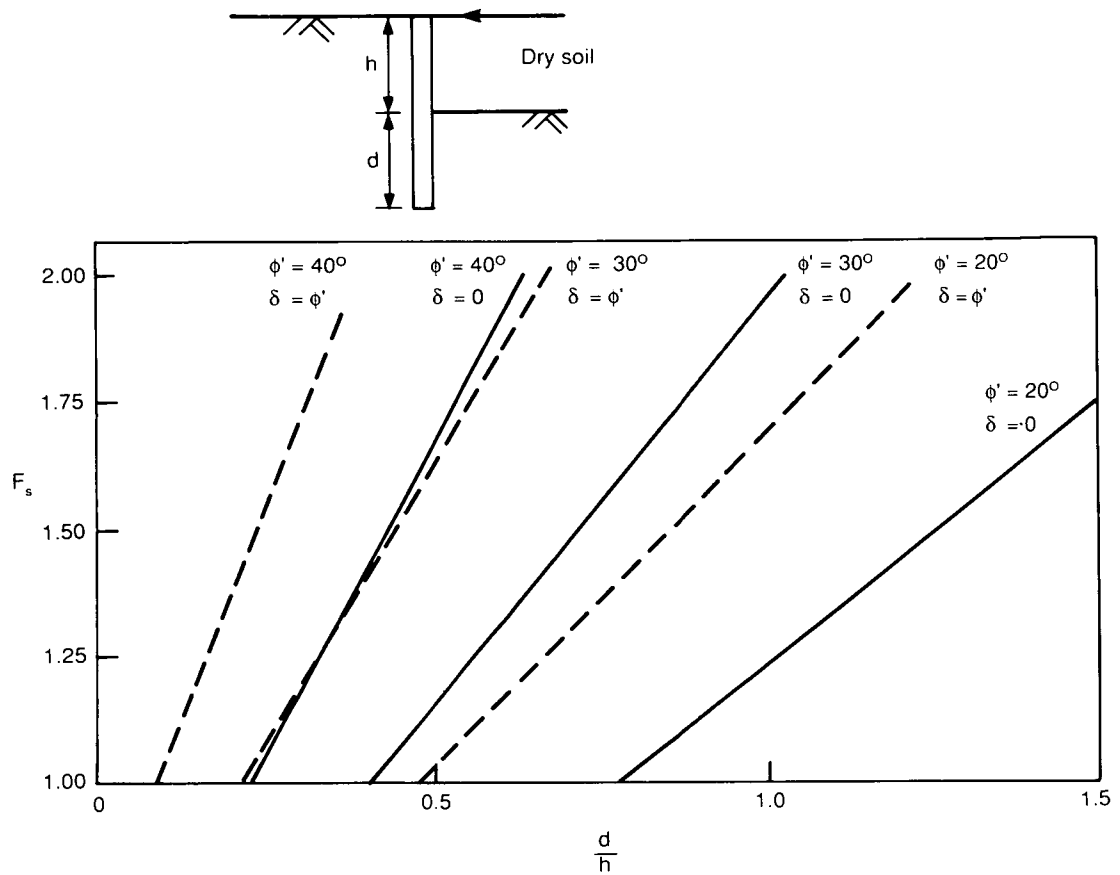**Fig.7 (a) Relation between factor of safety F_s and d/h for unpropped wall**

Fig.7 (b) Relation between factor of safety F_s and d/h for propped wall
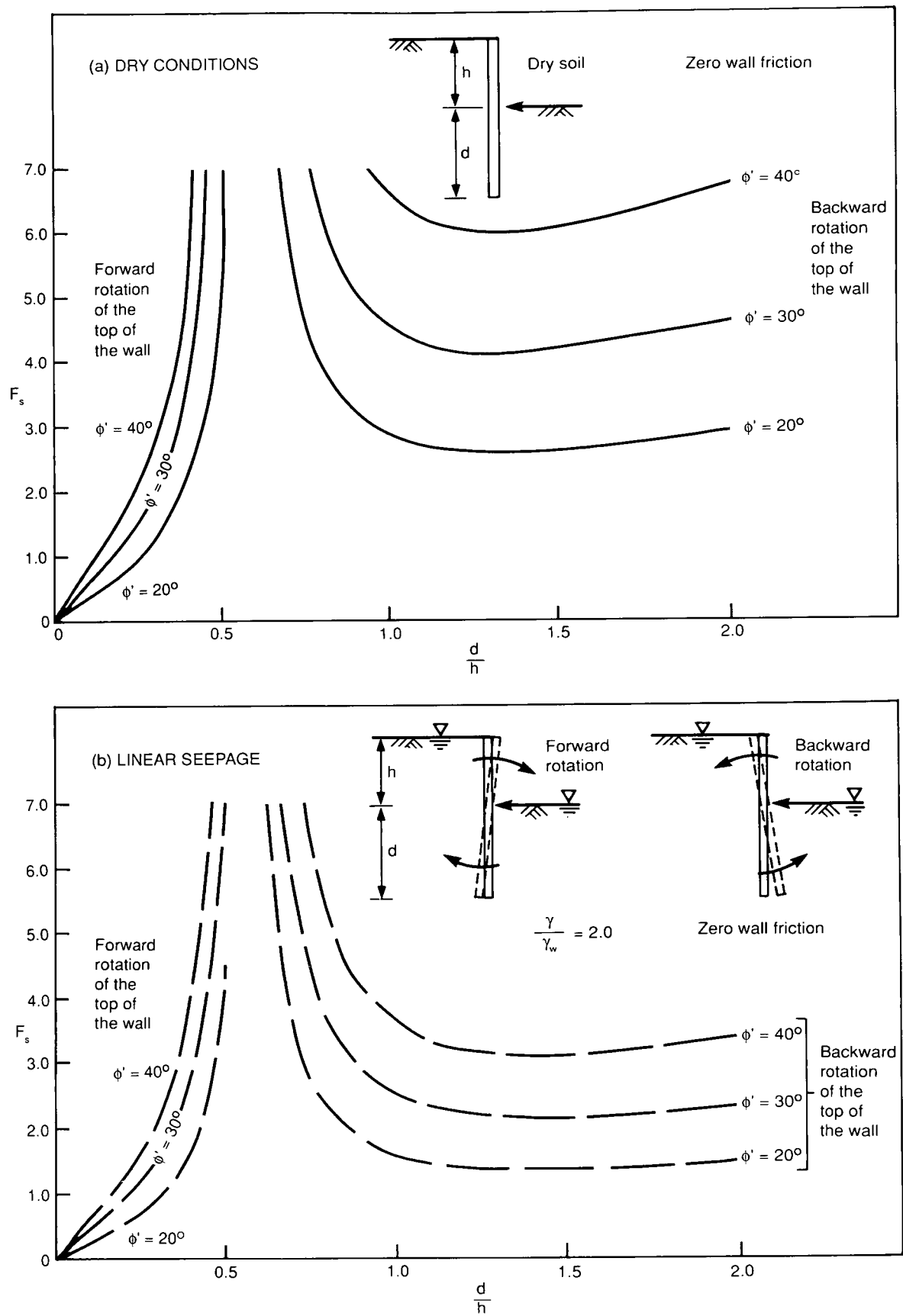
Fig.8 Relations between factor of safety (F_s) and d/h for wall propped at excavation level

both force and moment equilibrium and that the soil thrusts are everywhere at or within the limiting 'active' and 'passive' values. Fig 9 illustrates the linear pressures assumed to be acting on a propped wall in dry soil with a number of the methods previously described for determining the required depth of embedment for wall stability. With methods such as the Code of Practice approach (Case A), which applies the factor of safety as a reduction factor on the passive resistance of the soil, fully active pressures are assumed to act on the back of the wall in service. With the Strength Method (Case B) in which the shear strength parameters of the soil are reduced by the factor of safety, the pressure distributions at equilibrium correspond to pressures above the fully active values acting on the back, and pressures below the fully passive values acting on the front of the wall. For the Embedment Method (Case C), which applies an enhancement factor on the depth of embedment at failure, fully active and fully passive pressures are assumed to act on the back and front of the wall respectively, over the unfactored depth of embedment. These latter distributions (Case C) correspond to the recommendations given in CIRIA Report 104 (1984) for calculating the unfactored bending moments in unpropped walls and walls propped near the top in stiff clay. Also shown in Fig 9 as Case D are linear distributions of pressure at equilibrium which will maximise the bending moments and prop forces. These correspond to fully passive pressures acting on the front of the wall with enhanced active pressures acting on the back.

In considering the likely validity of these simplified distributions, it is necessary to consider the factors which will influence the soil pressure distributions and thrusts on an embedded retaining wall in service. Among the principal effects which need to be taken into account are the flexibility of the wall, the support system and consequent mode of wall displacement, the initial stress conditions in the ground and changes caused by the construction.

Model tests (Rowe 1952, 1955), numerical studies (Potts and Fourie 1985, Potts and Day 1990) and field measurements (Symons et al 1987) have been carried out to examine the effect of wall flexibility on bending moments and earth pressure distributions for embedded walls. These have shown that under comparable conditions stiffer walls attract larger bending moments than more flexible walls. For more flexible walls the reduced structural loading is largely a result of increasing non-linearity of the pressure distributions with enhanced wall deflection and depend on the support provided, the initial stress condition and soil stiffness. A design approach for sheet pile walls which takes into account the effect on bending moments of the relative stiffness of the wall and the ground is given by Barden (1974).

For walls which are very stiff in relation to the soil, movement will occur principally by bodily rotation and the mode of displacement is then largely governed by the location and type of support. The effects of translation and of rotation about the top and bottom for both rough and smooth rigid walls 5m in height and fully embedded in soil have been examined in numerical studies by Potts

and Fourie (1986). Fig 10(a) shows the development of active and passive thrusts from these studies for a rough rigid wall, starting from an initial earth pressure coefficient K of 2.0: Fig 10(b) gives the corresponding distributions of pressure at fully active and passive thrusts.

These results indicate that the mode of wall movement has a considerable effect on both the magnitude of displacements necessary to mobilise limiting active and passive conditions and on the distributions of earth pressure. They suggest that much larger wall movements are required to achieve limiting conditions for a stiff wall rotating about its base compared to rotation about its top or equal translation. In addition a wall rotating about either its top or base will produce pressure distributions which differ significantly from the linear distributions normally assumed in design.

Potts and Fourie (1986) also looked at the effect of the initial stress state in the ground on the magnitude of translatory wall movements necessary to achieve limiting pressures. Their results for a rough rigid wall starting from initial earth pressure coefficients of 2.0 and 0.5 are shown in Fig 11. These indicate that for high initial earth pressure coefficients, representative of stiff clays, active and passive conditions are mobilised at similar displacements: for soils with low initial earth pressure coefficients, the fully active condition is attained at much smaller displacements than are required to achieve fully passive conditions. This latter finding is in accord with the results of pilot scale studies using a sand backfill reported by Carder et al (1977).

The numerical analyses did not consider the effects of wall construction and therefore provide only qualitative information on the likely behaviour of an embedded wall. They nevertheless suggest that for an embedded wall in normally consolidated deposits, where the in situ earth pressure coefficient at rest (K_0), is low, full active thrusts are likely to be developed at equilibrium for both unpropped walls and walls propped at the top. Moreover in overconsolidated clays, where the insitu earth pressure coefficient at rest (K_0), is initially high, any reduction in horizontal stress caused by the process used to form the wall could be expected to result in significant reductions in bending moment and prop force. In this context, measurements close to the propped secant pile walls of the Bell Common Tunnel suggested reductions in horizontal total stress equivalent to a decrease in K from an initial value of about 1.5 to about unity in the London Clay at 0.6m from the wall, due to the installation of the wall (Tedd, Chard, Charles and Symons 1984). The magnitude of the reduction in in-situ horizontal stress is likely to be governed by the particular technique used to form the wall. Also, the reductions in horizontal total stress at 3m and 6m from the secant pile wall at Bell Common were only about a third of the reductions inferred at 0.6m from the wall, implying the mobilisation of large horizontal shear stresses close to the wall. Whilst this is likely to be possible under undrained conditions, the ability of the clay to sustain these high stress gradients under drained conditions in the longer term is less certain. In this context a gradual increase in total horizontal stress close to the wall was detected at Bell Common
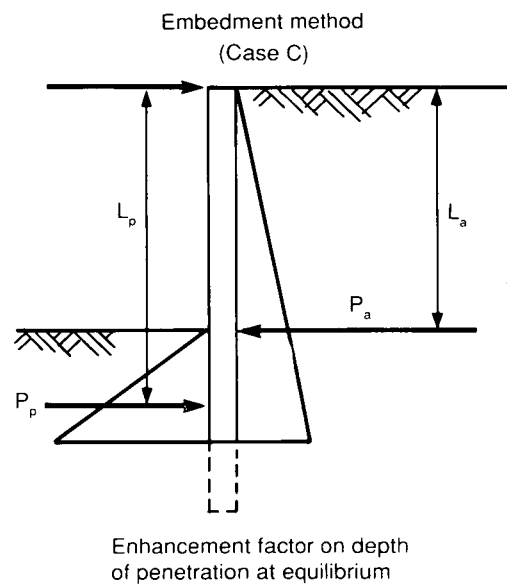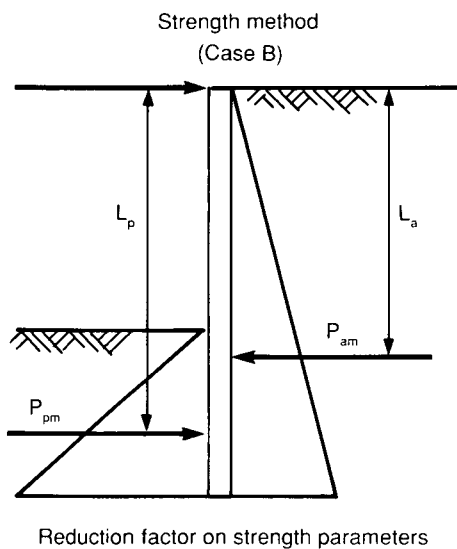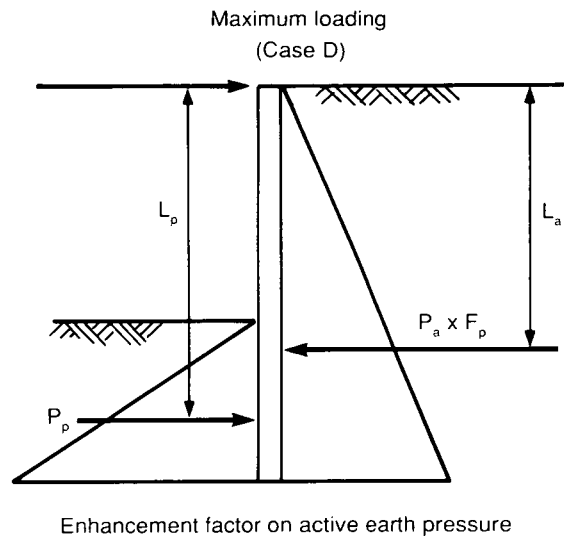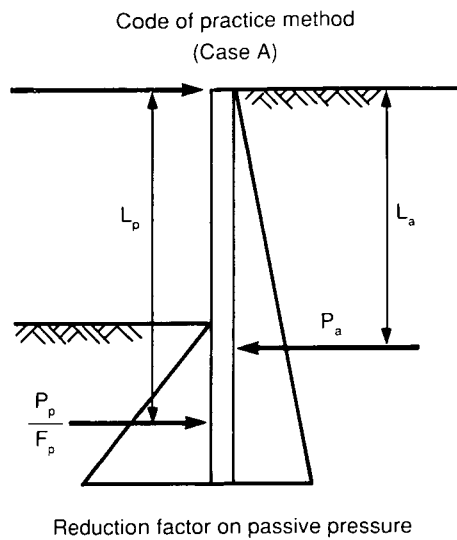
Fig 9 Earth pressure distributions assumed in a number of methods for determining the required depth of wall penetration
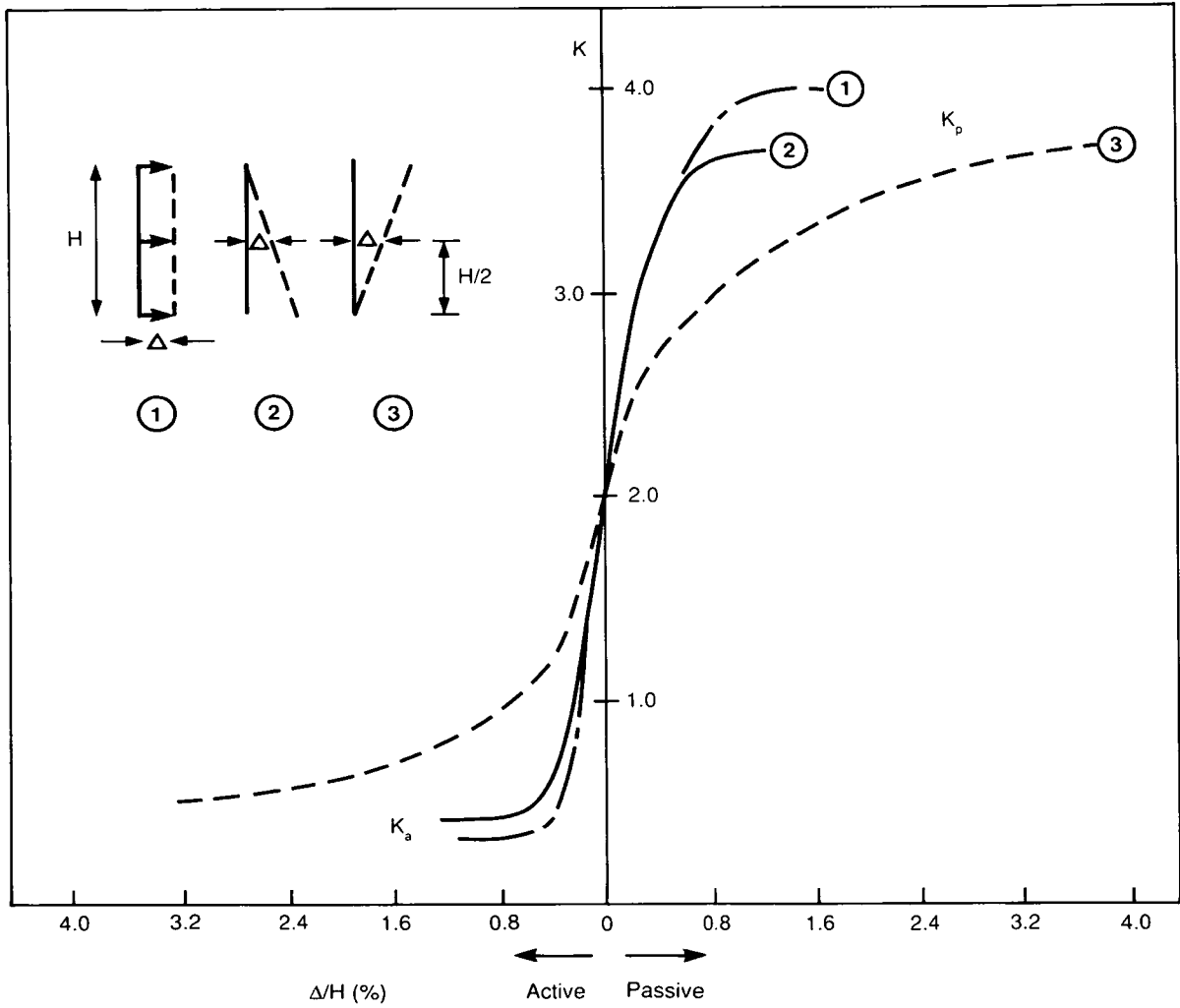
Fig.10 (a) **Development of active and passive pressure coefficients for a rough wall (after Potts and Fourie 1986)**
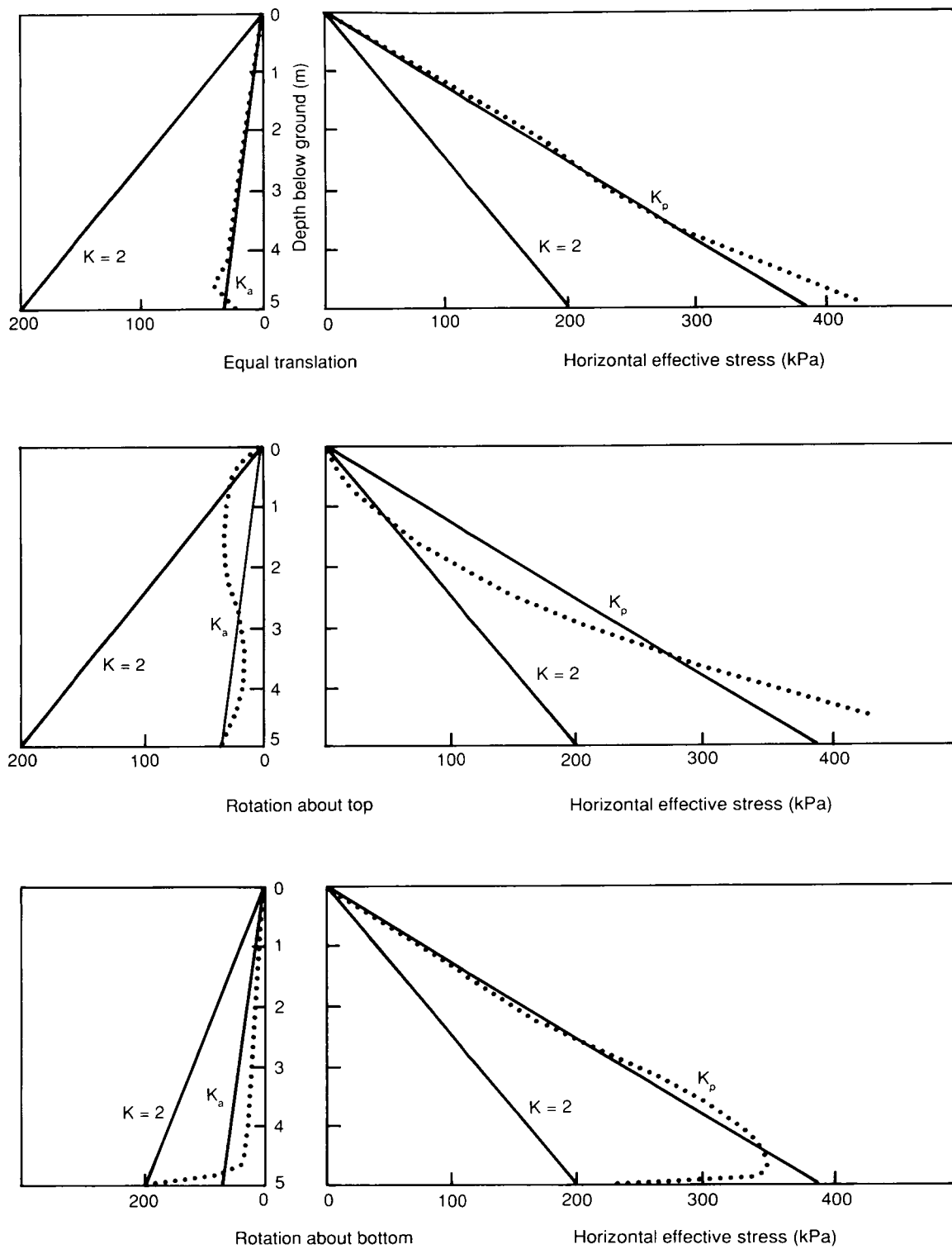
16

Fig.10 (b) Active and passive pressure distributions for a rough wall (after Potts and Fourie 1986)

17
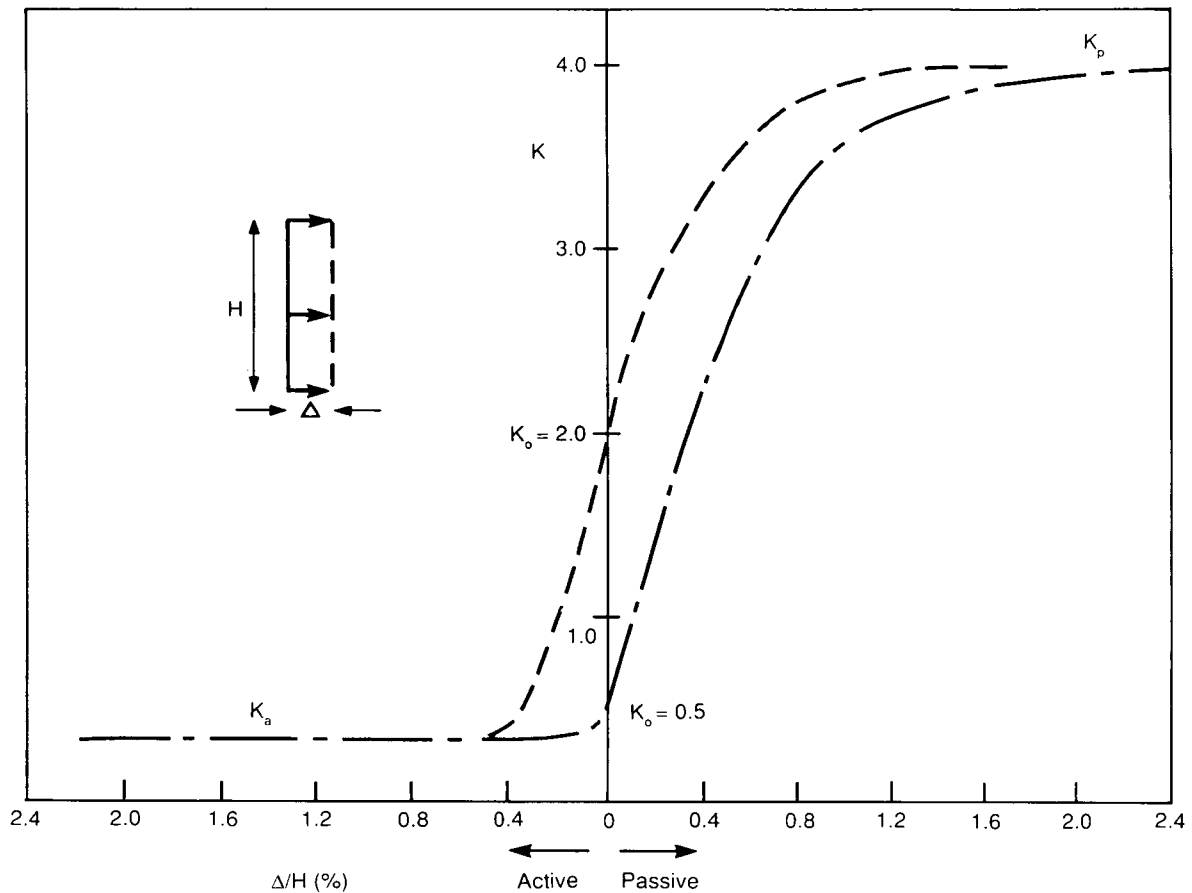
Fig.11   Effect of K_o on active and passive pressure coefficients – Rough wall and
horizontal translation (after Potts and Fourie 1986)

in the short period immediately following its installation and prior to the start of excavation in front of the wall.

For an embedded wall in overconsolidated clay, the reduction in vertical stress caused by excavation in front of the wall, is likely to lead to the development of a plastic zone extending to a considerable depth. Near to the centre of a wide excavation of depth h, an indication of the vertical extent, Z, of this failure zone within which Rankine passive pressures are mobilized can be assessed using the simple elastic-plastic analysis given in Appendix A. Assuming a hydrostatic distribution of porewater pressure below a water table located at ground level, both prior to and after excavation, the extent of this failure zone is shown in Fig 12 for different values of in-situ earth pressure coefficient at rest (K_o). Near the boundary of an excavation the extent of the plastic zone will reduce and Fig 13 shows results from a finite element analysis of a stiff wall completely restrained from any horizontal movement reported by Potts and Fourie (1984). Fig 13(a) shows the extent of the zone where the mobilised shear stress exceeded 99 per cent of the soil strength and Fig 13(b) and (c) show how the predicted lateral stresses in front of the wall at different excavation

depths compare with passive pressures assuming full and zero wall friction. This analysis was for a dry soil with an initial earth pressure coefficient of 2.0.

Consideration of the effect of the factors outlined above would indicate that the highest bending moments in service are likely to occur for stiff walls in overconsolidated clay, which are rigidly propped at the top and where the method used to install the wall is such as to minimise any reduction in lateral stress in the adjoining ground. To examine this further, bending moments calculated using the simplified linear distributions of pressure shown in Fig 9, have been compared with values predicted from finite element analyses for a wall rigidly propped at the top and for an unpropped wall. Both dry soil and conditions of linear seepage between water tables at ground level on both sides of the structure have been considered.

Fig 14 shows the geometry, soil properties and factors of safety determined by the methods described in Section 5.1 for dry conditions. The structural loading effects determined from limit equilibrium calculations for the four cases shown in Fig 9 are given in Table 1. The lowest

18

Ground level before excavation

h

Ground level after excavation

Z

Depth of failure zone

$\phi' = 20°$

$\phi' = 25°$

$\phi' = 30°$ $\mu = 0.2$

$\mu = 0.1$ $\mu = 0.3$

5

4

$\dfrac{Z}{h}$ 3

2

1

0

1.0

2.0

K_o
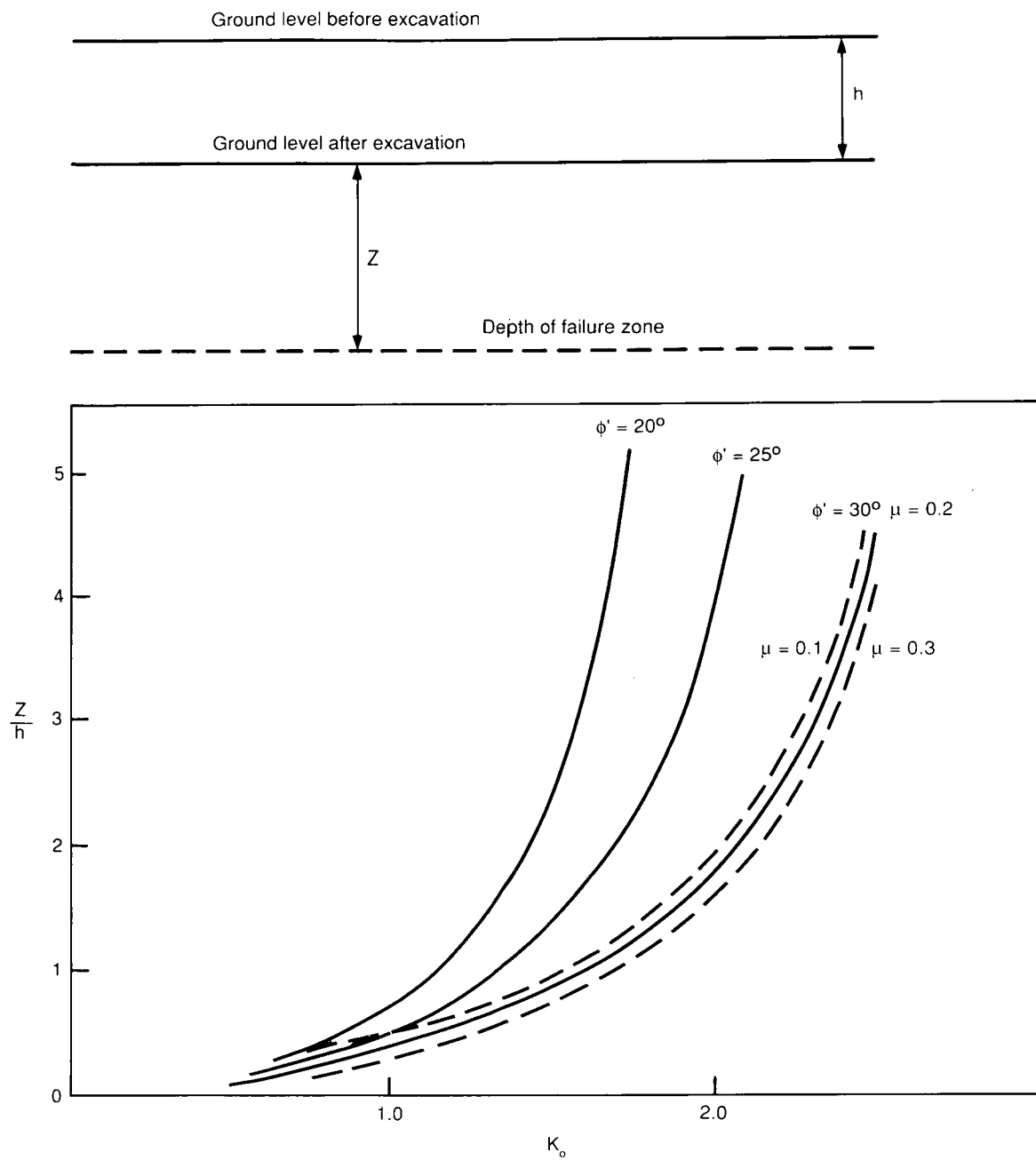
Fig.12   Depth of failure zone at centre of wide excavation in overconsolidated clay

19
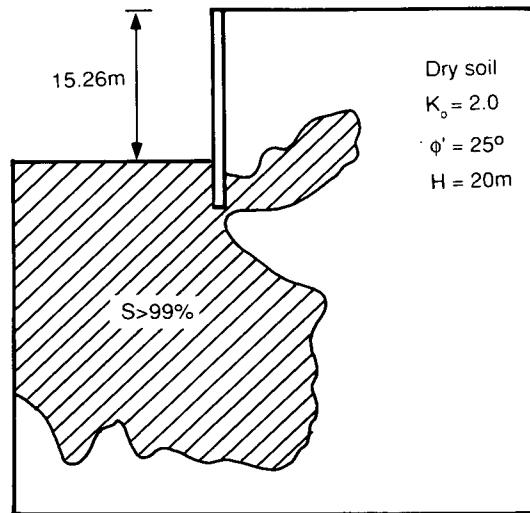
(a) Contours of stress level S>99%
at an excavation depth of 15.26m
for wall completely restrained
from horizontal movement

— — — — — — — Passive $\delta = 0$

——————————— Passive $\delta = \phi'$

●●●●●●●●●●●●●●●●● Predicted

(b) Lateral earth pressure
distribution for
excavation to 15.26m

(c) Lateral earth pressure
distribution for
excavation to 13.26m
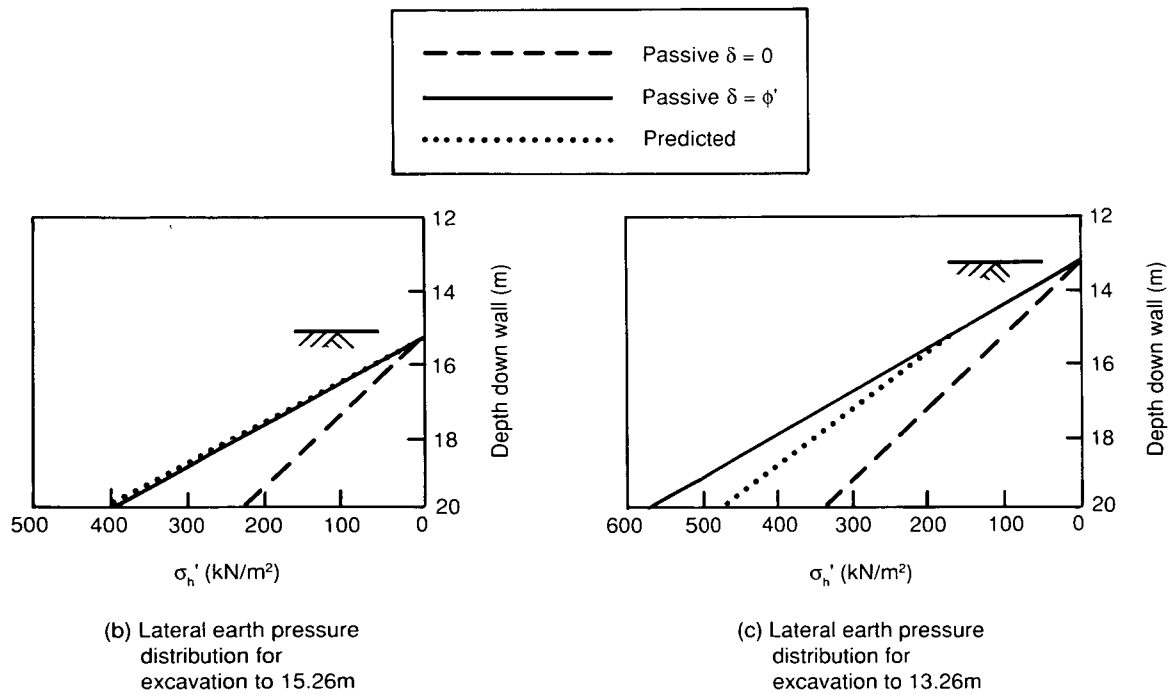
Fig.13   Stress level and earth pressure distributions in front of a wall restained from horizontal movement (after Potts and Fourie 1984)

(a) Propped wall

0m

13.26m

20m

$c' = 0$
$\phi' = 25°$
$\dfrac{\delta}{\phi'} = 1$
$\gamma = 20\text{kN/m}^3$

$F_p = 1.8$
$F_{np} = 3.5$
$F_s = 1.3$
$F_r = 2.0$
$F_d = 1.6$

(b) Unpropped wall

0m

8.85m

18.1m
20m

$c' = 0$
$\phi' = 25°$
$\dfrac{\delta}{\phi'} = 1$
$\gamma = 20\text{kN/m}^3$

$F_p = 1.6$
$F_{np} = 2.2$
$F_s = 1.3$
$F_r = 1.7$
$F_d = 1.4$
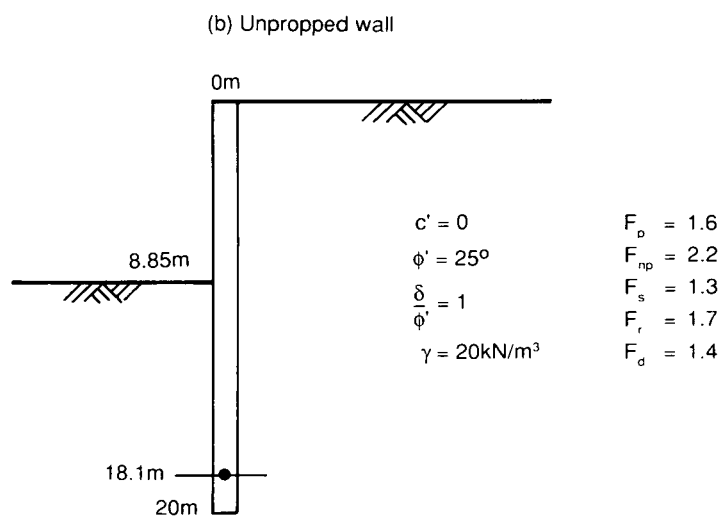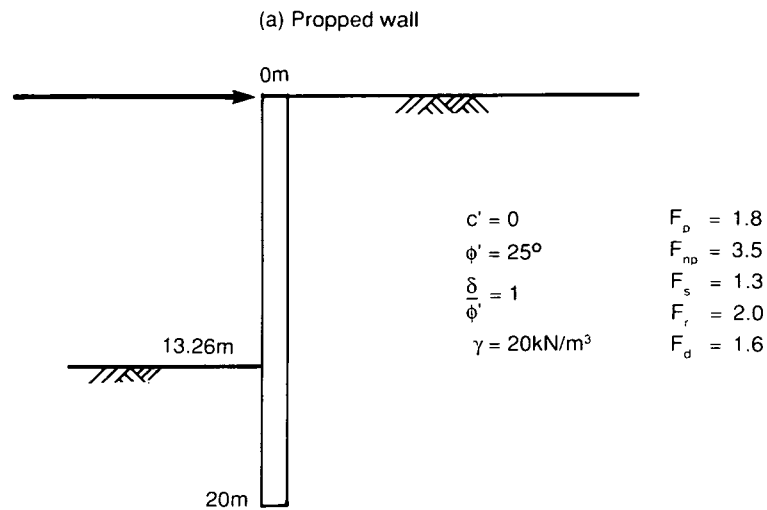
Fig.14   Wall geometry, assumed soil properties and factors of safety – dry soil

TABLE 1

Structural loadings from limit equilibribrium calculations

Propped Wall - Dry Soil (Fig 14a)

Method +	A	B	C	D
K Retained side	0.322	0.413	0.322	0.588
Maximum Bending Moment kNm/m	2130	2710	1680	3895
Prop force kN/m	320	409	273	585

Unpropped Wall - Dry Soil (Fig 14b)

Method +	A	B	C	D
K Retained side	0.322	0.392	0.322	0.520
Maximum Bending Moment kNm/m	1845	2250	1465	2980

+ Method   A   =   Factored passive pressure
            B   =   Strength method
            C   =   CIRIA Report 104
            D   =   Full passive pressure (δ/ϕ' = 1.0)

structural loadings are obtained by the method recommended in CIRIA Report 104 (Case C) and the largest by Case D. In this context Bolton, Powrie and Symons (1990) have shown that much smaller rigid body movements of a stiff rigidly propped wall are likely for the higher structural loading of Case D than for Case B.

Table 2 gives the corresponding structural loadings from finite element analyses reported by Potts and Fourie

(1984, 1985) and Fourie and Potts (1989) for walls having the same geometry and soil properties as in Fig 14. These analyses, which modelled the soil behaviour as linear elastic perfectly plastic, included a range of values of initial earth pressure coefficient.

The effect of wall stiffness was also examined for the case of a wall rigidly propped at the top and Table 2 includes their results for a 1 metre thick diaphragm or

TABLE 2

Structural loadings from finite element analyses
(After Potts and Fourie 1984, 1985, and Fourie and Potts 1989)

Wall Type	Propped Diaphragm Wall Dry Soil (Fig 14a)				Propped Sheet Pile Wall Dry Soil (Fig 14a)	
Initial K_o +	2.0	1.5	1.0	0.5	2.0	0.5
Maximum Bending Moment kNm/m	4400	3085	1820	1220	1160	816
Prop Force kN/m	1013	754	400	237	317	245

Wall Type	Unpropped diaphragm Wall - Dry Soil (Fig 14b)			
Initial K_o +	2.0	1.5	1.0	0.5
Maximum Bending Moment kNm/m	1620	1330	1130	1000

+ Assumes no effect of wall installation on K_o

secant pile wall ($EI=2.3 \times 10^6$ kNm^2) and a more flexible sheet pile wall ($EI=7.8 \times 10^4$ kNm^2). It should be noted that the factors of safety (Fig 14), and maximum bending moments from limit equilibrium calculations for Case C (Table 1), differ slightly from the values quoted by Potts and Fourie (1984, 1985), and Fourie and Potts (1989), due to marginal differences in the active and passive earth pressure coefficients employed in the calculations.

For the propped wall, comparing the results in Table 2 with the values from limit equilibrium calculations in Table 1 shows that, for a wall of the sheet pile type, the maximum bending moments are substantially lower than the values given by the method recommended in CIRIA 104 (Case C), while the prop forces are greater only for an initial earth pressure coefficient of 2.0. For a typical diaphragm wall, the predicted bending moments and prop forces from the finite element analyses are greater than the values from the CIRIA 104 method for initial earth pressure coefficients of 1.0 and above. Moreover, the predicted values with an initial earth pressure coefficient of 2, exceed those obtained for Case D with full upper bound passive pressures corresponding to δ/ϕ' of unity acting over the full depth in front of the wall. Fourie (1984) considers the effect of compressibility of the propping system and shows that the inclusion of a prop stiffness of 2.9×10^4 kN/m serves to reduce the prop force by about 13 per cent compared with the value given in Table 2 for a diaphragm wall in soil with a K_0 of 2. This prop stiffness is the same as that used by Potts and Burland (1983) in the analyses of the Bell Common tunnel to represent the combined effects of the concrete roof beams and compressible packing.

For the unpropped wall comparison of the results shows that for a typical diaphragm wall the predicted maximum bending moments from the finite element analyses are less than obtained from limit equilibrium calculations using the method given in CIRIA Report 104 (Case C), except for initial earth pressure coefficients K_0 approaching 2. The maximum bending moments, calculated assuming passive pressure acting over the full depth in front of the wall, substantially exceed the finite element predictions. This is mainly due to the markedly non-linear earth pressure distributions predicted from the finite element analyses, with lateral stresses close to active acting over most of the retained height (Potts and Fourie 1989).

The geometry, soil properties and factors of safety for walls designed for a condition of linear seepage between water tables at ground level on both sides of the structure are shown in Fig 15. The required depths of embedment were determined in accordance with the recommendations of CIRIA Report 104 assuming 'worst credible parameters'. The structural loadings from limit equilibrium calculations for Cases A to D are given in Table 3. Two sets of values are included for Case D corresponding to the following assumptions concerning the angle of wall friction on the excavated side:-

(i) $\delta/\phi' = 0.5$ as recommended in CIRIA Report 104 for calculating the required depth of embedment.

(ii) $\delta/\phi' = 1.0$ representing an upper bound assessment of passive pressures.

Table 4 gives the corresponding structural loadings from finite element analyses using the same soil model and stiffness properties as employed for embedded walls in dry soil (Potts and Fourie 1984). For the propped case these analyses considered initial earth pressure coefficients K_0 of 1.0 and 2.0 together with wall stiffnesses representative of a 1m thick diaphragm wall ($EI= 2.3 \times 10^6$ kNm^2) and of a much stiffer T shaped diaphragm wall ($EI = 4.0 \times 10^7$ kNm^2). For the unpropped case the maximum bending moments are calculated for a diaphragm wall in ground with an initial earth pressure coefficient of 2.0.

For the propped walls a comparison of the structural loadings given in Tables 3 and 4 indicates that for a typical diaphragm wall the method recommended in CIRIA Report 104 (Case C) gives good agreement with the structural loadings predicted for an initial earth pressure coefficient of unity; but significantly underestimates the structural loadings predicted for an initial earth pressure coefficient of 2.0. The upper bound assessment of structural loading from limit equilibrium calculations (Method D with δ equal to ϕ') gives a prop force close to the value predicted for a diaphragm wall with a K_0 of 2.0 and with the predicted bending moments and prop forces for the stiffer wall in ground with a K_0 of 1.0. However this method underestimates the predicted structural loading effects for the stiffer wall with a K_0 value of 2.0 where the numerical analysis indicates a non linear pressure distribution on the retained side with soil pressures corresponding to an earth pressure coefficient of greater than unity acting over the retained height.

For the unpropped wall, the limit equilibrium method recommended in CIRIA Report 104 (Case C) shows reasonable agreement with the predicted maximum bending moment in soil with a K_0 of 2.0. For this case the numerical analyses predict fully active pressures acting over the retained height with full passive pressures confined to the upper 5-7 metres of ground on the excavated side.

As would be expected, comparisons of the results from numerical analyses and limit equilibrium calculations for a condition of linear seepage show similar trends to those for dry conditions. However a substantial component of the structural loading under conditions of linear seepage results from differential water pressure. Thus, the effect on the results of the assumptions made concerning earth pressures in the limit equilibrium calculations and the initial in situ stress condition in the numerical analyses, are less pronounced.

The predicted structural loadings from numerical analyses are sensitive to the relative soil - wall stiffness as illustrated by the results of an additional finite element analysis for the propped diaphragm wall in ground with a K_0 value of 2.0 in which the soil stiffness was reduced by a factor of three. The predicted maximum bending moment increased by 43 per cent and the prop force by 27 per cent compared with the values in Table 4. Fourie (1984) also reports that reducing the Young's modulus of the wall by a factor of two to represent cracked concrete, had the effect of reducing the maximum bending

(a) Propped wall

0m

$\gamma = 20\text{kN/m}^3$ $F_p = 1.4$

$c' = 0$ $F_{np} = 2.5$

$\phi' = 25°$ $F_s = 1.2$

$\dfrac{\delta}{\phi'} = \dfrac{2}{3}$ $F_r = 1.5$ $F_d = 1.3$

10m

$\dfrac{\delta}{\phi'} = \dfrac{1}{2}$

22.4m

(b) Unpropped wall

0m

5m

$\gamma = 20\text{kN/m}^3$ $F_p = 1.4$

$c' = 0$ $F_{np} = 1.9$

$\phi' = 25°$ $F_s = 1.2$

$\dfrac{\delta}{\phi'} = \dfrac{2}{3}$ $F_r = 1.5$ $F_d = 1.3$

$\dfrac{\delta}{\phi'} = \dfrac{1}{2}$

18m

20.6 m
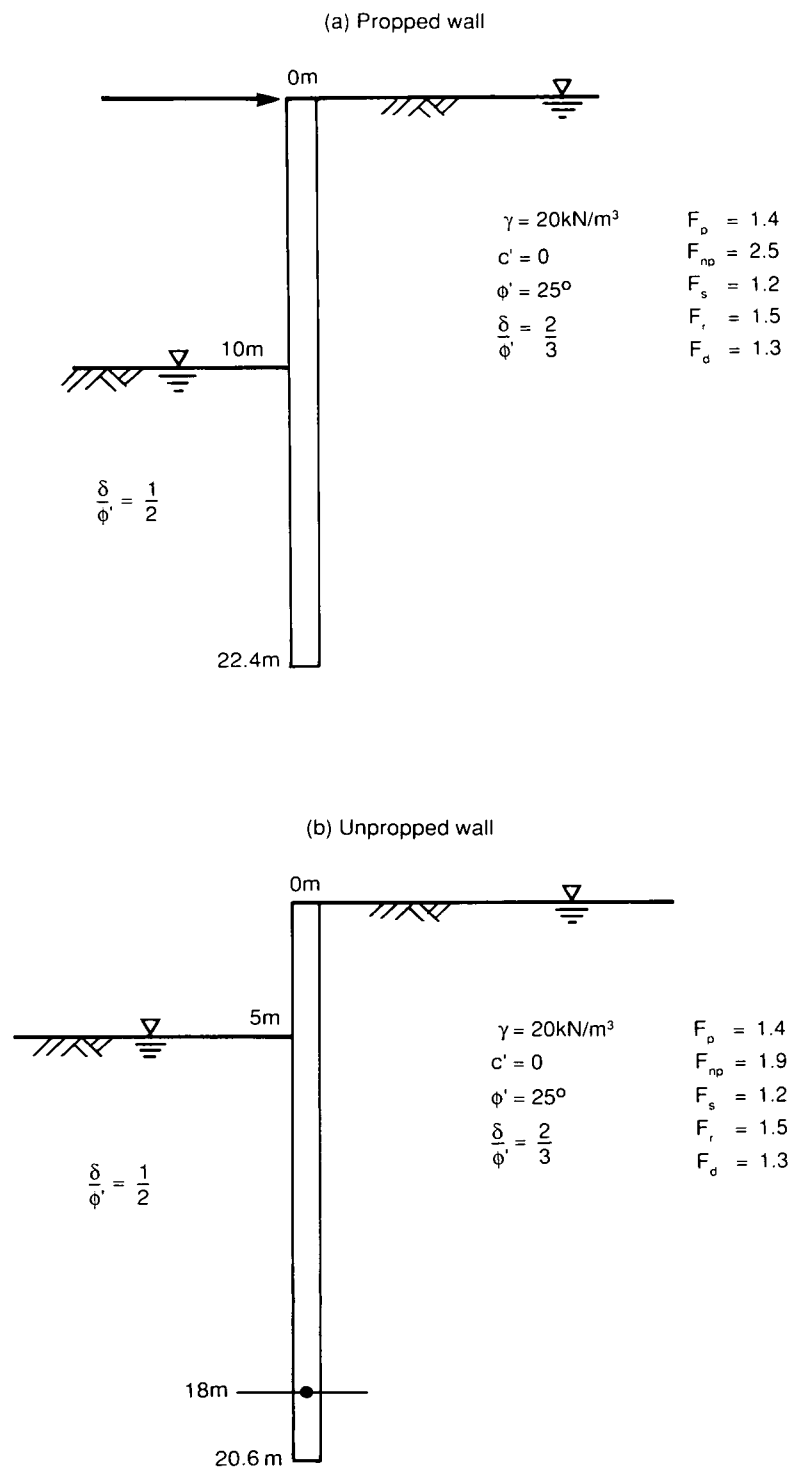
Fig.15 Wall geometry, assumed soil properties and factors of safety – linear seepage

TABLE 3

Structural loadings from limit equilibrium calculations

Method +	Propped Wall - Linear Seepage (Fig 15a)				
	A	B	C	D*	D**
K Retained Side	0.346	0.408	0.346	0.545	0.652
Maximum Bending Moment kNm/m	3360	3590	2570	4110	4520
Prop Force kN/m	525	565	440	645	710

Unpropped Wall - Linear Seepage (Fig 15b)

Method +	A	B	C	D*	D**
K Retained Side	0.346	0.411	0.346	0.600	0.737
Maximum Bending Moment kNm/m	1720	1820	1270	2130	2360

+ METHOD   A = Factored Passive
               B = Strength Method
               C = CIRIA Report 104
               D = Full Passive Pressure
                     * $\delta/\varnothing' = 0.5$
                     ** $\delta/\varnothing' = 1.0$

TABLE 4

Structural loadings from finite element analyses

Wall Type	Propped Diaphragm Wall Linear Seepage (Fig 15a)		Propped T Wall Linear Seepage (Fig 15a)	
Initial K_0 +	2.0	1.0	2.0	1.0
Maximum Bending Moment kNm/m	3200	2150	6450	4400
Prop Force kN/m	709	447	1062	713

Wall Type	Unpropped Diaphragm Wall Linear Seepage (Fig 15b)
Initial K_0	2.0
Maximum Bending Moment kNm/m	1020

+ Assumes no effect of wall installation

moments and prop forces by about 20 per cent for a diaphragm wall in dry soil with a K_0 of 2.0 (Table 2).

In the context of the results for high initial earth pressure coefficients, it should be noted that the finite element analyses did not model the effect of wall installation on the stress state in the ground. The results may therefore be somewhat pessimistic, as illustrated by Higgins et al (1989) in the re-analyses of Bell Common tunnel. The results nevertheless serve to demonstrate that wall stiffness and initial earth pressure coefficient, which are neglected in simple limit equilibrium calculations, can have a profound effect on the predicted structural loading in service.

It is suggested that in the absence of numerical analyses, the simple linear stress distributions corresponding to Case D (Fig 9) might represent a conservative upper bound approach for assessing the maximum structural loading effects in service on stiff walls propped at the top in overconsolidated clay. These assumptions are only likely to be valid where the initial earth pressure coefficient is high ($K_0 > 1.5$) and the method of construction is such as to minimise ground movements and stress charges during wall installation. For unpropped cantilever walls the results suggest that use of the Strength Method (Case B), or the CIRIA Report 104 method (Case C) with a small enhancement factor, may provide a conservative upper bound assessment of the maximum bending moments in service for stiff walls in overconsolidated clay.

For more flexible walls of the sheet pile type the results of numerical analyses (Table 2) and field measurements (Symons et al 1987) suggest that limit equilibrium methods are likely to overestimate the structural loading in service, particularly for unpropped walls in soil with a low initial earth pressure coefficient at rest. However the effect of pile driving may well be to increase the in situ lateral stresses above their initial value in the immediate vicinity of the wall.

There is at present little guidance available for assessing the required depth of embedment or structural loading on an embedded wall propped at or close to excavation level, and this support condition is excluded from CIRIA Report 104. However, for stiff walls in overconsolidated clays the soil loading effects are likely to be greater due to the enhanced stability provided by a support at this level (Figs 7 and 8). Powrie and Li (1991) have conducted finite element analyses of centrifuge model tests on walls propped at excavation level with different depths of wall embedment. They obtained reasonably good agreement between the maximum bending moments predicted from their finite element analyses and from limit equilibrium calculations using an earth pressure coefficient of unity; this closely approximated to the initial in-situ value prior to simulation of excavation in the centrifuge tests. Symons and Carder (1991) showed that the distributions of total lateral pressure measured on both sides of a wall propped below carriageway level were in reasonable agreement with those obtained from limit equilibrium calculations. In these calculations full passive pressures were assumed on the excavated side and a

'mobilised strength' approach was used on the retained side, starting from an initial stress distribution similar to that measured further from the wall.

6. SUMMARY AND CONCLUSIONS

This Report reviews the approaches currently available for the design of embedded retaining walls. Particular attention is focused on relatively stiff walls of the diaphragm or bored pile type in heavily overconsolidated sedimentary clays on which many urban areas in southern England are founded.

(i) The influence of previous stress history on the current stress state in overconsolidated clays is outlined and the effects of sample size, stress path and stress level on the measured strength are considered in the context of the selection of representative design values. The strain dependency of soil stiffness properties is also referred to in relation to the predictions of ground behaviour from numerical analyses.

(ii) Limit equilibrium methods for determining the depth of wall embedment are described and generalised relations presented which illustrate the influence of support conditions on wall stability.

(iii) The effect of the mode of wall displacement on the movements necessary to achieve limiting active and passive soil thrusts is reviewed and attention drawn to the effect of excavation on the equilibrium stress distributions in front of embedded retaining walls in overconsolidated clays.

(iv) Simple limit equilibrium methods of calculating the structural loading on retaining walls are outlined and their validity assessed using data from numerical analyses which take account of the in situ stress state in the ground and the relative stiffness of the structure. An upper bound approach is suggested for assessing the maximum structural loading in service.

(v) The bending moments and prop forces calculated using simple limit equilibrium methods for both an unpropped wall and a wall propped at the top are compared with finite element predictions. The results indicate that for stiff walls in overconsolidated clay the suggested upper bound approach is likely to give a conservative assessment for structures rigidly propped at the top, where the initial in situ earth pressure coefficient is high and the effect of wall installation is small. For stiff unpropped walls, a mobilised strength approach or the method recommended in CIRIA Report 104 with a small enhancement factor, is likely to provide a conservative assessment of the maximum bending moment in service.

(vi) Field measurements of the behaviour of full scale structures, both during construction and in service provide an essential component in the evaluation and development of improved design methods.

7. ACKNOWLEDGEMENTS

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APPENDIX A

Derivation of relation between depth of failure zone (z) below the final ground level and the thickness (h) of the layer of overburden removed

initial ground level

$\Delta\sigma_v' = h\gamma' = h\,(\gamma - \gamma_w)$

final ground level

d
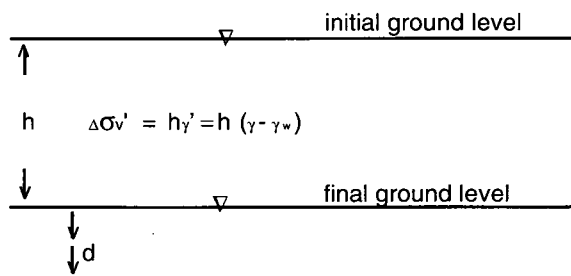
Subscript	$_i$ = initial
	$_f$ = final
	$_v$ = vertical
	$_h$ = horizontal

$$\sigma_{vi'} = (h+d)\,\gamma' \qquad (1)$$

$$\sigma_{hi'} = K_0\,\sigma_{vi'} = K_0(h+d)\,\gamma' \qquad (2)$$

$$\sigma_{vf'} = \sigma_{vi'} \cdot \Delta\sigma_v = (h+d)\,\gamma' - h\gamma' = d\gamma' \qquad (3)$$

$$\sigma_{hf'} = \sigma_{hi'} \cdot \Delta\sigma_{h'}$$

Assuming elastic behaviour:-

$$\Delta\sigma_{h'} = \frac{\mu\Delta\sigma'_v}{1-\mu} \qquad (4b)$$

$$\sigma_{hf'} = K_0\gamma'(h+d) - \frac{\mu}{1-\mu}h\gamma' \qquad (4c)$$

$$K_{final} = \frac{\sigma_{hf'}}{\sigma_{vf'}} = \frac{k_0\,(h+d)}{d} - \frac{\mu}{1-\mu}\frac{h}{d} \qquad (5)$$

For: $K_0 = K_{0nc}$

$K_{final} = K_0$ overconsolidated

$$\text{Overconsolidated ratio} = \frac{(h+d)\gamma'}{d\gamma'} = OCR$$

$$K_0 \text{ overconsolidated} = K_{0nc}\,OCR \cdot \frac{\mu}{1-\mu}\,(OCR-1) \qquad (6)$$

Equation (6) is identical with Wroth (1975).

For failure in extension:-

$$\frac{6\,Sin\phi'}{3+Sin\phi'} = \frac{3\,(\sigma_{hf'} - \sigma_{vf'})}{2\sigma_{hf'} + \sigma_{vf'}} \qquad (7)$$

$$\frac{2\,sin\phi'}{3+Sin\phi'} = \frac{\sigma_{hf'} - \sigma_{vf'}}{2\sigma_{hf'} + \sigma_{vf'}} \qquad (7a)$$

For failure extending over a depth d = z:-

Replacing d by z in equations (3) and (4c) and substituting these equations into equation (7a) leads to:

$$\frac{z}{h} = \frac{(1-sin\phi')\,[K_0\,(1-\mu)-\mu]}{(1-\mu)\,[(K_0+1)\,sin\phi' -(K_0-1)]}$$

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TRL has published the following other reports on this area of research:

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RR273 Long term performance of a propped retaining wall embedded in stiff clay, D R Carder and I F Symons, Price Code B

RR288 Behaviour of an embedded retaining wall on the A6 Chapel-en-le-Frith by-pass, P Darley, I F Symons and D R Carder, Price Code C

RR313 Long term performance of an anchored diaphragm wall embedded in stiff clay, I G Carswell, D R Carder and I F Symons, Price Code B

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