

Embedded retaining walls: theory, practice and understanding

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Embedded retaining walls: theory, practice and understanding

Les murs de soutènements encastrés: théorie, pratique et interprétation

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ABSTRACT: Embedded retaining walls commonly comprise steel sheet piling or concrete walls, built as diaphragm walls in slurry trenches or using piling methods. Since the early 20th century, sheet piling has been in common use, particularly for waterfront structures and temporary works. More recently, concrete walls have been used extensively for construction of basements and underground infrastructure in urban areas. The performance and design of embedded walls has been debated extensively by Terzaghi, Brinch Hansen, Rowe, Tschebotarioff and many more recent authors, whilst codes of practice aim to specify design procedures. Although understanding has increased in some respects, controversy remains, notably in relation to distribution of earth pressures on walls subject to flexure, adoption of working or collapse states in design, and application of safety factors. This paper aims to summarise and extend this debate, and to suggest future developments which might help to clarify understanding and design procedures

RÉSUMÉ: Les murs de soutènements encastrés, murs en palplanches ou béton, comprennent la réalisation de panneaux de parois moulées ou de pieux. Au vingtième siècle, les murs en palplanches furent régulièrement employés en front de mer et pour les travaux temporaires. Récemment, les murs en béton furent largement utilisés lors de la construction de sous-sols et d'infrastructures enterrées en zone urbaine. La performance et le dimensionnement des murs de soutènement furent longuement discutés par différents auteurs. Les normes ont pour but de spécifier les procédures de dimensionnement. Malgré l'accroissement de la compréhension, le sujet reste fortement controversé particulièrement sur la répartition de la poussée des terres sur des murs soumis à la flexion, sur la définition de l'état ultime et l'état de service et sur l'application des facteurs de sécurité. Cette publication vise à résumer et élargir le débat, ainsi qu'à suggérer une démarche qui pourrait clarifier les procédures de dimensionnement.

1 HISTORICAL BACKGROUND

Embedded retaining walls are walls that penetrate into the ground and rely to a significant extent or even completely on the passive resistance of the ground for their support. In the first half of the twentieth century, embedded walls were generally formed of either soldier piles or steel sheet piles, the latter being the subject of most of the debate and development of design methods. In the second half of the century, concrete walls formed either in slurry trenches or by contiguous or intersecting (secant) piles became increasingly common, often retaining natural clay rather than coarse grained soils.

Classical methods of retaining wall analysis can be traced back to the work of Coulomb (1776) and Rankine (1857). Coulomb carried out upper bound calculations assuming a planar wedge failure mechanism from which he derived the limiting (active) *force* on a retaining wall, as a function of depth below the retained soil surface. This form of calculation does not indicate a unique stress distribution. Rankine carried out lower bound calculations based on the assumption that the stress field behind the wall was in a uniform state of plastic equilibrium; from this he derived limiting earth *pressures* which, due to his assumptions, increased linearly with depth in uniform materials. For the simple case of a frictionless wall in uniform soil, the two solutions coincide provided it is assumed that the active force calculated using Coulomb's approach results from a lateral earth pressure that increases linearly with depth.

Rankine's calculation gives lateral earth pressure coefficients, that is ratios of horizontal to (notional) vertical effective stress, at any depth. These form the basis of most limit equilibrium analyses of embedded retaining walls. Although Coulomb's original approach was based on a consideration of the overall equilibrium of an entire wall, it can be used to calculate earth pressure coefficients if it is assumed that the total lateral thrust results from a lateral earth pressure distribution that increases linearly with depth.

Later workers have used more complex calculations to determine earth pressure coefficients, based on either upper bound (following Coulomb) or lower bound (following Rankine) approaches, to refine the results and to extend them to include wall friction, sloping ground surfaces, and non-vertical walls. For example, Sokolovski (1960) used a lower bound method, while Caquot & Kerisel (1948), and most other workers, used upper bounds. The degree of refinement is now such that the practical difference between the bounds is small, at least in cases where it can be assumed that earth pressures increase linearly with depth.

It is considered that all the authors noted throughout this paper would agree that the active and passive forces calculated in this way are limits that cannot be infringed. However, there has been a considerable debate about how the earth pressures giving rise to these forces may be distributed, linearly or otherwise, both at collapse and under working conditions. Earth pressure redistribution, and the distinction between design approaches based on lateral stress distributions at collapse (or an ultimate limit state) and under working conditions (or a serviceability limit state), are two of the key issues addressed in this Paper.

1.1 *Idealised stress distributions at collapse*

Unpropped embedded walls rely entirely for their stability on an adequate depth of embedment. They are not supported in any other way, and will tend to fail by rotation about a pivot point near the toe. An idealised stress distribution at failure, based on limiting active or passive stresses in zones of soil where the wall is moving away from or into the soil, is shown in Figure 1a. An embedded wall propped at the crest will tend to fail by rigid-body rotation about the prop, with the idealised effective stress distribution at failure shown in Figure 1b.

With the stress distributions shown in Figure 1 and limiting active and passive lateral earth pressures, the equations of moment and horizontal force equilibrium can be used to determine

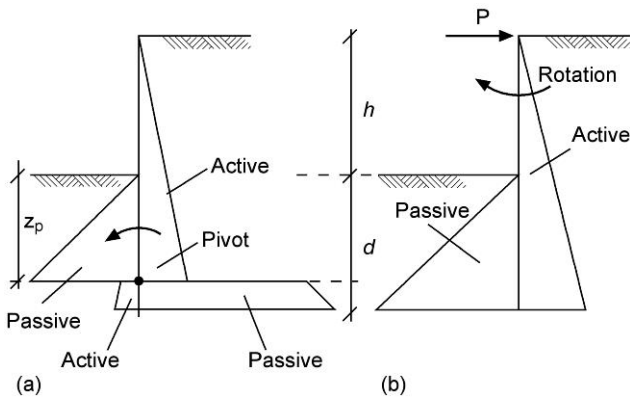


Figure 1. Idealised linear effective stress distributions.

the two unknowns in each case. In other words, these stress distributions are statically determinate.

The stress distributions shown in Figure 1 are highly idealised. In general, a prop or anchor plate will be of finite depth, and may well result in a local increase in lateral stress in the soil at that level. Nonetheless, it has been shown with reference to both finite element studies (e.g. Potts & Fourie 1984) and analyses of real walls (Powrie 1996) that limit equilibrium calculations using the full (unfactored) soil strength and the stress distributions shown in Figure 1 can give a reasonable indication of the embedment depth at the onset of large wall movements.

Procedures for the design of anchored walls, based on the idealised stress distribution at collapse (Figure 1b), were established by authors such as Blum (1930), summarised by Terzaghi (1943). At that time these were generally sheet pile walls. In Blum's method, the earth pressures were assumed to increase linearly with depth, as shown in Figure 2a, with a factor of safety applied as a reduction to the linear passive pressure (F_p , as defined below).

Krey's (1936) approach was similar to that of Blum (1930), except that the passive resisting force was assumed to be distributed as shown in Figure 3. This places the point of action of the passive force slightly higher and so gives a reduced tie force and bending moment. This presages Rowe's work (1952, 1955) on sheet pile walls, in which an increase in the height of the centre of passive pressure was attributed to the effects of wall bending. Wall bending is just one possible factor causing a redistribution of the lateral stresses away from the linear-with-depth assumptions that stem from the simple application of classical earth pressure theory.

1.2 Factors of safety for idealised stress distributions at collapse

The stress field distributions shown in Figure 1 correspond to limiting conditions, when the wall is on the verge of rotational failure. The stresses behind the wall are at their minimum possible values (the active limit), while the stresses in front of the wall are at their maximum possible values (the passive limit). A real wall must be sufficiently remote from collapse not to deform excessively under working conditions and must also have margins of safety to guard against unexpected conditions. It is therefore necessary to increase the depth of embedment beyond that required merely to prevent collapse. This is traditionally achieved by applying a *factor of safety* F to one or more of the parameters in the collapse calculation, to estimate the depth of embedment required for serviceability and safety. Some alternative forms of the factor F are shown in Figure 2.

Although current practice in the UK is generally to apply the factor of safety (F_s) to the soil strength, this has not always been the case. Certain of the traditional methods of applying a factor

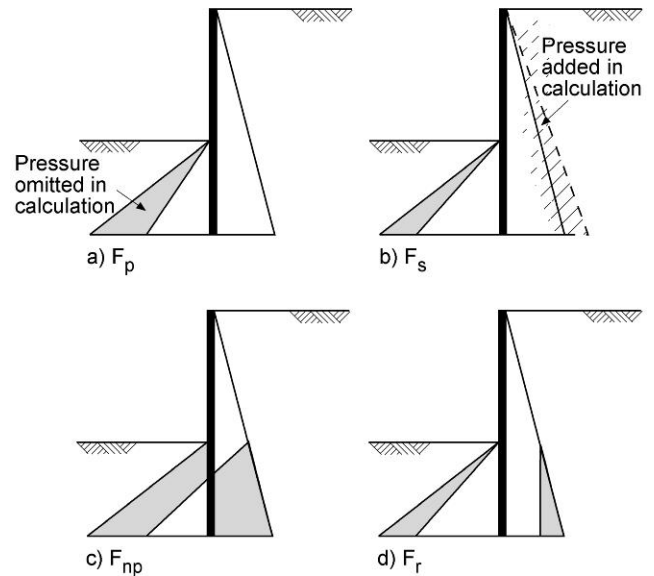


Figure 2. Definitions of factors of safety, after CIRIA 104.

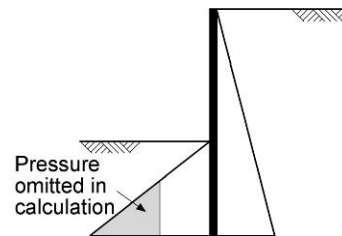


Figure 3. Krey's assumed distribution of passive pressure.

of safety are now known to be potentially unsafe, or inappropriate in some circumstances. This was identified and discussed by Burland, Potts & Walsh (1981), some of whose findings are summarised below.

In many soils, particularly sands, the *in situ* lateral earth pressure coefficient $K_o = \sigma'_h / \sigma'_v$ is close to the active limit. In these conditions, the stresses in the soil behind the wall fall to their active values after only a small movement of the wall (Terzaghi 1943). In front of the wall, rather larger movements than are acceptable under working conditions are required for the stresses to rise to the passive limit. In these cases, the wall would be expected under working conditions to be in equilibrium under the action of the active pressures in the retained soil, and lower-than-passive pressures in the soil in front of the wall. The full passive pressures are therefore reduced by a factor F_p , which should ideally be chosen with regard to the rate of increase in stress in front of the wall with wall movement, and the acceptable limit of deformation. This is the traditional procedure given in the former UK code of practice for retaining walls, CP2 (IStructE 1951) and elsewhere.

Figure 4a,b (from Burland et al 1981) compares the ratios (H/h) of the overall wall height H to the retained height h calculated using various factors of safety ($F_p=2$, $F_s=1.5$) with (H/h) at collapse ($F_s=1$), for various values of ϕ with a smooth wall ($\delta=0$) and a water table on both sides of the wall at the level of the excavated soil surface (i.e. there is no seepage around the wall, and the pore water pressures below the water table are hydrostatic). Figure 4 illustrates that, for an effective stress analysis, $F_p=2$ is unduly conservative at values of ϕ less than about 27° .

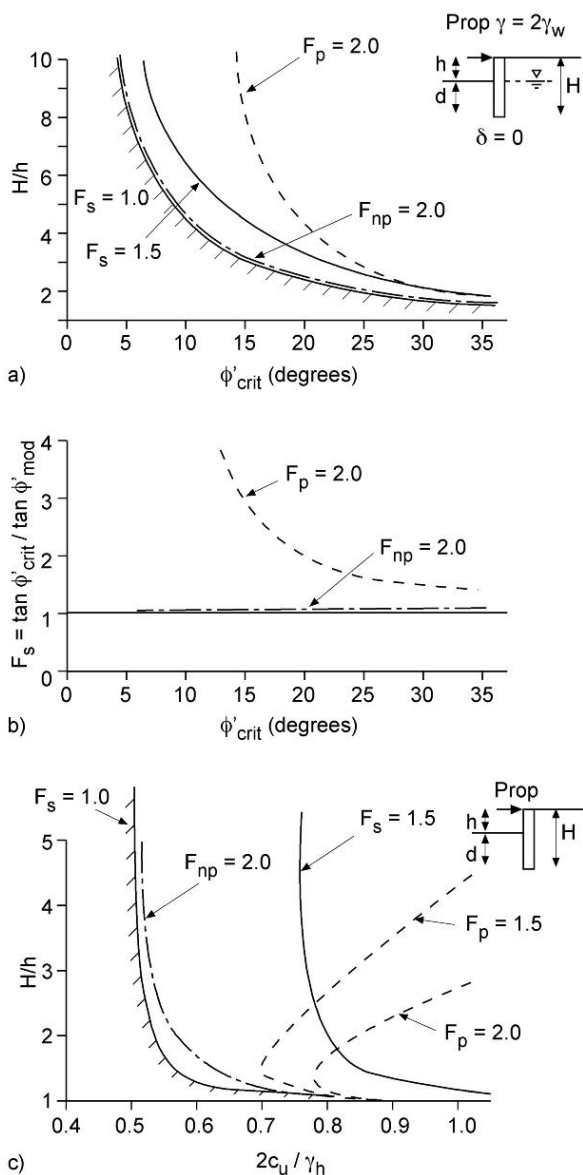


Figure 4. Factor of safety calculated using various methods against factor of safety on soil strength (after Burland *et al* 1981).

For a total stress analysis, Figure 4c shows the ratio of the overall wall length H to the retained height h (H/h) as a function of the non-dimensionalised undrained shear strength $2c_u/\gamma h$, calculated using $F_s=1.5$, $F_s=1$ (i.e. at limiting equilibrium), $F_p=1.5$ and $F_p=2$. By considering the variation in F_p with H/h at constant c_u ($2c_u/\gamma h=0.9$, for example), it may be seen that F_p is reduced as H/h is increased. Given that an increase in H/h corresponds to an increase in the wall length H at constant retained height h (i.e. an increase in the depth of embedment d), this result is clearly unrealistic.

For walls propped at the crest, the "net pressure method" is described in the British Steel Sheet Piling Handbook (1988), using the factor F_{np} as shown in Figure 2c. A net pressure diagram based on fully-active and fully-passive pressures is plotted, and the depth of embedment is chosen such that the moment about the prop of the net pressure in front of the wall is equal to a factor F_{np} (normally 2.0) times the moment of the net pressure behind the wall.

This method is fundamentally unsound, and potentially dangerous. Figure 4 shows that an apparently satisfactory numerical value of $F_{np}=2$ may correspond to a factor on soil strength (F_s)

which is little greater than unity (Burland *et al* 1981). Padfield & Mair (CIRIA Report 104 1984 on the design of retaining walls embedded in stiff clays) recommend against its use. Williams & Waite (1993) suggest that it might be acceptable in an effective stress analysis, provided that the strength parameters are selected conservatively. Their reluctance to recommend against using the method is probably linked to their observation that it has been used successfully in sheet pile cofferdam design for more than 50 years. This may be the case, but Figure 4 shows that this success must have relied very heavily on the selection of conservative strength parameters: $F_{np}=2$ corresponds to a factor of safety F_s (on soil strength) of rather less than 1.1 over the range $10^\circ \leq \phi' \leq 35^\circ$.

Burland *et al* (1981) proposed a further "revised" factor of safety, F_r , as illustrated in Figure 2d. Their justification for this factor was that it gave results consistent with F_s , so it appears to give no real advantage in design.

For the following reasons, it is considered that the application of the factor of safety to the soil strength represents the most appropriate approach in the design of an embedded retaining wall.

1. Design calculations involve many uncertainties, including loads, effects of loads, and materials properties. Simpson (2000) points out that factors should be applied to the uncertainties themselves, so that their effects from derived quantities will result from the calculation. On the other hand, to reduce the risk of mistakes, the number of factors should be kept to a minimum. As soil strength is often the principal uncertainty in geotechnical design, it follows that this is where the factor of safety should be applied.
2. Through the assumption of simple idealised deformation mechanisms, the mobilised soil strength can be related to wall movement - at least for stiff walls that are either unpropped or propped at the top (Bolton *et al* 1990a,b). Thus the application of the factor of safety to soil strength can in principle be quite clearly correlated with an acceptable level of deformation under working conditions.
3. The application of a factor of safety in some other way (e.g. on passive pressure coefficient, net pressure or depth of embedment) can give unexpected results, as shown by Burland *et al* (1981) and summarised above. This can be illustrated further by the extreme example of the use of a factor of safety on passive pressure coefficient in the case of a wall of zero embedment depth.

Increasing the depth of embedment of a flexible wall may result in a reversal of bending moment near the base and a consequent reduction of the major bending moment at higher level. In an attempt to take advantage of this, some authors (e.g. Williams & Waite 1993) describe the use of a "fixed earth support" calculation for a propped embedded wall, with the idealised effective stress distribution and bending moments shown in Figure 5. Without a further assumption, the idealised stress distribution shown in Figure 5 is statically indeterminate. Williams & Waite (1993) adopt a conventional proposal that the prop force and the depth of embedment may be calculated by assuming that the point of contraflexure, at which the bending moment is zero, occurs at the level where the net pressure acting on the wall is zero. If this is done, the calculated embedment depth will be somewhat greater than that in a free earth support analysis, so it is not necessary to apply a factor of safety to determine the design embedment depth. The calculations could alternatively be carried out by seeking an embedment depth that will limit the maximum bending moment to a required value.

1.3 Investigations of stress states under working and collapse conditions

During the 1940's and 50's, considerable research effort was focussed on the effect of wall flexibility on reducing bending moments and prop loads to below those calculated on the basis of fully active lateral stresses in the retained soil (Terzaghi 1943 1954, Tschebotarioff 1951, Rowe 1952, 1955, 1956). This work

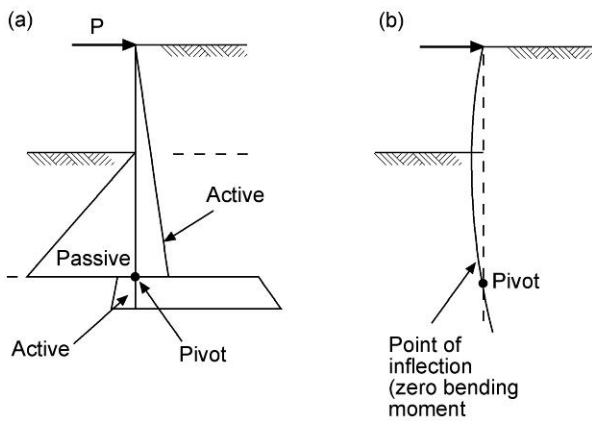


Figure 5. Fixed earth support conditions for a flexible embedded wall propped at the crest: (a) idealised stresses; (b) deformed shape.

represented a change in emphasis, as it was directed at identifying lateral stresses under working conditions for use directly in design, in contrast to a design based on the stress state at ultimate failure.

In working conditions, earth pressures can be caused to redistribute in various different ways by relatively small movements of props or anchors, or by variations in the stiffness of the ground.

1.4 Comments of Terzaghi and Brinch Hansen

Terzaghi (1943) noted that the flexure of a retaining wall could lead to redistribution of the active earth pressure, as shown in Figure 6. This has the effect of increasing the load on the ties and reducing the bending moment in the wall. He concluded that an adequate theory for the evaluation of this effect was not available, but that in “a mass of clean sand” a reduction in bending moment of 50% could be expected as a result of this phenomenon. Terzaghi recognised this as a form of arching and argued that “since arching is maintained solely by shearing stresses in the soil, it is no less permanent than any other state of stress in the soil, which depends on the existence of shearing stresses ... On the other hand, every external influence which causes a supplementary settlement of a footing or an additional outward movement of a retaining wall under unchanged static forces must be expected to reduce the intensity of existing arching effects. Vibrations are the most important influence of this sort”. It might be noted that vibration can equally cause an increase in active pressures in the absence of arching.

Terzaghi (1954) was a seminal paper on Anchored Bulkheads. This continued the debate about the distribution of active earth pressures, taking account of early work by Rowe, noted below, and concluded that “there is no longer any justification for assuming fixed earth support without considering flexibility of the sheet piles”. The concepts of redistribution of active earth pressure were again noted, and described as the “actual distribution”, but in the light of Rowe’s finding that very small yield of the anchor could lead to a return to linearly distributed earth pressures, Terzaghi concluded that “it does not seem justified to rely on the benefits to be derived from a difference between the real pressure distribution and the distribution computed on the basis of the Coulomb theory”.

In the discussion to Terzaghi’s paper, Brinch Hansen raised another important idea. “The problem of the design of flexible anchored bulkheads is evidently one in which proportionality between total load and maximum stress does not exist, and in such a case the usual concept of allowable stresses is not suitable. In other fields of engineering such problems are investigated by considering the state of failure; the structure is then de-

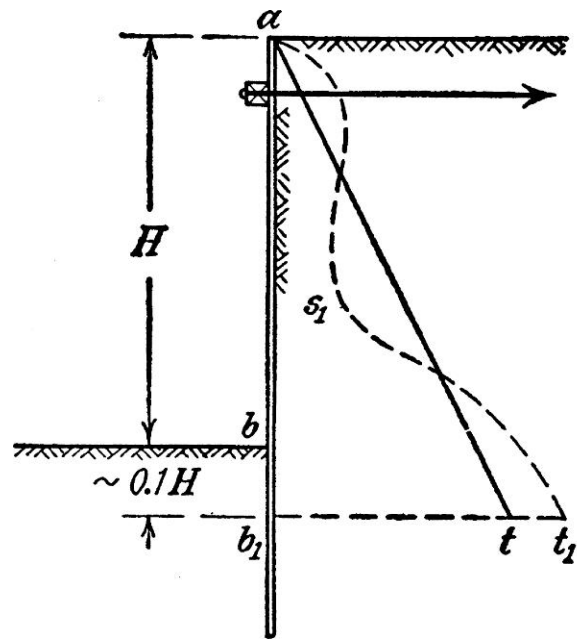


Figure 6. Terzaghi’s “assumed (unbroken line) and real (dashed line)” active pressure distributions, after Terzaghi (1943).

signed so as to possess a certain safety against failure. Similar design methods are also used for structures (such as slabs and shells) which might have been designed by the application of the theory of elasticity.” In effect, Brinch Hansen was arguing for design based on analysis of states of ultimate failure, rather than in-service states. This debate continues today, and we will return to it later in the paper.

1.5 The work of Rowe

For a wall of given overall length H and flexural rigidity EI , bending effects are most significant when the wall is propped at the crest. This was investigated by Rowe (1952) in a series of model tests on anchored sheet pile walls of various stiffness, retaining dry sand. Rowe quantified the stiffness of a wall by means of a flexibility $\rho = H^3/EI$, where H is the overall height of the wall and EI is its bending stiffness.

In general terms, wall deformation occurs partly due to rigid body rotation (in the case of a propped wall, about the position of the prop), and partly due to bending (Figure 7). Rowe (1952) found that the lateral stress distribution in front of the wall depended on the relative importance of the bending component of wall deformation, and hence on the bending stiffness of the wall.

If the wall was stiff, so that the deflexion at the level of the excavated soil surface was of the same order as the deflexion at the toe, the stress distribution in front of the wall was approximately triangular. Measured bending moments were in agreement with those from a limit equilibrium calculation based on a fully-active triangular stress distribution behind the wall and a smaller-than-passive triangular stress distribution in front (i.e. factored by the value needed for wall equilibrium, Figure 8a).

If the wall was more flexible, so that the deflexion at dredge level was significantly greater than that at the toe, the centroid of the stress distribution in front of the wall was raised (Figure 8b). This led to smaller anchor loads and bending moments than those given by the (factored) limit equilibrium calculation, more in keeping with the ideas of Krey (1936).

Rowe (1955) presented an analysis of anchored sheet pile walls in which it was assumed that the lateral effective stress behind the wall had fallen to the active limit, and the change of lateral

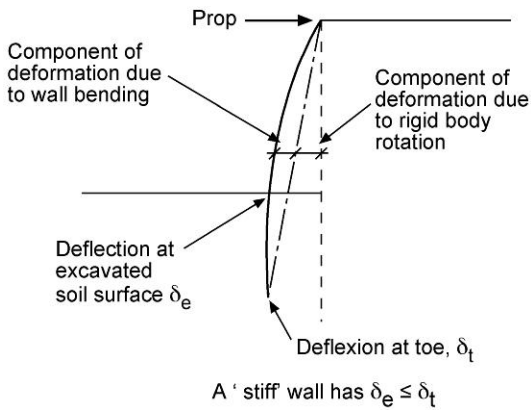


Figure 7. Components of wall displacement and definition of a "stiff" wall.

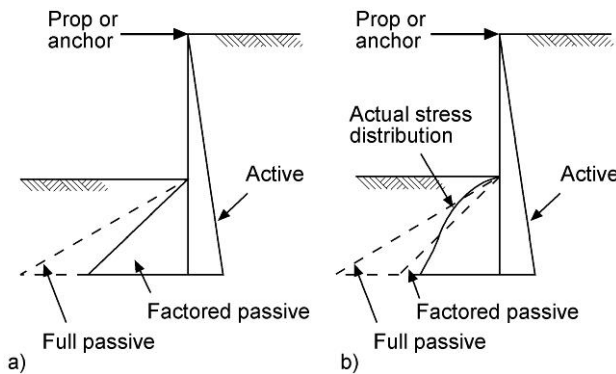


Figure 8. Stress distributions behind and in front of (a) stiff and (b) flexible embedded walls (after Rowe 1952).

effective stress from the initial state Δp_b in front of the wall at a depth x below formation level was given by the expression.

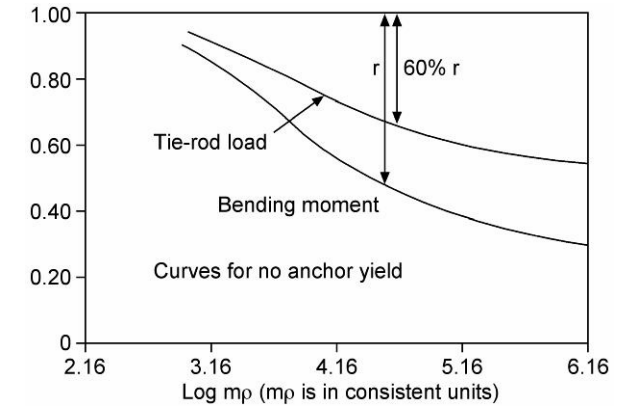
$$\Delta p_b = mxy/d \quad (1)$$

where d is the embedment of the wall, y is the deflexion and m is a soil stiffness parameter. Although Rowe describes how m may be measured or deduced, this definition of soil stiffness is unusual. Rowe's approach is equivalent to assuming a coefficient of subgrade reaction mx/d that increases linearly with depth from zero at the surface.

Rowe carried out analyses of walls of various retained height ratio $\alpha = h/H$ and depth βH to the anchor point, with and without surcharges at the retained soil surface, and with different degrees of anchor yield. He concluded that, within the ranges of these variables likely to be encountered in practice, the results of the analyses could be presented as a single moment reduction curve for design use. This curve (Figure 9) shows the bending moment as percentage of the free earth support value as a function of the logarithm of $m\rho$. In consistent units, $m\rho$ is dimensionless: however Rowe's values have to be multiplied by 144 to achieve this, because Rowe has m in lbf/ft^3 and ρ in $\text{ft}^5/\text{lbf}\cdot\text{in}^2$. (The free earth support bending moment was calculated with fully active pressures behind the wall and a passive pressure coefficient dictated by the requirements of equilibrium.)

Rowe's design chart (Figure 9) represents his analytical solution to within $\pm 10\%$ for anchored walls with retained height ratios h/H (where $H = h + d$) in the range 0.65-0.75; anchor depths βH in the range $0 \leq \beta \leq 0.2$; surcharges acting at the retained soil surface of up to $0.2\gamma H$ in magnitude; and a movement at the anchor point of up to $0.008H$. In a later paper, Rowe (1956) presents experimental data indicating prop loads in excess of the

free earth support values for anchors yielding up to $0.002H$ sup-



port

Figure 9. Moment reduction as a function of wall flexibility (after Rowe 1955). $m\rho$ is in consistent units.

ing walls with retained height ratios h/H in the range 0.6 – 0.8 and $0.1 \leq \beta \leq 0.3$. These data show a consistent increase in tie load (relative to the free earth support value) with increasing retained height ratio, and therefore provide evidence of earth pressure redistribution (arching onto a prop that is effectively rigid in comparison with the wall) as failure is approached.

Rowe normalised his results with respect to the bending moments calculated in a limit equilibrium calculation with active pressures behind the wall and lower-than-passive pressures in front. (The passive pressures were reduced by the amount needed to give equilibrium with linear pressure diagrams, as already stated). For sheet pile walls in sand, in which the pre-excavation lateral earth pressure coefficient is low, this gives a reasonable upper bound on bending moments (but not necessarily, according to Rowe (1956) on prop forces). However, the validity of the analysis based on fully active pressures in the retained soil as a "benchmark" probably depends on the initial lateral stresses being low, and may not apply in the case of an embedded wall in an overconsolidated clay.

Rowe's design chart may not be suitable for walls where the groundwater level in the retained soil is high, because retained height ratios h/H of less than 0.65 would probably be required. If the chart is used in such circumstances, it must be remembered that only the component of the bending moment due to effective stresses should be reduced. Also, the type of anchor available at that time would have been a dead man, with a rather softer response than an effectively unyielding modern prestressed anchor which gives a more or less fixed force. Pre-stressing of props or anchors could lead to higher wall bending moments, depending on the level of prestress.

1.6 Bjerrum, Frimann Clausen & Duncan (1972)

The Fifth European Conference on Soil Mechanics and Foundation Engineering, held in Madrid, addressed the theme "Structures subjected to lateral forces". A state-of-the-art report on "Earth pressures on flexible structures" was prepared by Bjerrum, Frimann Clausen & Duncan (1972). The report considered both anchored and braced excavations, and the question of arching in active earth pressures was a major theme of both the report and the discussion contributions to the conference session. For anchored walls, the report notes Rowe's view that arching effects, leading to a reduction in earth pressures between the anchor and the dredge level, could (being dependent on an unyielding anchor) be unstable and so should not be used to advantage in design. It notes, however, that "when additional anchor yield and backfill settlement do not destroy the arching between dredge and anchor level, the moments in the wall will be smaller

than those calculated by Rowe's method". For braced excavations, the report clearly anticipates that arching effects will take place, giving reduced earth pressures either towards the bottom of the excavation, or, in deep deposits of soft clay, in the ground beneath the excavation. However, the report's main emphasis for braced excavations is the effect of arching in increasing strut loads, rather than in reducing bending moments. In his verbal presentation, Bjerrum seemed to have favoured allowing the full effects of arching, but it appears that this debate has never been properly concluded.

1.7 Danish and German practice

In parallel with the developments described above, Danish practice was based on plasticity theories (i.e. conditions at collapse) of increasing sophistication, which allowed significant redistribution of active earth pressure used to advantage in reducing calculated bending moments (Bretting 1948; Brinch Hansen 1953; and, more recently, Mortensen 1995). In these designs, safety factors are applied as factors on the strengths of the soil, on both active and passive sides of the wall. An example from Mortensen (1995) is shown in Figure 10. It should be noted that, although pressures above the prop will tend to increase, soil/wall friction will be in the opposite direction to that in the passive zone in front of the wall. This will result in a very significantly reduced passive earth pressure coefficient.

The German Committee for Waterfront Structures, Harbours and Waterways (1996) recommends the use of Blum's method for sheet pile walls anchored near the top, with stress redistribution as shown in Figure 11 to account for increased lateral stresses in the vicinity of the prop or anchor. If the anchor is placed below the crest, the expected sense of wall rotation would lead to increased lateral pressures (as a result of the tendency towards passive conditions) in the soil above the prop. This is reflected in the dependence of stress redistribution on anchor depth indicated in Figure 11.

1.8 Stress states under working and collapse conditions

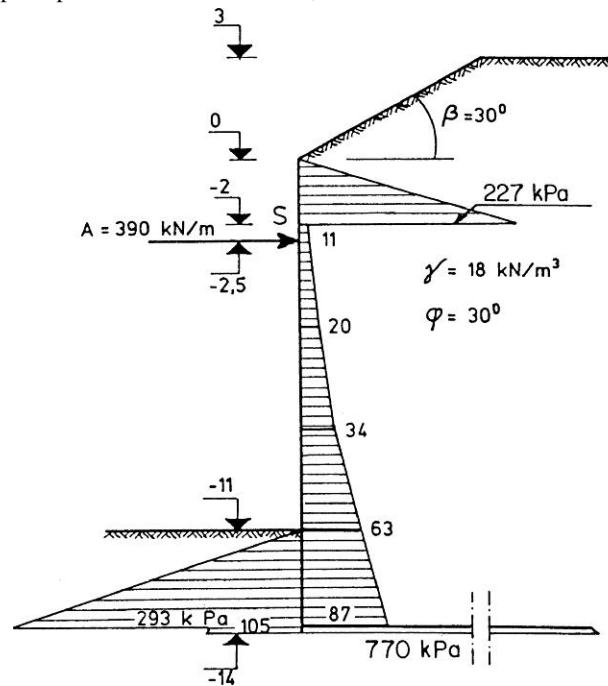
In working states, earth pressures can be caused to redistribute in various different ways by relatively small movements of props or anchors, or by variations in the stiffness of the ground. However, if states of failure are considered, involving either rigid body rotation of the wall or bending failure, the stiffness of the prop or anchor is effectively much greater than that of the failing structure. Hence, in this state, considerations of yield at the support points are not relevant and it is reasonable to assume that full arching takes place. Put another way, if the support point were to move slightly, the wall at failure would simply move a little more and arching would be re-established. Thus the Danish approach, though appearing to take a more optimistic view of arching phenomena, is internally consistent and makes the ambiguous argument about arching in the working state unimportant to design. It may be concluded, as a minimum, that the beneficial use of arching in design will not lead to ultimate failure of an embedded wall, provided there is sufficient ductility in the structure to allow moderate displacements to take place and that the overall active/passive force envelope is respected.

The choice between design for a working state (or a serviceability limit state) or an ultimate collapse limit state will be discussed further later in the paper.

2 RECENT CODES AND ADVISORY DOCUMENTS

Codes of practice, and other advisory documents which substitute for codes, are published in many countries. Their development is related to understanding of the behaviour of structures, and also depends heavily on knowledge of practical successes and failures, even where full analysis of these is not possible. The purpose of this section of the paper is to discuss some of the

principle features of these codes, and the discussion will be illus-



trated by reference to three documents drafted in the last 2 to

Figure 10. Reduction of earth pressure, after Mortensen (1995).

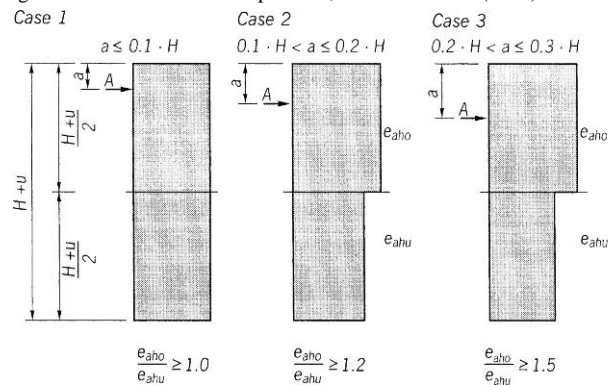
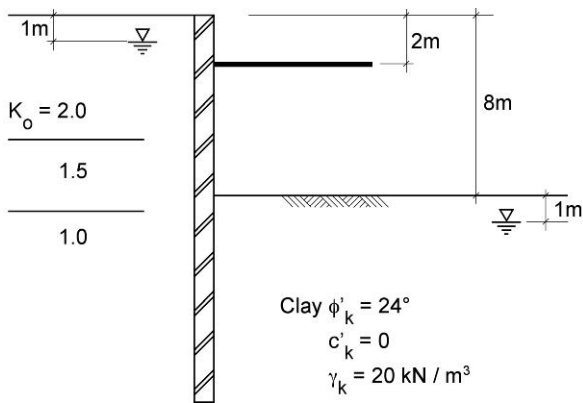


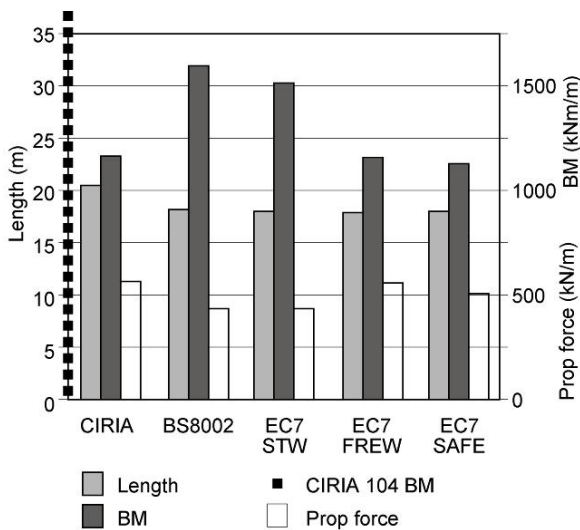
Figure 11. Redistribution of earth pressure, after EAU (1996).

3 decades which take differing approaches. These are: CIRIA 104 (Report 104 of the Construction Industry Research and Information Association by Padfield & Mair (1984)); BS8002 (British Standards Code of Practice for Earth Retaining Structures (1994)); and EC7 (Eurocode 7: this paper considers mainly the ENV version published in 1995; the forthcoming revision is noted towards the end of the paper). Figure 12 shows a simple design example involving a propped retaining wall supporting an 8m deep excavation. Results obtained on the basis of these documents are shown in Figure 13, and will be discussed below.

Two features of these documents require definition and discussion before the individual documents are considered: (a) they use the language of limit state design, and (b) they attempt to inform the user on the degree of conservatism to be adopted in deriving parameter values used as a starting point for design calculations.



2.1 Limit state design



The term limit state design may be used so broadly as to be meaningless or with various alternative, and incompatible, narrower definitions. The latest draft of Euronorm 1990 "Basis of

Figure 12. Design example.

Figure 13. Comparison of results for CIRIA 104, EC7 and BS8002.

design" defines limit states as "states beyond which the structure no longer satisfies the relevant design criteria". If it is taken that the basis of limit state design is to avoid the occurrence of limit states, it could be inferred that all approaches to design are essentially limit state design. This, however, would render the term useless and fail to identify the main feature of approaches taken in some of the more recent codes.

Alternatively, the term is sometimes used to be equivalent to "partial factor design", or to "probabilistic design". Whilst it is true that partial factor design and limit state design have developed together, the present authors would agree with Krebs Ovesen (1995) that the two approaches are actually quite separate. Similarly, methods using explicitly probabilistic techniques have no specific relationship to limit state design.

It is proposed that the most useful understanding of limit state design can be obtained by contrasting it with "working state design". In the latter, the attention of the designer is on what it is expected will actually happen, with the construction performing in a successful manner. Stresses which will be mobilised in this

working state are calculated, and margins between them and the limiting strengths of materials are required.

By contrast, in limit state design attention is directed to unexpected, undesirable and hopefully unlikely states in which the construction is failing to perform satisfactorily. This is done by taking pessimistic values for the leading parameters involved in the design, strengths, loads and geometric features, and checking that even for these, the structure would not fail. The degree of pessimism to be associated with the parameters depends on the severity, or consequences, of the particular limit state. Hence there is an implicit probabilistic element in the approach, but this has rarely been developed in an explicit manner, either by designers or code drafters.

2.2 Ultimate and serviceability limit states

Draft Euronorm 1990 defines ultimate limit states (ULS) to be those that concern the safety of people and the safety of the structure. It requires that the following be considered where relevant:

- loss of equilibrium of the structure or any part of it, considered as a rigid body;
- failure by excessive deformation, transformation of the structure or any part of it into a mechanism, rupture, loss of stability of the structure or any part of it, including supports and foundations;

failure caused by fatigue or other time-dependent effects.

It is important to note that this definition does not mention what type of analysis will be used in studying the limit state, or whether the materials will be responding elastically or in a plastic mechanism. Rather, the definition is based entirely on the practical issues of degrees of danger, damage and, by implication, cost of repair. Thus, for example, if a structure supported by a retaining wall collapses because of wall displacement, an ultimate limit state has occurred despite the fact that the wall has merely deflected "elastically" without forming a mechanism in the ground.

Draft Euronorm 1990 defines serviceability limit states (SLS) as those that concern the functioning of the structure or structural members under normal use, the comfort of people and the appearance of the construction works. It notes that serviceability requirements should often be agreed for each individual project. Serviceability limit states are generally more difficult to define since they refer to a subjective appreciation of relatively minor problems. They are sometimes given more precise definition in contracts, but it is difficult for codes to set requirements for them which have generality.

In design, a broad appreciation of serviceability requirements often dictates the type of construction adopted - steel or concrete embedded wall, sequence of construction, number of strutting levels, etc - but reliable calculation of ground movements is usually very difficult.

2.3 Conservatism in parameter values

Practical design involves the selection of parameter values, whether a single value or a range, from information derived from diverse sources including site observations, previous experience, publications, and various tests including both index tests and direct measurements of relevant quantities. In total, this information is often quite limited in extent and may contain both uncertainties and inconsistencies. Older codes generally gave little guidance or even discussion of this matter, assuming that the designer will come to the process of calculation with a defined set of parameter values. Some of the more recent documents have attempted to give definition to the process of selecting values from the available information, or at least to indicate what degree of caution has been assumed when factors of safety have been written into the codes.

All the documents discussed here refer to both undrained strength, c_u , and drained strengths of materials, to be used as

relevant. Drained strength is defined in terms of angle of shearing resistance, ϕ' , and effective cohesion, c' .

CIRIA 104 gave two *alternative* approaches for which the designer was to consider “moderately conservative” or “worst credible” values of parameters. EC7 requires “characteristic” values of soil parameters. BS8002 requires “representative” values of both peak and critical state soil strengths. Definitions of these terms will be noted below.

All three of the documents noted above have attempted to help designers to understand how pessimistic they should be in making assessments of parameter values. In all cases, the aim is to remind the designer to consider all the available information, including effects of site history, construction processes, soil structure, and so on. Nothing new is intended here, simply that best practice, recognised by experienced engineers, is employed as the norm. It is doubtful whether there is any practical difference between moderately conservative (CIRIA 104), characteristic (EC7) and representative values (BS8002). In normal practice, engineers rarely use genuine “best estimates”, ie what they really consider to be most likely, in calculations, but prefer, quite sensibly, to include a degree of caution in their chosen values. On the other hand, they also know that if they enter most code procedures with worst credible values they will obtain uneconomic designs.

2.4 CIRIA Report 104

Published in 1984, CIRIA Report 104 was limited by intention to the design of cantilever and singly propped embedded retaining walls in stiff clays. It proposed that cantilever and singly propped walls could be designed using simple, linear diagrams of active and passive pressure, as illustrated in Figure 1, though it acknowledged that more complex pressure distributions exist in reality, especially for propped walls. In the absence of other available guidance, however, many of its recommendations, including factors of safety, have been used for a wider range of materials, and for multi-propped and even non-embedded walls. Its factors have also been used in conjunction with finite element and subgrade reaction methods, which yield earth pressure distributions redistributed from those shown in Figure 1.

CIRIA 104 did not attempt to dictate one particular approach to safety factors, but provided differing values to be used with any of the methods illustrated in Figure 2. It specifically recommended that the net total pressure method, Figure 2c should not be used since the factor of safety used has very little real effect on the design. CIRIA 104 gave two *alternative* approaches for which the designer was to consider “moderately conservative” or “worst credible” values of parameters. Moderately conservative values are said to be “a conservative best estimate”, the approach “used most often in practice by experienced engineers”. Worst credible values are “the worst which the designer could realistically believe might occur”, “not the worst physically possible, but rather a value which is very unlikely to be exceeded”. For drained conditions, it is required that the worst credible $c=0$, with ϕ set, in effect, at a critical state value, though the document does not use that term. CIRIA 104 also gave different factors for temporary and permanent works, resulting in the table of factors shown in Table 1.

Table 1. Factors of safety proposed by CIRIA 104 (simplified)

Method		Moderately conservative parameters		Worst credible parameters	
		Temp	Perm	Temp	Perm
F_s	c', ϕ'	1.2 **	1.5**	1.0	1.2
	c_u	1.5	-	-	-
F_p	$\phi \geq 30^\circ$	1.5	2.0	1.0	1.5
	$\phi' = 20-30^\circ$	1.2 to 1.5	1.5 to 2.0	1.0	1.2 to 1.5
	$\phi \leq 20^\circ$	1.2	1.5	1.0	1.2
	c_u	2.0	-	-	-

F_t	c', ϕ'	1.5	2.0	1.0	1.5
	c_u	2.0	-	-	-

** lower values for $\phi \geq 30$

The factors included in Table 1 were selected partly on the basis of current use and partly in order to ensure that comparable designs would be obtained from all the alternative methods. In practice, the factors given for the Burland-Potts-Walsh method often led to slightly more economic designs than obtained from the other methods, so this approach became popular for competitive tendering.

The factors shown in Table 1 were to be used for determination of the length of the embedded wall. In order to derive its structural strength, however, a different calculation was required. For this, the moderately conservative values were used unfactored, and a factor of safety was then applied to the derived bending moment to derive an ULS design value on which the structural section would be based. This process is illustrated in Figure 14, which shows the results of the two separate calculations, using the factor on strength method (F_s) as an illustration; these results are also shown in Figure 13, for comparison with other methods. Calculation (a) is used to derive the length and implies a bending moment which is then disregarded (the broken line in Figure 13). This is often greater than that found from calculation (b), even after the latter has been factored to obtain the ULS design value (the bending moment indicated for CIRIA in Figure 13). Thus if ever the wall should need to use the length calculated in (a), the strength provided by (b) will be insufficient to allow this, at least without considerable infringement of the margins of safety normally required on the material properties of the wall structure. This approach gives an inconsistency of length and strength; the walls are either longer than they need to be, or not strong enough.

For propped walls, some redistribution of earth pressure is likely, reducing the bending moment below that shown in Figure 14a; however, this is not possible for cantilevers, and for propped walls, as noted above, the factors of CIRIA 104 have sometimes been used with methods which take advantage of redistribution, so removing this possible extra margin. In practical design, two features may make the walls stronger than required by CIRIA 104. For concrete walls, serviceability requirements for crack widths often increase the reinforcement beyond that required for the ULS design. For steel sheet piles, it is often found that in order to drive the steel sections to the depths required by CIRIA 104 they have to be stronger than required for the ULS bending moments. Hence it is difficult to be certain from the experience of designs carried out in this way and successfully implemented whether the calculated bending moments are in fact too small, or the lengths too great.

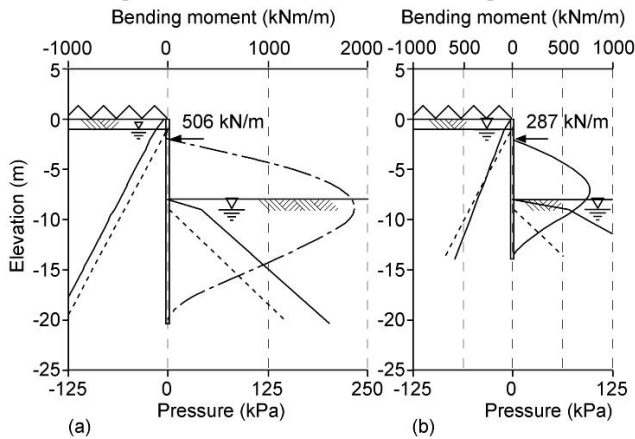
Figure 15 shows the result of a study carried out in 1990, in which a cantilever retaining wall in stiff clay was designed for permanent conditions by representatives from seven different European countries. Although this exercise was part of the development of EC7, the representatives were asked to design the wall “as they would in their normal national practice”. It can be seen that the British design, using the strength method of CIRIA 104, gave a wall longer than would have been adopted, apparently successfully, in most of the other countries. This is not surprising since the strength factors given by CIRIA 104 for clays are greater than generally used outside the UK. Thus it seems that designs by this method, with incompatible length and strength, probably have unnecessary length.

The very short walls shown in Figure 15 were derived using assumptions about water pressure different from the other designs. It can be seen that this has a greater effect than all other considerations, a point which must be considered in all designs.

2.5 Eurocode 7

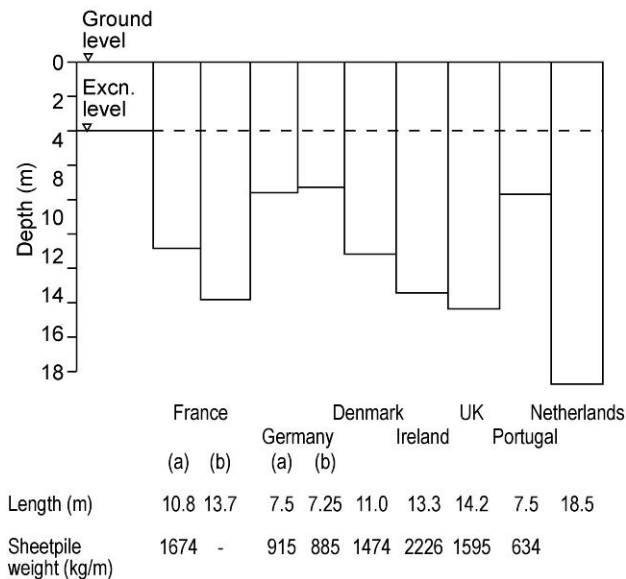
Eurocode 7 (EC7) is the geotechnical member of a unified set of codes for complete design, involving both geotechnical and

structural requirements in a consistent manner. In particular, it is



intended that the design will proceed from geotechnical to structural aspects without difficulty or confusion.

The Eurocodes generally adopt both a limit state format and partial factor methods. The partial factor methods were initially developed by engineers with an interest in probabilistic methods,



but in practice the values adopted in the codes have been selected on a more pragmatic basis, with the aim that they will not

Figure 14. Two calculations required by CIRIA 104: (a) for length and (b) for bending moment.

Figure 15. EC7 retaining wall comparison.

change designs very much from previous practice but will provide adequate margins of safety in a wide variety of situations - a failing of many lumped factor methods.

In common with other Eurocodes, the calculations in EC7 are primarily directed to the ultimate limit state. This is partly because ultimate limit states are more readily defined, as discussed above, and partly because, particularly in geotechnics, the more reliable of the available calculations deal with strength limits and failure mechanisms rather than serviceability requirements.

The limit state approach generally requires that both ultimate and serviceability limit states be considered. In EC7, this means that ground and structure displacements must be considered, as must structural serviceability requirements such as crack widths, consistently with other Eurocodes. However, it is recognised that calculation of displacement is particularly difficult, and the code

drafters wanted to avoid demands for unnecessary, difficult and possibly spurious calculations. Thus, for example, EC7 notes that "the design methods and factors of safety required by this code for ultimate limit state design are often sufficient to prevent the occurrence of [serviceability limit states] provided the soils involved are at least medium dense or firm, and adequate construction methods and sequences are adopted. Special concern is, however, required by some highly overconsolidated clay deposits in which large at rest horizontal stresses may induce substantial movements in a wide area around excavations." It can be seen from this that the choice of partial factor values adopted in the code is partly influenced by the need to prevent serviceability failures whilst relying on mechanism calculations. In addition, the section on retaining walls requires that the designer first makes an assessment of likely displacements on the basis of experience, and resort is only made to displacement calculations if this suggests that serviceability could be marginal.

Eurocode 7 follows the approach adopted in the other Eurocodes and most modern structural codes, using "characteristic" and "design" values, but with an important change in the definition of characteristic values for geotechnical design. In structural codes, characteristic values are generally defined as a fractile of the results of particular, specified laboratory tests on specimens of material. However, EC7 defined characteristic geotechnical parameter values as a *cautious estimate* of the value *affecting the occurrence of the limit state*, ie whichever limit state is under consideration. Surrounding text makes it clear that this 'cautious estimate' is an assessment made by the designer, rather than a value derived from statistical manipulation of test results, and that it is to represent what actually governs behaviour in the ground. Thus the designer is to take account of time effects, brittleness, soil fabric and structure, the effects of construction processes and the extent of the body of ground involved in a limit state, in relation to its variability. The designer's expertise and understanding of the ground are all encapsulated in the characteristic value; he is to consider both project-specific information and a wider body of geotechnical knowledge and experience. No specific requirements about the use of peak or critical state values are given. The selection of characteristic values is discussed at greater length by Simpson & Driscoll(1998).

The ENV version of EC7 requires that designs be checked for three "cases" or sets of partial factors, as shown in Table 2.

Table 2. Partial factors from ENV1997-1.

Case	Actions			Ground Properties			
	Permanent	Variable		$\tan \phi$	c'	c_u	q_u 1)
	Unfavourable	Favourable	Unfavourable				
A	1.00	0.95	1.50	1.1	1.3	1.2	1.2
B	1.35	1.00	1.50	1.0	1.0	1.0	1.0
C	1.00	1.00	1.30	1.25	1.6	1.4	1.4

1) Compressive strength of soil or rock.

Both the geometry, the length of wall determined from geotechnical calculation, and the structure are to comply with all three cases, though Case A is generally non-critical for embedded retaining walls. This makes it possible to ensure that the length and strength of the wall are compatible. The two sets of factors specified for Cases B and C ensure that both safety and reasonable economy can be obtained for a wide range of design situations, in which uncertainties in ground loads, external loads and soil properties may combine in varying degrees.

EC7 requires that embedded retaining walls be designed as though the level of the soil surface of the supporting (passive) soil were up to 0.5m below any level reasonably foreseen by the

designer. This was intended to give a margin for unexpected events which cannot reasonably be covered by factors of safety, especially for walls with small penetration into highly frictional soils (Simpson & Driscoll 1998).

In contrast to CIRIA 104 and BS8002, EC7 does not specify how the earth pressure distribution to be used for the design of embedded walls. Its requirements are simply that equilibrium must be demonstrated, with compatible strains and using loads and strengths with the specified partial factors applied. Thus, simple earth pressure diagrams like that shown in Figure 1 can be used, but it is also permissible to take advantage of redistribution calculated by numerical analysis or other rules such as those illustrated in Figures 10 and 11. For example, Figure 16 shows an acceptable ULS design calculation in which the propped wall is just stable, with large displacement, and earth pressures have been redistributed towards the prop.

The length and bending moment calculated using EC7 for the example shown in Figure 13 were derived from Case C and the strut force from Case B, since these proved to be the critical cases. For a simple conservative design, the program STAWAL uses pressure diagrams like those of Figure 1, producing the results marked STW in Figure 13. But for greater economy, the EC7 calculations were repeated using the FREW program, discussed below, and a finite element program SAFE. The length from EC7 is somewhat less than the previous British practice, represented by CIRIA 104, as is usually the case. Design of struts to Case C only would give a reduction in strut force of 20 to 30%, which might be considered undesirable, but the governing value is given by Case B. The EC7 bending moment is less than the CIRIA moment calculated using factored strength, the broken line in Figure 13 which is disregarded in the CIRIA method, but more than that adopted for ULS design by CIRIA 104. In this example, the effect of the 0.5m unplanned excavation was to increase the bending moment by 39% and the strut force by 48%. Simpson & Driscoll (1998) show that for cantilevers the effect may be even greater. Clearly, designers need to be aware how sensitive the structures are to this geometric parameter; how codes should best deal with this remains a matter of debate.

The final design of the structure depends on the structural code in use as well as the geotechnical code. For design of reinforced concrete walls, EC7 interfaces with Eurocode 2, the ULS and SLS requirements of the two codes being compatible. It is often found that the structural design of embedded walls is dominated by SLS crackwidth limits, so the on-going debate about the limits appropriate to concrete in the ground requires urgent resolution.

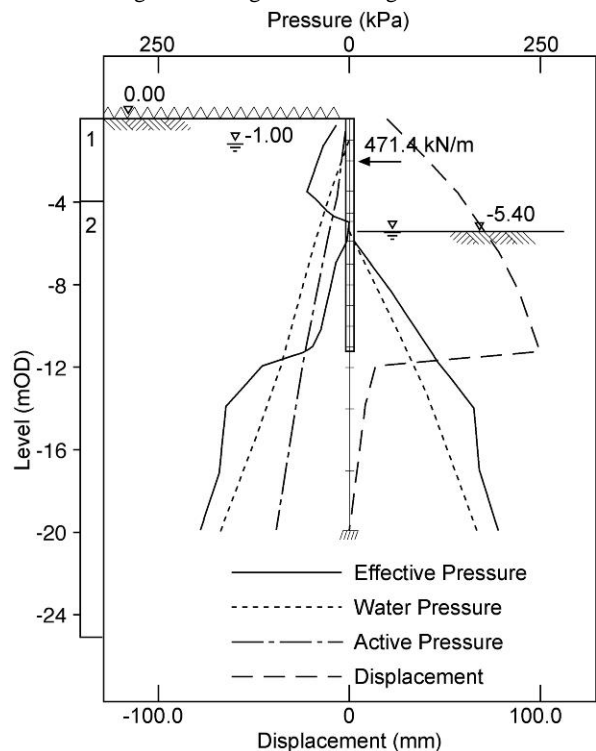
For steel sheet pile walls, EC7 interfaces with Eurocode 3 Part 5, which allows plastic design. Figure 17 shows the moment-curvature relationships allowed for four classes of sheet piling (Schmitt 2000). Very thin sections, such as trench sheeting, fall into Class 4 for which local buckling prevents the attainment of the full elastic moment of resistance (M_R). For more robust sections in Class 3, the full elastic M_R can be used and the full plastic M_R can just be attained for Class 2. Larger sheet pile sections fall into Class 1 for which a prescribed degree of rotation at a plastic hinge is allowed. Economies of up to 30% in materials are anticipated as a result of this procedure.

Eurocode 3 Part 5 also requires that the effect of slippage between unwelded U-section sheet piles be allowed for. In the absence of international agreement, the significance of this is left to national decision. Further research is required, and is underway, into the significance of clutch slippage and the possible advantages or limitations of crimping the clutches.

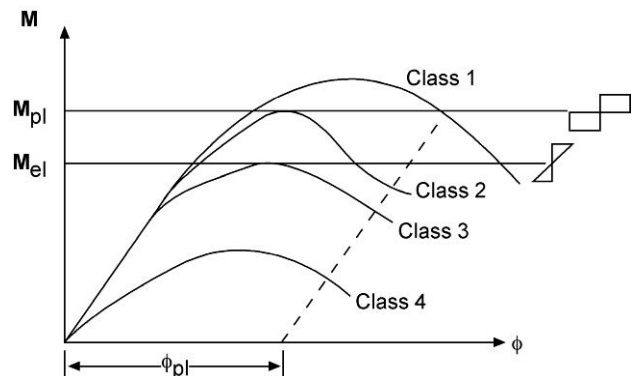
2.6 BS8002

BS8002 is the British Standard Code of practice for Earth retaining structures, published in 1994. Its approach to safety and serviceability is different from most other recent documents, being based on the belief that serviceability rather than ultimate limit

states should govern design of retaining walls. It includes fac-



tors, which are in effect partial factors or strength factors, with values fairly similar to EC7 Case C. However, these are regarded as “mobilisation factors”, M , with the express purpose of limiting displacements of walls at the serviceability limit state.



Bolton et al (1990a,b) showed that the displacements of retaining walls could be related approximately to the shear strains occurring in the ground and that these shear strains could in turn

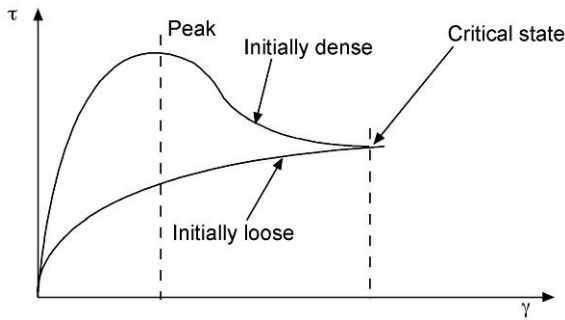
Figure 16. EC7 Case C ULS design.

Figure 17. Moment-curvature curve considered in EC3 Part 5.

be related to the proportion of the soil’s strength which has been mobilised. Hence using a mobilisation factor M to limit the proportion of strength mobilised should also limit wall displacements. In essence, this was also acknowledged in EC7, as noted above, though EC7 saw factors primarily as covering uncertainty in relation to ultimate limit state design.

Design values used in limit equilibrium calculations are derived from representative values by dividing strength terms by $M = 1.2$ on peak values of $\tan\phi$ and c' , and $M = 1.5$ on c_u . The representative soil strength is said to be a “conservative estimate

of the mass strength of the soil”; “conservative values” are “val-



ues which are more adverse than the most likely values”, tending “towards the limit of the credible range of values”. For drained behaviour, a second check on the strength parameters is obtained by requiring “representative” values of two strengths of the soil: peak strength (c'_p, ϕ'_p) and critical state strength ($c'=0, \phi'_{crit}$), as illustrated for two shear strength tests in Figure 18. The critical state strength of a soil is the strength available at large shear

Figure 18. Peak and critical state shear strengths.

strains when any dilation has ceased. The design values must not exceed the representative critical state strength of the soil. In practice, $\tan \phi'_{crit}$ is very unlikely to be less than $\tan \phi'_p/1.2$, but the critical state requirement may govern when c'_p is significantly large.

BS8002 requires that the earth pressure distribution used for design will be of the simple form shown in Figure 1. This effectively precludes methods which take advantage of stress redistribution, including numerical methods, and also may lead to an underestimate of prop forces.

BS8002 also has a requirement for “unplanned excavation”, slightly more severe than that of EC7, and it stipulates a minimum surcharge of 10 kPa to be imposed behind all retaining walls. These two requirements, taken together, have been found to be rather severe, especially for small retaining walls; they are being modified in a forthcoming revision.

BS8002 results for the design example of Figure 12 are shown in Figure 13. The calculated length is similar to EC7, but BS8002 leads to a high value for bending moment, mainly because it has no provision for redistribution of earth pressures.

BS8002 recommends that structural forces and bending moments calculated using its mobilisation factors should be used both as SLS and ULS design values for structural design. This leads to some practical difficulties:

- It clearly means that SLS will generally govern structural design. Reinforced concrete design to modern codes generally starts from ULS calculations with SLS as an additional check, so it is inconvenient to have a design governed by SLS requirements. In practice the ULS structural strength will inevitably exceed the demands of BS8002 by a further margin.
- Some structures do not have SLS requirements. This applies to structural failure of masonry structures, not really relevant to embedded walls, and to some sheet pile walls, especially in temporary works, if displacement is not a critical criterion.

The interface between geotechnical and structural design has been seen by designers as BS8002’s greatest weakness. An attempt to overcome some of the problems for reinforced concrete walls has been made by Beeby & Simpson (2001). For embedded walls, they recommend that the results of BS8002 design should be used for ULS design of the wall, and that for SLS the same mobilisation factors should be used but that unplanned excavation should be omitted from the calculations.

2.7 Some recommendations

Both CIRIA 104 and BS8002 suggest that the designer consider two different values, either moderately conservative and worst credible, or peak and critical state, with different factors applied to the different values in design calculations. This is a valuable process because it reminds the designer to consider explicitly the range of variability of the material and how its available strength could change in response to deformation. CIRIA 104 offers two *alternative* calculations whilst BS8002 requires that *both* peak and critical state values be considered in the process of deriving a single value for one calculation. The latter process requires very little additional effort on the part of the designer whilst providing the benefits of a double check. The present authors recommend that two checks should always be made, expressed here in the terminology of EC7:

- The designer should check that the ULS design (factored) value of any parameter is never more optimistic than his assessment of the worst value which could credibly govern the field situation.
- The ULS design value of the shear strength should never be greater than a cautious (ie “characteristic”) estimate of the critical state strength of the material.

The strength of an embedded wall must be sufficient to use its length. It is therefore unwise to derive length and strength from separate, unrelated calculations.

The possibility of unplanned excavation must be considered very carefully by designers and possibly as a code specification.

Although some of the effects of factors of safety, or mobilisation factors, are clear, their full effect is difficult to quantify. Some uncertainties are appreciated at the time of design and others are not known. When failures occur, an element of human error can often be identified, but investigations often also reveal that many successful structures had errors in their designs or construction which did not cause failure because there were sufficient margins of safety. Large errors are likely to be spotted, but errors of up to 50% may not be. This gives an additional reason, beyond strength mobilisation and physical uncertainties, why factors of safety are needed and why their values should be judged so that further economies can gradually be achieved, avoiding step changes which might have unpredictable results.

3 RECENT DEVELOPMENTS IN UNDERSTANDING

3.1 General

Perhaps the most significant developments in embedded retaining walls have been

1. the introduction from the early 1960’s of diaphragm and bored pile installation techniques, which enabled the construction of very stiff embedded walls in overconsolidated clay deposits, and, to some extent associated with this,
2. the development of novel and more economical temporary and permanent support systems, sometimes (particularly for temporary works) in conjunction with the Observational Method of construction (Peck 1969a, Nicholson et al 1999).

It soon became clear that the design methods developed for sheet pile walls in sands were not necessarily applicable to much stiffer concrete walls in overconsolidated clays. First, the traditional application of a factor of safety to the passive earth pressure coefficient appeared to give unrealistically large depths of embedment. Secondly, there was a concern that the traditional application of a factor of safety to the passive earth pressure coefficient might in any case not be appropriate. It was thought that, owing to the high *in situ* lateral earth pressures associated with overconsolidated clays, the lateral stresses in the retained soil might not fall to the active limit with wall movements small enough to be acceptable under working conditions. A wall that was successful in limiting movements would then have to with-

stand bending moments much greater than those based on fully active conditions in the retained soil.

The first of these was addressed by Burland et al (1981), who showed that the application of a factor of safety to the passive earth pressure coefficient $F_p=2$ is unduly conservative (in comparison with a uniform factor of safety on soil strength) at values of ϕ' less than about 27° - i.e. values typical of clays (Figure 4).

Finite element analyses carried out by Potts & Fourie (1984) appeared to show that an embedded wall in an overconsolidated clay deposit could indeed suffer bending moments much larger than those associated with fully active conditions in the retained soil. Their analyses were carried out with using a linear / Mohr-Coulomb model and with zero pore water pressures. Similar results were reported from a non-linear model with high stiffness at small strain and more normal water pressures by Simpson (1992), who also showed that a substantial drop in bending moments could occur if the stiffness of the wall is modelled for a cracked concrete section. These analyses neglected both the stress relief and recent stress history resulting from wall installation. Consideration of more recent analyses and field data suggests that the lateral earth pressure coefficient might be expected to fall by about 10% to 20% as a result of wall installation. It is possible that the soil behind the wall, unloading from high horizontal stresses, will move more rapidly towards the active condition than some of the analyses suggest (Powrie et al 1998).

3.2 Berms

Berms have been used to help stabilise embedded retaining walls for decades (Peck 1969b). In given ground conditions, the degree of support offered by a berm will depend on the height H , the bench width B and the slope S (Figure 19). The maximum slope S will be governed by soil and groundwater conditions, while H and B may be limited by considerations of space and access. In soils of low permeability, the drainage conditions assumed in design and the length of time for which the berm is required to remain effective will be important. Most methods of representing the effect of a berm in a limit equilibrium analysis are semi-empirical even if conditions on site approximate to plane strain, while if the berm is removed in sections along its length to allow permanent supports to be installed, a three dimensional analysis may be required to assess stability. The difficulty of analysis may explain why berms have often been used in conjunction with the Observational Method (Tse & Nicholson 1993, Powrie et al 1993, Gourvenec et al 1997).

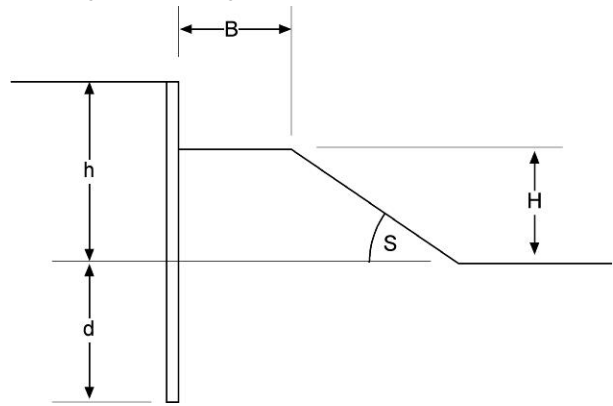
Common methods of representing an earth berm in a limit-equilibrium analysis are

- as an equivalent surcharge (Padfield & Mair 1984; Fleming *et al* 1992);
- by means of a raised effective formation level (Fleming *et al* 1992: Figure 20); and
- by carrying out a single (NAVFAC Design Manual 7.02 1986) or multiple (Figure 21) Coulomb wedge analysis.

Powrie & Daly and Daly & Powrie (submitted) describe the results of a series of plane strain centrifuge model tests of embedded cantilever retaining walls of various embedment depths supported by berms of different sizes. They also analyse the model tests using each of the three limit equilibrium methods outlined above, with the following results.

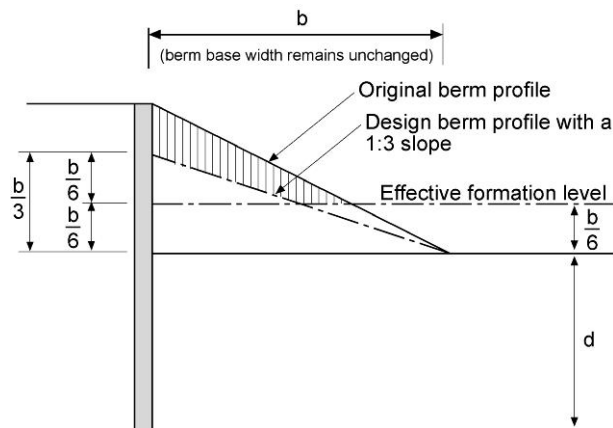
- The equivalent surcharge method is highly conservative, giving factors of safety between 15% and 25% less than the multiple Coulomb wedge analysis for the berm/wall geometries

investigated. The degree of conservatism increases with in-



creasing berm size and decreasing embedment depth.

- The raised effective formation level approach is conservative but less so, giving factors of safety between 5% and 11% less than the multiple Coulomb wedge analysis. The degree of



conservatism increases with decreasing embedment depth, but is less sensitive to berm size.

- For a wall of given embedment, the factor of safety is increased significantly if a larger berm is used. Indeed, increasing the size of the berm is more effective in enhancing wall stability than increasing the depth of embedment of a wall supported by a smaller berm.
- For a berm of a given geometry, the mobilisation factor shows no significant increase as the depth of embedment of the wall is increased.
- The berm prevents swelling of the soil immediately in front of the wall below formation level, so that soil/wall friction in this zone may not act upward on the wall as usually assumed in passive conditions – at least in the working state. However, there may still be upward movement, and beneficial wall fric-

Figure 19. Definition of berm geometry.

Figure 20. Representation of a berm by raising the effective formation level.

Figure 21. Check on berm effect using multiple Coulomb wedges.

tion, if the passive wedge starts to move, i.e. at the onset of collapse.

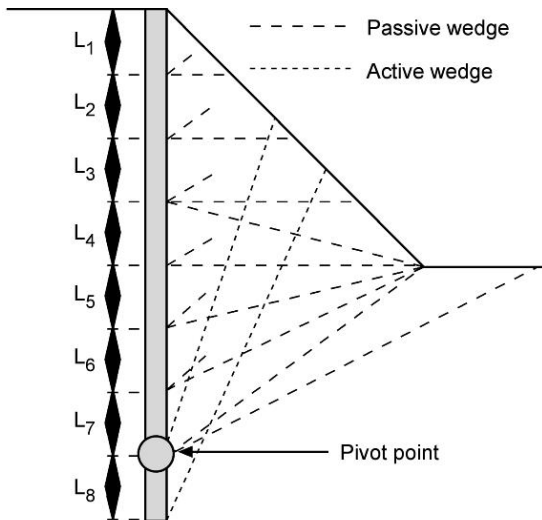
Perhaps the main shortcoming of the analyses described above is that they refer to conditions of plane strain. In other words, they assume that the berm remains intact over the entire length of the wall throughout the excavation and construction period. In reality, it will usually be necessary to remove the berm in sections so that the permanent support (e.g. in the case of a road cutting, a formation level prop) can be installed.

Gourvenec & Powrie (2000) carried out a series of three dimensional finite element analyses to investigate the effect on wall movements of the removal of sections of an earth berm supporting an embedded retaining wall in overconsolidated clay. In general terms, the results showed that

- removal of a section of an earth berm will result in localised displacements in the vicinity of the unsupported section of the wall, the magnitude and extent of which increase with the length of the berm section removed and with time following excavation;
- wall movements during removal of a berm in sections can be minimised by reducing the width of the sections removed, and
- a number of sections can be removed simultaneously without increasing wall movements, as long as successive unsupported sections are separated by a sufficient length of intact berm.

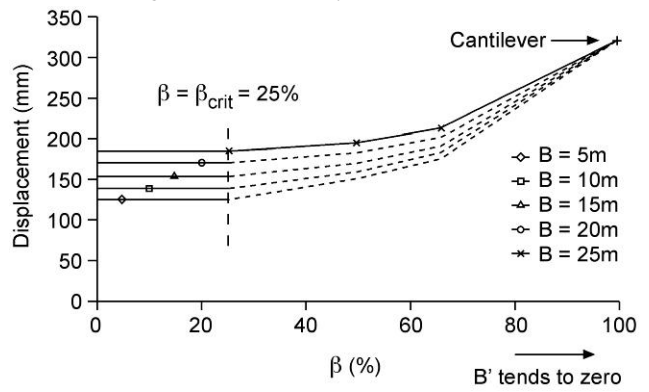
For a wall along which bays of length B are excavated simultaneously at regular intervals separated by sections of intact berm of length B' , the degree of discontinuity β may be defined by the ratio of the excavated length to the total length i.e. $\beta = B/(B+B')$. Then, for a given wall/berm geometry, ground conditions and time period there is a critical degree of berm discontinuity $\beta (=B/(B+B'))$ that is independent of the length of the unsupported section B . It can be seen in Figure 22 that

- if the degree of discontinuity β of a berm supported wall is less than its critical value β_{crit} , displacements increase linearly with the length of the unsupported sections;
- if β exceeds its critical value then displacements become a function not only of the length of the unsupported section but



- 1 Subdivide the problem as shown
- 2 Check active and passive wedges
- 3 Draw equivalent earth pressure diagram

also of the degree of discontinuity; and



- as β is increased above its critical value displacements increase more rapidly with continued increases in β .

In practical terms,

when several sections along the berm are removed simultaneously, as may well occur with construction of a long retaining wall in bays, the sections removed should be separated by a section of intact berm between one and three times as long as the section removed (i.e. $\beta = 25$ to 50%). This is because the maximum wall movement (at the centre of the unsupported section) begins to increase with β above $\beta = 25\%$, and the minimum wall movement (at the centre of the supported section) increases with β when $\beta > 50\%$.

- If minimisation of wall movements is critical, the length of the unsupported bays should be as small as possible, as the additional wall movement (compared with the case of an intact berm) increases in proportion to the length of berm section removed.

Easton *et al* (1999) carried out three dimensional finite element analyses of a berm supported retaining wall having the cross sectional geometry shown in Figure 23, with berms of different height within the profile envelope indicated. Along the wall, the berm was divided into three central bays 5m in length and two outer bays each 30 m in length. The analysis involved excavation of the berm from the central bay over a period of 30 days and placement of a formation level prop slab (1 day), followed by excavation (30 days) and propping (1 day) of each of the other two 5m bays in turn, and finally the two outer (30m) bays together (30 days in total).

By carrying out comparative analyses in which excavation and propping took place with a uniform dredge level in each bay, Easton *et al* (1999) deduced a relationship between berm height and the equivalent uniform increase in formation level in front of the wall to give the same maximum wall movement (Figure 24). The analyses were carried out for soil strength parameters $\phi' = 22^\circ$ ($c' = 0$ and 20 kPa) and $\phi' = 28^\circ$ ($c' = 0$ and 10 kPa).

Figure 22. Normalised wall crest displacement at the center of the unsupported section against degree of discontinuity β for different excavated bay lengths B . (The data points and solid lines represent confirmed findings, and the broken lines conjecture.)

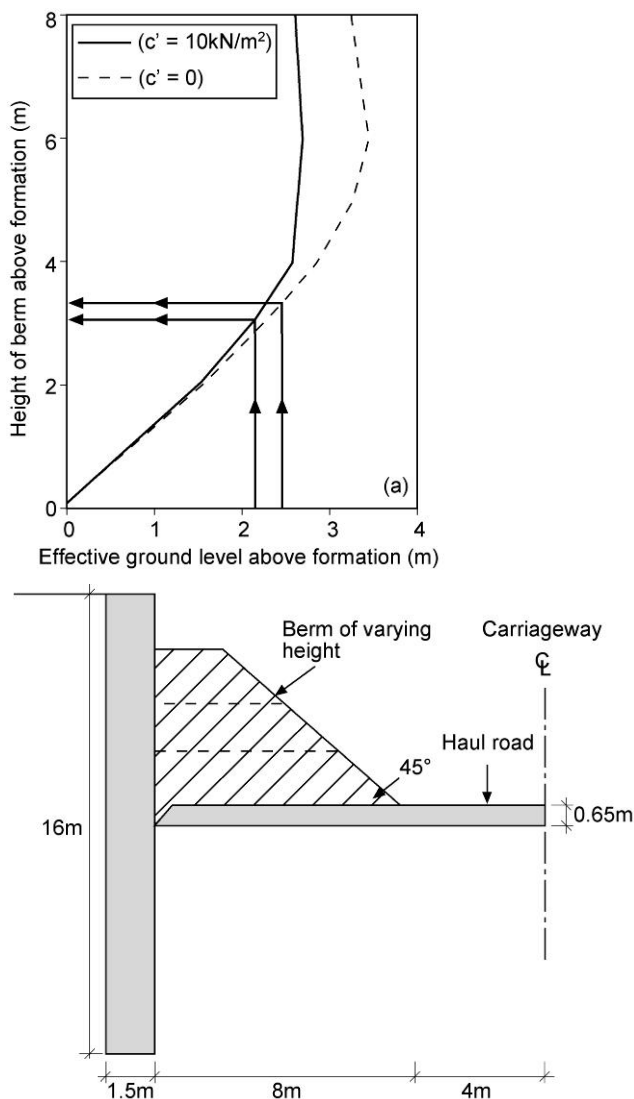


Figure 23. Wall/berm cross sectional geometry investigated by Easton *et al* (1999).

In all cases, increasing the berm height above about 5 m has little effect, but this is partly a result of the overall berm envelope adopted in this case.

3.3 Temporary props

Embedded walls retaining the sides of large excavations are often supported at some stage in the construction process by temporary props at one or more levels, in order to reduce wall and ground movements. The provision of temporary props is costly in terms of money, time and risk to the site operatives involved in installing and removing them. The advantages of reducing the number of temporary props and/or eliminating some levels of propping in a large excavation are therefore considerable.

Until the mid-1990's, there was a widely-held view within the construction industry that the procedures used in design tended to overestimate actual prop loads. Powrie & Batten (2000) and Batten & Powrie (2000) investigated this with reference to field data and analyses of the temporary prop loads developed during the construction of the London Underground Jubilee Line Extension stations at Canada Water and Canary Wharf. Prop temperatures were also measured, to assess their influence on prop loads.

Figure 24. Relationships between berm height and equivalent uniform increase in formation level, $\phi' = 22^\circ$.

For the stiff, reinforced concrete walls at Canada Water and Canary Wharf, temporary prop loads similar to those measured in the field (neglecting temperature effects) were calculated using limit equilibrium and finite element analysis, provided that appropriate soil parameters and input assumptions were used. In finite element analyses, the key factors were the effect of wall installation and the timescale of excess pore water pressure dissipation in low permeability strata. In limit equilibrium analyses, realistic prop loads were calculated on the basis of fully-active conditions in the retained soil and pore water pressures in equilibrium with the prevailing groundwater regime. However, it is quite possible that this approach may have overestimated the pore water pressures (if long term equilibrium conditions had not yet been achieved) and underestimated the lateral effective stresses.

In general terms, the results presented by Powrie & Batten (2000) and Batten & Powrie (2000) suggest the following

1. Although in design a margin of safety is essential to allow for events such as the accidental removal of a prop, the over-prediction of prop loads seemed to be the result of a consistently conservative set of design assumptions rather than any flaw in the underlying soil mechanics principles.
2. Temperature-induced axial loads may account for a significant proportion of the total load carried by a prop installed at a low temperature. Temperature-induced loads can be estimated from the anticipated temperature rise, the coefficient of thermal expansion of the prop, and the degree of end restraint provided by the wall and the soil behind it. For props near the crest of a stiff wall, the degree of end restraint could be of the order of 50%. Low-level props might be restrained with an effectiveness of perhaps 65%, but the range of temperature experienced by a prop may reduce with depth within the excavation.
3. In the absence of non-uniformities due to a lack of fit at the ends of a prop, bending moments due to wall rotation and/or temperature gradients across the prop of the same order as those due to self-weight effects must be expected. However, a lack of fit between the walings and the ends of a prop could increase secondary bending effects substantially - a point which may need to be considered in design.

3.4 Grouted anchors

Grouted and proprietary ground anchors can also be used as temporary supports, giving the advantage of an open site unimpeded by props. Anchors can be prestressed, and because they are usually fairly extensible they give a relatively constant force, unaffected by wall displacement. Hence the prestress can be used to dictate the distribution of earth pressures on the wall, both under working conditions and at collapse. In this respect they are unlike the passive deadman anchors investigated by Rowe.

3.5 Wall flexibility

In general terms, it may be shown that on excavation in front of an embedded wall in a soil of unit weight γ , rigid body rotation is governed by γ/G^* , while bending deformation depends on $\gamma H^4/EI$. (G^* is the rate of increase of shear modulus G with depth.) A flexibility number quantifying the relative importance of wall deflexions due to bending and rigid body rotation (Figure 7) may then be identified as $(\gamma/G^*) \div (\gamma H^4/EI) = G^*H^4/EI$. This might be viewed as analogous to Rowe's dimensionless group m_p .

For multi-propped walls, in which the opportunity for rigid body rotation may be limited, Clough *et al* (1989) defined a sys-

tem stiffness $EI/\gamma_w h_{av}^4$, where γ_w is the unit weight of water and h_{av} is the average distance between the supports. They then produced a design chart, based on finite element analyses, to relate the system stiffness to maximum lateral wall displacements, and hence ground movements, for a given factor of safety against basal heave.

Addenbrooke *et al* (2000) defined a further measure of wall flexibility, which they termed the displacement flexibility number $\Delta = EI/h^5$, which has units of kN/m^4 in plane strain (i.e. with EI in kNm^2/m). As with Clough *et al* (1989), h is the distance between supports. By means of an extensive series of finite element analyses of undrained excavations in stiff clay, they showed that, for a given initial stress regime and prop stiffness, support systems with the same flexibility number will result in practically the same maximum wall deflection and ground displacement profile.

3.6 Long-term lateral stresses

Designers are sometimes concerned about the possibility of the *in situ* lateral stresses becoming re-established against the wall, for example due to creep. This is a particular problem in stiff clays, where the *in situ* K_0 is greater than unity, so a return to *in situ* values would involve very high final earth pressures. This seems unlikely: provided that the soil can sustain shear stresses, it is quite possible for the lateral stress some distance away from the wall to differ from the lateral stress acting on the wall itself – provided of course that the condition of horizontal equilibrium is satisfied. However, if the soil tends to creep so that in the long term shear stresses reduce, it is likely that the coefficient of earth pressure will tend towards unity, rather than increasing to values > 1 after the end of construction.

Long-term measurements behind embedded walls retaining London Clay at Walthamstow, Hackney, Reading and Malden generally indicate a slight reduction in the measured lateral stresses near the wall over an eight year period following construction (Carder & Darley 1998).

Additional evidence against the long-term re-establishment of *in situ* lateral stresses comes from Page (1995). Page carried out plane strain centrifuge model tests using overconsolidated speswhite kaolin clay, in which the stress changes associated with the excavation of a diaphragm wall trench under bentonite slurry and subsequent concreting were simulated. Details of the centrifuge model were broadly as given by Powrie & Kantartz (1996), except that at the opposite end of the centrifuge strong-box from the trench the initial *in situ* lateral stresses were maintained (i.e. this boundary was stress- rather than strain-controlled). The overall width of the model was 55 m at field scale. Total lateral stress transducers installed in the soil near the trench measured no tendency for the reinstatement of the *in situ* lateral stresses following hardening of the model diaphragm wall and the establishment of long term equilibrium pore water pressures.

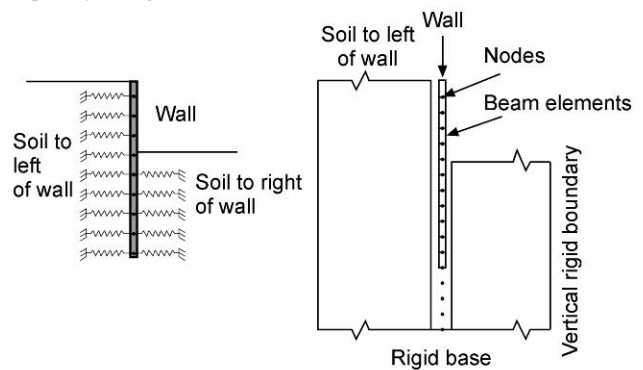
4 ANALYTICAL METHODS

Analytical methods which are used in the design of embedded retaining walls may be divided into five types:

- hand calculations, finding equilibrium between active and passive pressures
 - software which replicates (a) and may have rules or theories about how the active pressures are redistributed for propped walls, such as SPOOKS
 - subgrade reaction analyses such as MSHEET or WALLAP
 - pseudo finite elements such as FREW, described further below
- full finite element analyses, which may extend into 3D analysis.

Although performing hand calculations may be good for developing understanding, it is very tedious and prone to error in all but the simplest examples. Many walls are designed quite

adequately using software as in (b); some of the available theo-



a) Subgrade reaction model

b) FREW

ries about redistribution of earth pressures (eg as shown in Figures 10 and 11) improve the economy of design by these methods. Subgrade reaction methods may also assist in the understanding of earth pressure distribution and are generally found to give bending moment results similar to more advanced finite element models. However, they rely heavily on precedent and experience, particularly in evaluation of the needed coefficient of subgrade reaction which is not a material parameter and so cannot be related simply to soil test results or theories of deformation behaviour.

4.1 FREW

The set-up of a subgrade reaction program is compared with that of FREW in Figure 25. To the user, the input, use and speed of calculation of FREW are very similar to a subgrade reaction program. However, whereas subgrade reaction represents the ground as a set of individual springs, with no interaction between them, FREW represents a linear elastic continuum, with the earth pressures at the interface with the wall limited by active and passive considerations. The elastic continuum redistributes earth pressures in a fairly realistic manner and the limits on earth pressure ensure that the degree of redistribution is comfortably within the strength capacity of the ground.

To model the elastic continuum, two flexibility matrices, computed by finite element runs each with 100 load cases, have been inverted to give stiffness matrices and pre-stored. One of these represents ground with constant stiffness with depth, and the other with linearly increasing stiffness from zero at the surface. Proportionate addition of the two matrices has been shown by comparison with finite element analysis to give a good approximation to the stiffness matrix of ground with any linear increase of stiffness with depth. For irregularly varying stiffness, a best fit linear increase is first found, which is then adjusted in accordance with an energy-based formulation published by Pappin *et al* (1986).

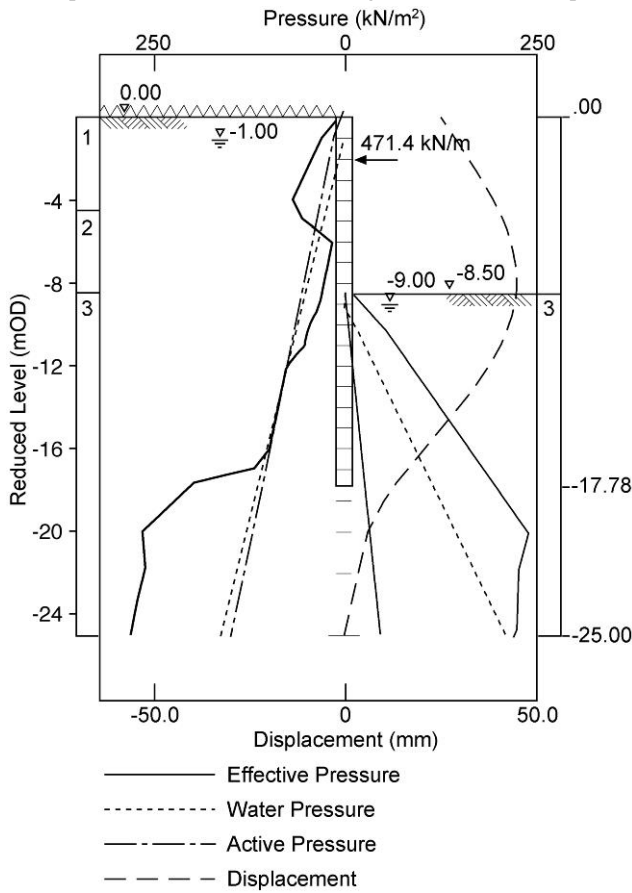
FREW works within the upper bound approach, discussed earlier, with overall limits on active and passive forces, but without the assumption of linear increase of limiting earth pressure with depth. There are however other, more detailed constraints on the earth pressure distribution, which relate to changes of stress over short distances potentially causing local failures within the soil mass, as described by Pappin *et al*.

FREW is used for both SLS and ULS checks. Figure 16 shows an extreme example in which the wall is on the point of

Figure 25. Comparison of subgrade reaction model and FREW.

failure, but is just satisfying EC7 Case C, while Figure 26 shows the earth pressure distribution computed by FREW for EC7 Case C in Figure 13. In both cases, the redistribution of earth pressure to the prop is apparent.

Simpson (1994) noted that omitting to check vertical equilib-



rium is often a cause of wall failure, and that vertical shear forces between soil and wall help to restrict displacement. These effects are not considered by either subgrade reaction programs or FREW. They are automatically included in finite element analyses, however.

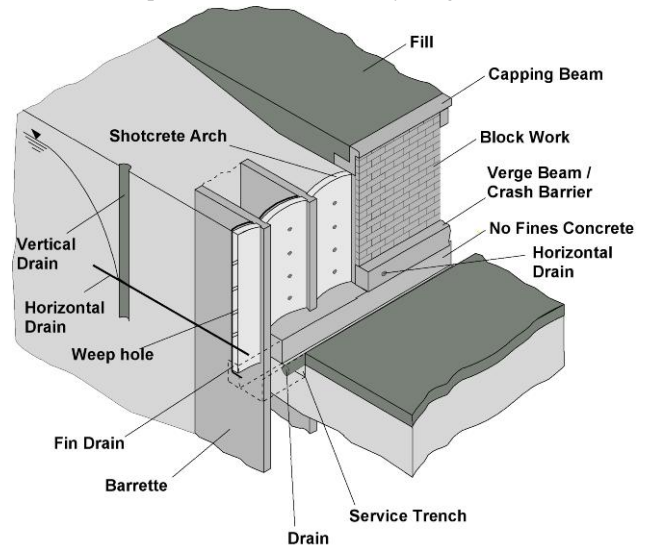
4.2 Finite element and finite difference programs

Finite element and finite difference programs make it possible to study the “complete” problem, of a strutted excavation, for example, in 2 or 3 dimensions. They can also be used powerfully to provide insights into details of the problem, such as the construction of a diaphragm wall panel, as discussed above.

The emphasis in numerical analyses has often been on computation of ground movements, with the behaviour of the wall itself as a secondary consideration. Nevertheless, one of the advantages of such analysis is the possibility of involving both the wall and other connected structure in a complete interaction analysis. Figure 27 is a cross section through both temporary and permanent works for a deep station box, showing computed bending moment diagrams for a double skin sheet pile cofferdam and the permanent diaphragm walls, with a piled wall between, used to allow access for the diaphragm walling. Some of the embedded walls in this project were T-section diaphragm walls, which are sufficiently thick, overall, that “plane sections do not remain plane” as assumed in ordinary beam analysis. The finite element analysis allows for this, the friction on the wall having a significant moment about its neutral axis, which generally acts in a beneficial manner.

Figure 28 shows an embedded wall consisting of concrete barrettes placed normal to the line of the wall, with arches spanning between them in plan, formed of sprayed concrete. In this project, water pressures on the back of the wall are permanently relieved by drainage, as shown in Figure 29. A finite element study was carried out when the project was built to determine likely wall movements, and more recently time-dependent analy-

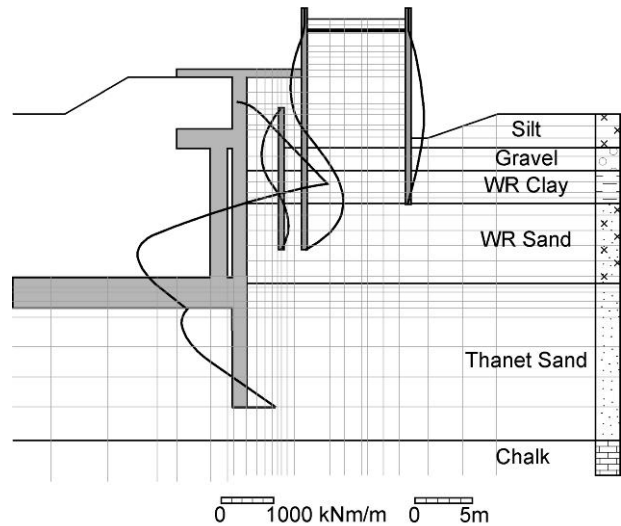
ses have been performed to check likely long term movement.



The contours on Figure 29 show the computed piezometric levels and the arrows indicate computed displacement. This example used the Brick model developed for analysis of excavations in stiff clay and published by Simpson (1992). On this and other projects it has been found that although the Brick model gives a good match to the best available small strain laboratory tests it tends to over-estimate displacements measured in full scale constructions in stiff clay. The reasons for this require further inves-

Figure 26. EC7 Case C calculation in FREW.

Figure 27. Interaction analysis between four embedded walls.



tion, particularly of the small strain stiffness of undisturbed natural clays.

It is evident from the earlier sections of this paper that the behaviour of embedded walls in service is complex and depends critically on the non-linear stress-strain properties of the ground, and also of the structure. The lack of adequate models, especially for 3D work, may partly explain why relatively few analyses have been published. Finite element methods offer the opportunity to model the stress-strain properties of the ground as accu-

Figure 28. Barrette wall with shotcrete arches.

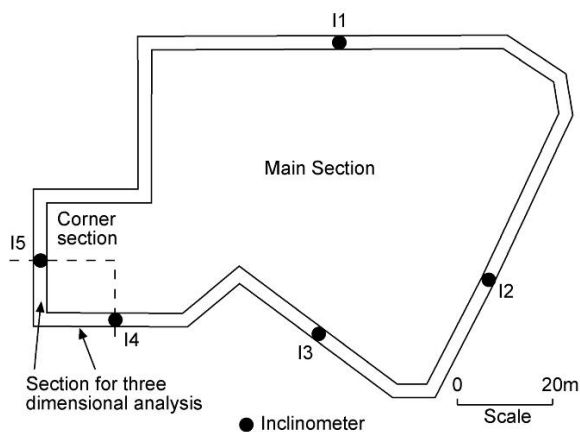
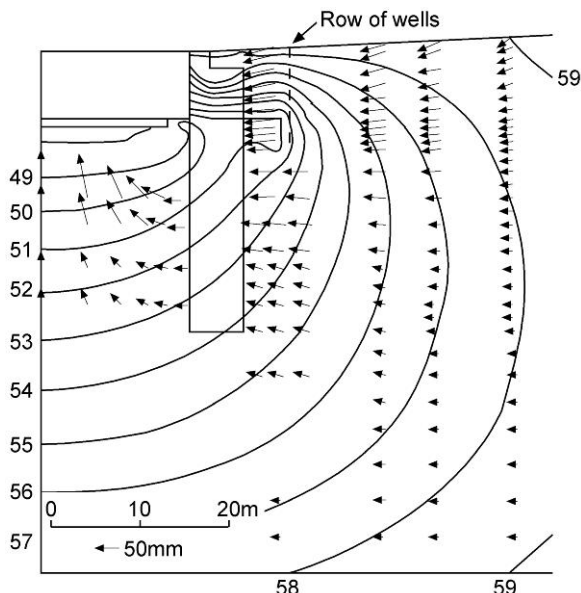


Figure 29. Section through barrette wall, showing drains, displacements and piezometric levels.

rately as knowledge allows. A wide range of stress states often results from an analysis, so one of the principles adopted in de-



velopment of Brick was that it should be able to cover as wide a range of soil behaviour as possible. Thus, although it was initially developed for stiff London Clay, it was shown that a soft clay of similar marine origin could be modelled using essentially the same parameters, given the pre-consolidation pressure (Simpson 1992).

Most finite element analyses carried out to date of complete constructions, as opposed to details, have been in 2D. This often requires considerable approximation, and it is difficult to use field measurements to improve predictions when the effect of the geometric approximation may be large, particularly since the stiffness of ground is usually very non-linear. Some interesting 3D examples from recent publications are noted here.

Ou *et al* (1996) report a study related to the excavation for the Hai-Hua building, Taipei, shown in plan in Figure 30. Owing to the limitations of computing power, which are still significant

Figure 30. Plan of the Hai-Hua building, after Ou *et al* (1996).

for 3D work, they investigated carefully the effects of mesh gradation and also developed correlations between 2D and 3D re-

sults. This results in a useful development of understanding of the likely significance of 3D effects. The ground conditions involved were principally firm clays, and parameters were developed for a Duncan and Chang model. The paper concentrates on displacements, and a full 3D analysis of the complete excavation was not undertaken. At inclinometer positions I1, I2 and I3, on long sides of the excavation, it was found that a 2D analysis gave very close agreement with field measurements, but the 3D effects were important at inclinometers I4 and I5. Here, both their 3D analysis and their method of correcting 2D results worked well. Ou & Shiau (1998) extend this work, introducing the use of infinite elements in order to reduce computing demands.

Simic & French (1998) used a 3D analysis of an underground station box, formed in diaphragm walls, to seek savings in reinforcement when comparing results with plane strain analysis. They concluded that steel quantities could be reduced by about 25% overall for the project they studied, mainly because the walls near the corners of the excavation were computed to be less heavily loaded. Lee *et al* (1998) also compared results at midsides and near corners of a basement, showing that the relative difference depended on features such as the stiffness of propping systems and the depth below the excavation to relatively rigid strata.

To date, numerical analyses have been used to study details of behaviour, or to predict the behaviour of complete structures. It is hoped that they may be used further in future to help resolve some of the outstanding issues in the codes which were discussed above.

5 THE FUTURE

Developments in the foreseeable future may be considered in two groups: (a) developments of codes and standards, and (b) derivation of new technical information.

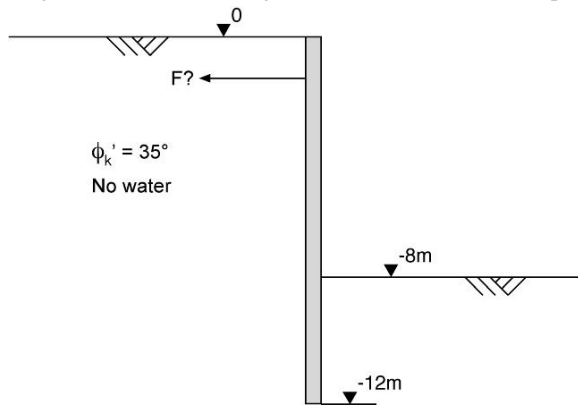
5.1 Codes and standards

In general, the development of codes and standards is based on reconsideration of existing information, much of which has been available for some time. Forthcoming documents include an English translation and revision of the German Recommendations of the Committee for Excavations, EAB-100; a replacement for Report 104 of the UK Construction Industry Research and Information Association, which is to have a particular emphasis on improving economy in design; and the formal "EN" version of Eurocode 7 Part 1, EN1997-1, which is scheduled for publication in 2002 and will be discussed further here.

In relation to embedded walls, the major changes from the ENV (ie EC7 1995) to EN version of Eurocode 7 will be (a) a less prescriptive approach to overdig (BS8002 plans a similar change), and (b) the introduction of two further sets of partial factors, to provide alternative approaches to that of the ENV. The first of these approaches is based on carrying out calculations using characteristic values of loads and material properties. Safety factors are then applied at a later stage in the process to action effects, as opposed to actions, and resistances, as opposed to material strengths. In this context resistances are quantities such as bearing capacity or passive force or earth pressure, while action effects would be computed bending moments, prop forces, etc.

For many situations factors can be tuned to give similar results wherever they are applied in the calculation procedure. However, the practice of calculating characteristic action effects, then factoring these, can lead to unsafe situations in some cases. This was the essential problem with the net pressure method described above, though this method has not been proposed for use with Eurocode. The problems might be overcome by introducing additional rules, externally to the Eurocode, but in the absence of these, the situation shown in Figure 31, in which a sheet pile retaining wall is required, is considered to be problematic.

Using unfactored soil strengths, a cantilever wall could pro-



vide equilibrium with a length of 11.6m. However, any system of factoring would indicate that it requires a longer length to work safely as a cantilever. For example, the ENV approach would give a length of 14m for a cantilever, as shown in Figure 32. Now suppose that, for reasons external to the design calculations, it is decided that the length of the wall will be 12m and further safety is to be provided by an anchor acting at 1m from the top of the sheet pile as shown in Figure 31. This could occur because sheet piles of 12m length are readily available, or possibly because the sheet pile wall is already in place when the required depth of excavation in front of it becomes known. The design requirements now are to check that the wall is sufficiently stable with a tie at 1m depth, and to find the required design resistance for the tie. For economy, the designer wants to adopt the minimum allowable design tie force.

Relevant calculations are summarised in Table 3. Calculations to Case C confirm that a length of 11.9m will be sufficient provided the tie has a design resistance of 75 kN/m, as shown in Figure 33. The calculation for Case B is less severe, so Case C determines the design. Since a wall length of 11.6m was sufficient to give equilibrium as a cantilever using unfactored characteristic soil properties, the minimum tie force calculated for a 12m length using characteristic properties is 0.0, as shown in Figure 34. Hence an approach which calculates this characteristic action effect, then applies a factor to it, will require a minimum design tie force of 0. It is important to place the factors near the source of the uncertainty they represent; factors applied to the action effects, such as the tie force in this case, come too late in the calculation.

Table 3. Summary of calculations for tied retaining wall.

	Case C without anchor	Case C with anchor	Characteristic state
γ kN/m ³	17	17	17
ϕ'_k °	35	35	35
γ_ϕ	1.25	1.25	1.0
ϕ'_d °	29.3	29.3	35
δ/ϕ' active	$\frac{2}{3}$	$\frac{2}{3}$	$\frac{2}{3}$
δ/ϕ' passive	-1	-1	-1
K_{ad}	0.29	0.29	0.22
K_{pd}	5.4	5.4	8.35
Design anchor force kN/m	-	75	0
Length m	13.93	11.86	11.56
Data	B-2CC	B-2CP	B-1

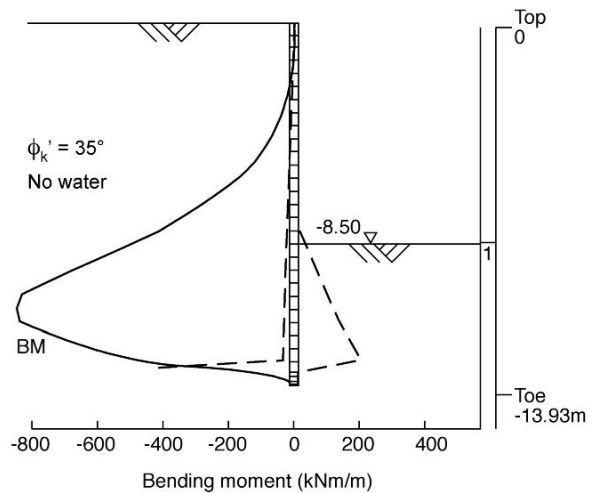
Figure 31. Situation for which a retaining wall is required.

Figure 32. Cantilever: ENV approach.

Figure 33. Tied retaining wall: ENV approach.

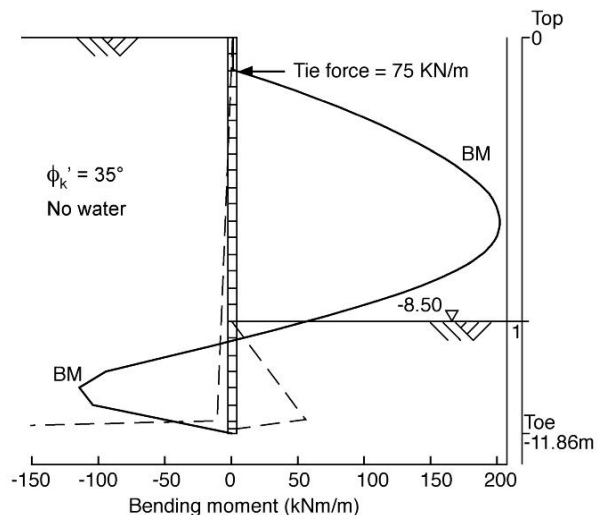
The second alternative approach to be included in EN1997-1 essentially has the ENV factors of Cases B and C used in combination. This will inevitably lead to more conservative design.

Figure 34. Tied retaining wall, characteristic state.



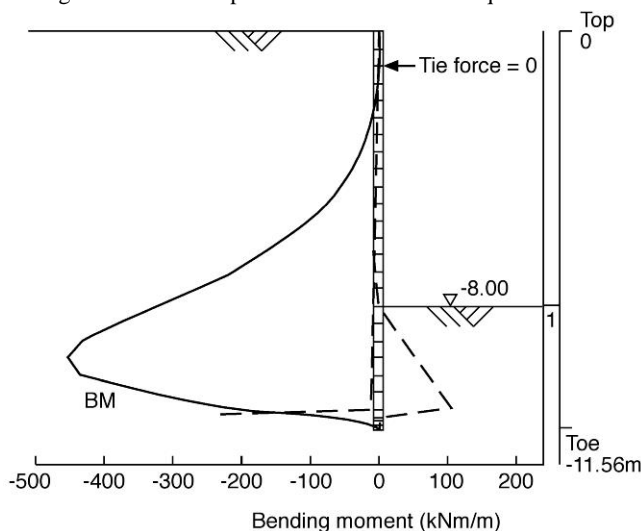
5.2 New technical information

This paper has noted that many features of the behaviour of em-



bedded retaining walls are still poorly understood, for both

working states and collapse states. Modern techniques could be



used with advantage to revisit some of these problems. New technical information can be expected from three main sources: (a) better numerical analysis, (b) results of field monitoring, and (c) new modelling data, possibly from centrifuge testing.

Increasing computing power now makes it possible to carry out 2D analyses, using relatively sophisticated soil models, in a matter of minutes. 3D analyses are also viable in the design of more major structures, but take a little longer and require a greater degree of expertise. Both of these sources inform designers about likely modes of behaviour and may help to rule out some of the more unlikely suggestions, narrowing down the options to be considered both in design and code drafting. Also, three dimensional numerical analysis can be used to investigate in general terms three dimensional effects, e.g. due to excavation or prop removal in bays or sections along a retaining wall or corners in near-square excavations. Once quantified, three dimensional effects can be used with more confidence to achieve an economic construction sequence and overall structure.

To date, and particularly in the period 1960 to 1990, field monitoring added greatly to knowledge of ground movements associated with retaining structures, and it was also possible to obtain some information on earth pressures and structural effects. It is vital to renew momentum in this monitoring process if knowledge is to develop. In the UK, a renewed interest in the Observational Method, including the publication of a CIRIA report by Nicholson et al (1999), seems to be encouraging more monitoring because clients see a direct benefit from the measurements taken. The use of data loggers to obtain continuous records of prop loads, and earth/pore water pressures is improving our knowledge of construction sequence and transient effects on the overall performance of retaining walls. New techniques such as use of optical fibres and possibly satellite positioning should add to the possibilities of accurate measurement achieved economically. Direct measurement of earth pressures remains problematic, however.

Much was learnt from the laboratory modelling of Terzaghi, Tschebotarioff, Rowe and others including, more recently, Bolton et al (1990a,b). Modern techniques, especially using the centrifuge, give an opportunity to re-visit some of this work and perhaps to resolve more of the outstanding questions.

6 CONCLUDING REMARKS

Embedded retaining walls have had increasing use through the twentieth century, with large concrete walls becoming very important to the development of deep basements and underground infrastructure. Many design guides and codes have been drafted, and research and debate have improved understanding of both

performance and design requirements. Nevertheless, debate con-

tinues.

The redistribution of earth pressures caused by wall flexure has been particularly contentious and still merits further research. Several of the newer codes of practice or design guides allow some reduction of bending moment related to redistribution, including some degree of arching whereby active pressures are reduced between stiff support points. It is noted in this paper that some of the early dispute about this related to deadman anchors, which tend to displace under load. Stiff props displace much less, whilst prestressed anchors give guaranteed forces which can largely dictate the equilibrium earth pressure distribution. Hence allowance for a degree of arching seems appropriate to these more modern forms of wall support.

The question of redistribution is also related to the choice between design based on analysis of working or collapse states. As collapse approaches, whether due to rotation or bending failure, any form of support which does not itself fail is effectively rigid compared to the collapsing wall. Hence in a collapse analysis it is appropriate to allow for arching.

The application of factors of safety is also still in dispute. This paper has argued that safety factors can most generally and most usefully be applied to soil strength, rather than to passive resistance, rotational moments, or structural load effects such as bending moments and prop forces. The factors allow for uncertainties in soil properties and a necessary limitation on the degree to which soil strength is mobilised in the working state. They must also provide a margin for minor errors, which are inevitable both in design and construction. In general terms, the approach of Eurocode 7 ENV1997-1, in which two "cases", or sets of partial factors, are analysed, is preferred.

It is important that codes convey to designers what degree of conservatism they are to adopt in assessing parameter values. The use of a "double check" on a parameter value provides extra assurance with negligible extra effort on the part of the designer.

Whatever procedures are used for basic design calculations, it remains essential for the designer of embedded walls to understand their behaviour, the ground movements associated with them, the significance of construction sequence and procedures, and details such as temperature effects on struts and behaviour of berms. Some recent work on these topics has been summarised.

It is hoped that further investigation, using advanced numerical analysis, physical models and especially field monitoring will clarify the issues that remain and so enable design procedures to be unified on a more widely accepted basis of understanding.

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