Codes, Standards and Design Guides

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BACKGROUND TO CODES IN THE UK: CP2

The Civil Engineering Code of Practice CP2 Earth Retaining Structures was published in 1951 by a Joint Committee of the Institutions of Civil, Municipal, Water and Structural Engineers. Code writing then passed to BSI where revision of CP2 has been under way for about twenty years. Some delay was caused by the apparent conflict between traditional "factor of safety" or "permissible stress" methods of evaluating designs and the new "limit state" approach advocated by structural committees, and widely accepted in Europe.

The old CP2 served simultaneously as a design standard, a guide to calculation procedures, and a record of what was considered good practice in detailing and construction. This paper will concentrate on essential design standards.

It is first necessary to recall the mixture of safety provisions which were used in CP2. With regard to the possibility of a deep slip circle carrying away the whole facility, there was to be a factor of safety of at least 1.25, defined as restoring moment divided by overturning moment. For embedded walls, there was to be an arbitrary reduction factor of 2 applied only to passive resistances. This is tantamount to reducing both the self-weight and soil strength components of passive resistance, while maintaining full values on the retained side. This was shown by Burland et al (1981) to be illogical and, for the long-term analysis of walls in clay, impossible to implement. For gravity walls on spread bases a safety factor of 2 was to be applied to sliding resistance, but otherwise the approach was based on permissible states. The middle third rule was to be applied to the eccentricity of the base thrust, while the maximum bearing pressure at the toe was to be shown to be smaller than the allowable bearing capacity, which was not defined.

LIMIT STATE DESIGN: THE ANALYSIS OF CRITICAL EVENTS

Bolton (1989) advocated the replacement of arbitrary rules and definitions of "factor of safety" by the introduction of two concepts compatible with limit state design methodology as it was emerging through the Eurocode process. *Design situations* are those combinations of harsh environmental circumstances, severe loads, and pessimistic soil-structure characteristics, which the facility is intended to survive. *Limit modes* are the independent ways in which the new facility (or a neighbouring facility or service) might come to grief in any one of those situations, distinguishing between ultimate limit states (uls) which endanger people and serviceability limit states (sls) of excessive deformation affecting appearance, efficiency, cost or durability.

At the core of limit state design evaluation is the explicit checking of the various critical events - each of the possible limit modes in each of the design situations. Analysis of structures in normal working states is to be avoided. These precepts are now adopted in both the current draft of Eurocode 7 Geotechnics for the EC Standards organisation CEN, and in the current draft of BS8002 (revising CP2) for BSI.

Setting standards

The objective description of design situations is a feature of the draft BS8002 not coherently covered in CP2. Each design situation should be sufficiently severe, and the collection sufficiently varied, to encompass all foreseeable conditions during construction, normal operation, and foreseeable classes of accidental loading. Examples of recommended conditions are listed below.

a) Environment. Groundwater levels should be the most onerous that are considered reasonable, taking into account storms, seasonal changes, trends, or accidental leakages. Allowance will be made for the adverse effects of upward hydraulic gradients in the passive zones beneath excavations.

b) Geometry. Provision should be made for over-dig in excavations, no less than the larger of 0.5m, or 10% of the vertical span concerned. Provision should also be made for a service trench in front of walls, which could eliminate the passive zone in front of gravity walls, for example.

c) Unit weight. A cautious estimate of soil unit weight shall be made.

d) Loads. Surcharge will be not less than 10kPa overall. The transient or dynamic effect of live loads must also be taken into account: it may be noted that the average ground pressure beneath the multi-axle bogey of a special transporter unit can be in the range 50 to 100kPa over an area circa 3m by 4m.

e) Soil conditions. Water-filled tension cracks will be invoked at a retained face in fine-grained soils, where the possibility exists of a significant inflow due to existing perched water, burst pipes, or storms.

Evaluation of both stability and deformation

Although the principle of limit state evaluation of critical events is now very widely accepted, the exact means of selecting critical parameters has been much more contentious. Structural code writers have generally quoted "characteristic values" for loads and strengths, and then specified "partial factors" on all these parameters so that design values could be derived automatically by division or multiplication. Bolton (1981) argued that partial factors should be applied only to items where statistics can be applied to allow for variation within some experimentally determined frequency distribution. Geotechnical engineers recognise the overwhelming importance of inferences deduced from an experienced interpretation of site investigation data available only to the engineer on the job, not to the code-writers. Critical values of soil parameters should therefore be selected by the designer, under advice from the code. A specimen code for bridge abutments on spread bases was produced for TRL on this basis: Bolton (1991a).

Soil parameters should, in any event, be relevant to the limit state under consideration. Different parameters should be required for deformation (sls) checks and collapse (uls) checks. In the past, textbooks usually implied that equivalent elastic stiffness moduli should be used for deformation calculations while plastic strength parameters were to be used for collapse checks. Most practising engineers have, on the other hand, used a "safety factor" against collapse to guard against excessive shear deformation during construction, and have followed this up with a separate calculation of settlements due to consolidation. It is proposed below that a modification of this latter approach is actually more consistent with research evidence of deformations, and less subject to errors of judgement in selecting parameter values, than the "elastic" approach.

CP2 placed all its emphasis on stability (uls) calculations. Figure 1 shows two different episodes of embankment construction on soft clay, observed in centrifuge model tests (Sharma, 1992). The collapse event in Fig 1(b) is rather similar to the quay wall failure at Gothenburg Harbour, discussed in CP2 as a global slip circle failure. This class of event was to be prevented by the use of a factor on undrained soil strength of 1.25. But the existence of a small "factor of safety" in Fig 1(a) clarifies that as collapse is approached shear strains can accumulate to produce significant surface displacements (exaggerated in the figure). If the depth of soft soil affected by the plastic mechanism is D, and the mean shear strain mobilized within the mechanism is γ_{mob}, the horizontal surface displacement δ will be $\gamma_{mob}D$.

The following lessons must be appreciated:

- deformation mechanisms prior to collapse mobilize shear on the same family of lines as potential ultimate collapse mechanisms;
- whereas collapse involves a monolithic rotation, shear deformation can be characterized by a transition from far-field displacements which may be negligible to near field displacements which increase towards the ground surface;
- the line of demarcation between the far-field and near-field zones of pre-failure deformation coincides approximately with the ultimate slip surface;
- the magnitude of pre-failure deformation is proportional to the degree of shear strain permitted, which is easily related to the degree to which the strength of the soil is mobilized in an appropriate stress-strain test.

Strength reduction factors

A mobilization factor M (defined as a reduction factor on peak strength) of about 1.5 is required to limit the undrained mobilization of shear strain to below 1% in a range of typical clays; M = 3 would be required to limit shear strains to about 0.25%. If the depth of the near-field plastic zone is 5 to 10m, M = 1.5 would imply a displacement $\delta \approx$ 50 to 100mm. It is unlikely that a larger displacement would prove tolerable for an embankment carrying a structure such as a quay wall. A larger value M = 3 might reduce lateral displacements to $\delta \approx$ 12 to 25mm.

The confusion engendered by the terms "lump factor of safety" and "partial factor on soil strength" can now, perhaps, be dispelled. The CP2 "lump safety factor" of 1.25

might best be regarded as a factor of safety on soil strength. It is likely that a value of 1.25 will be required to assure against collapse, considering the variability of the deposit, and dependence of undrained strength on the mode of deformation. The use of $F = 1.25$ on a slip circle might, in a limit state code, be described as a partial factor $\gamma_m = 1.25$ to be applied to some carefully defined characteristic element strength in order to derive a design value capable of assuring sufficiently against collapse. However, soil is a compliant material, and a much higher strength-reduction factor M between 1.5 and 3 is required to assure serviceability on undrained clays. Serviceability is evidently the critical design consideration.

Terminology can be important. The old British practice of calling the typical strength-reduction factor of 1.5 on earthworks and 3 on more sensitive foundations a "factor of safety" is confusing, because safety is not the issue which has demanded factors as large as these. A lawyer who could show that the true strength-reduction factor was 1.3 not 1.5, or 2 not 3, could presently argue that safety was compromised whereas the real threat would be to serviceability. The new European practice of achieving the same reduction factor through the successive application of a chain of "partial factors" will be equally confusing if it is not made clear that the threatened limit state is sls not uls. It is the certainty of deformation, not the uncertainty of strength, which generally controls the value of soil strength mobilized in geotechnical design. The reduction factor M should therefore be called a "mobilization factor" as recommended in the current draft of BS8002.

SOIL STRENGTH

CP2 (1951) offers the reader the opportunity to reflect on the slow rate of diffusion of fundamental ideas from the universities into engineering practice, especially in its treatment of the strength of soil. It invoked cohesionless soils (sands etc) which satisfied a simple constant-ϕ failure criterion, and cohesive soils (clays) which ideally possessed a constant cohesion. Most soils, and certainly silts, were thought of as intermediate between the two extremes, possessing both c and ϕ at failure on a Mohr-Coulomb diagram of total stress. The principle of effective stress had then been understood for twenty years, and the textbooks of Terzaghi and Taylor were in the bookshops. It should therefore have been possible in 1951 to explain the transition of fine grained soils from apparently cohesive ($\tau = c_u$) at constant water content during construction, to apparently frictional ($\tau = \sigma' \tan\phi'$) in the long term, as excess pore pressures dissipated.

However, the principle of effective stress was missing from the treatment of clay in CP2, probably because its authors were uncomfortable about transcending current practice. Textbooks teaching that soil strength could be written in total stress terms $\tau = c + \sigma\tan\phi$ were still being produced twenty years later, possibly because some university teachers were uncomfortable about transcending current codes. Stiff fissured clays were thought troublesome because it was necessary to estimate their softened strength: some values were quoted without much conviction. Fortunately, CP2 also introduced a long-stop in the form of a rule that active earth pressure in stiff clays should never be taken to be less than that which would be exerted by an equivalent fluid of density $30 lb/ft^3$. This has much the same effect as specifying an active earth pressure coefficient of 0.25 with zero pore pressures, a situation which would obtain with under-drained clayey silt attaining $\phi' \approx 32°$ in a "fully softened" condition according to effective stress analysis.

BS8002 will emphasise the use of effective stress analysis - with an appropriate value of ϕ' and carefully estimated pore pressures - irrespective of soil type, whilst permitting undrained strength calculations for fine-grained soils of low mass permeability prior to any pore water equilibration.

Strength parameters

It is necessary to distinguish between peak strength and ultimate strength, as shown in the effective strength envelopes drawn in Fig 2a. Peak strength is important because it terminates the rising branch of the stress-strain curve, upon which the normal working states of the soil should reside. It can therefore act as an asymptote or delimiter to states of acceptable deformation. Ultimate strength is important because it will be engendered during the large deformations associated with collapse. Here "ultimate" must be taken to imply the strength measurable after indefinitely large shear strains. Most soils deform uniformly up to a brittle peak and then strain soften in shear bands, about 5 to 10 particle diameters thick, as they dilate to a "critical state" in which further shear causes no further change of density. Soils with a predominant clay fraction exhibit a further phenomenon allied to mechanical polishing: the strength can drop even further to "residual" values on slip surfaces which continue shearing.

The successful back-analysis of large deformations at collapse in the field, or in centrifuge model tests, invariably demands the use of ultimate soil strengths. Because residual strengths require large relative displacements across shear bands, the problem effectively falls in two parts: recognizing pre-existing slip surfaces where they exist, and specifying critical state parameters for all other uls checks. If there is then an attempt to restrict soil states to the pre-peak condition, it is generally held that residual slip surfaces will not initiate. A sufficiently close estimate of ϕ_{crit} can generally be found from the ultimate states of triaxial or shear box tests carried out on saturated samples reconstituted to be loose (if granular) or normally consolidated (if clayey). Reasonable estimates may also be obtained from the effective angle of shearing ultimately developed on rupture surfaces observed in shear tests on in-tact material.

While ϕ_{crit} controls the effective stress analysis of collapse, the role of the peak strength, described in Fig 2b either with a secant angle of shearing ϕ_{max} or a (c',ϕ') tangent envelope, is to act as a limit to normal working states. Although practitioners have become used to the tangent description of peak strength, the strong curvature of the envelope near the origin makes the secant definition much more convenient since the variation of only one parameter, ϕ_{max}, has to be accounted for. Furthermore, secant ϕ_{max} is a fundamental parameter related to the relative density and crushability of the aggregate, and is predictable from soil classification and relative density alone: Bolton (1986).

Where total stress analysis is admissible, with the short-term undrained strength of saturated and relatively impermeable plastic soils, both the peak and ultimate strengths $(c_u)_{max}$, $(c_u)_{ult}$ should be determined. Here, the ultimate strength can best be defined as the peak strength of the clay remoulded at its in-situ water content. There are severe difficulties in selecting an appropriate ultimate strength for some soils which are highly sensitive, brittle, or cemented, and with cyclic loading events such as earthquakes which cause pore pressures in low plasticity soils to rise much higher than in monotonic undrained shear tests and which therefore cause a "degradation" in undrained strength. In any such case, expert geotechnical advice must be sought. Otherwise $(c_u)_{ult}$ should be used in collapse calculations, and $(c_u)_{max}$ can serve in serviceability checks following the strength-reduction approach.

Following the independent assessment of the peak and ultimate strengths of various soil elements, there must follow a process of decision-making. First, where a variety of values has been inferred for a particular region which is to be treated analytically as homogeneous, representative values of both peak and ultimate strength must be derived. Representative values should be cautious estimates of values which might govern performance in the field in a limit state. Usually, there will be little or no uncertainty in ϕ_{crit} governing collapse. Uncertainty in ϕ_{max} will be greater due to variations in density and stress level: the worst credible value will pertain to a representative element with least possible dilatancy. A conservative bound will be provided by simultaneously inferring the lowest relative density of sands, or the lowest preconsolidation of clays, together with the highest stress level expected in the region of interest.

More difficulty may be experienced in selecting representative undrained strengths. Excess pore pressures at peak and ultimate are a great deal more variable than the underlying effective stress parameters. However, for soils which are not especially sensitive or brittle it has been demonstrated earlier that deformation control is likely to be more critical than assurance against collapse. A conservative lower bound to the population of individual peak strength values could be used as the definition of representative value. It may, however, be possible for the experienced geotechnical engineer to discount certain low values which might be due to sample disturbance or swelling for example.

Mobilizeable strength

The current draft of BS8002 advises on the selection of a single design value of strength τ_{mob} which will be mobilizeable under all foreseeable conditions, taking both safety and serviceability into account. This requires two inequalities: $\tau_{mob} \leq \tau_{ult}$; $\tau_{mob} \leq \tau_{max}/M$ where M is the specified mobilization factor to assure acceptable deformations. In total stress analysis M is set to 1.5 for isolated retaining walls, while in effective stress analysis M is set at 1.2. The intention of these values for mobilization factor M is that shear strains would be less than 1%, so that $\delta/H \leq 0.5\%$ in service, if the soil is not classified as soft or loose. Otherwise, stress-strain data would be required from stress-path triaxial or pressuremeter tests so that higher values of M could be chosen to achieve the required limit on strain.

The two possible cases of (a) deformation-limited and (b) strength-limited design values are shown in Fig 3. Each is intended to be conservative, and a more accurate value might be derived by advanced techniques of investigation, testing and analysis if the project warranted it. In (a) the conservatism comes from ignoring the possibility of favourable preloading, which might reduce the strains necessary to mobilize any given strength, compared with those offered in the virgin loading of triaxial tests. This is identical to the use of the "proof stress" (the stress at which the plastic strain on first loading exceeds a given value, eg 0.2%) in aluminium alloy structures. In (b) the conservatism comes from denying the possibility that states such as B are permissible. Any pre-existing softened zone, or unforeseen accident, could lead to progressive failure with the strength of the soil dropping below that required at B to maintain static equilibrium. Such an incident would lead to catastrophic dynamic collapse. Engineers generally go to some trouble to avoid the consequences of brittle failure, ignoring the upper yield point of low-carbon steels in favour of the long plateau of the lower yield strength, and then ignoring subsequent strain hardening on the grounds that brittle fracture would be too close for comfort if design relied on any plastic strength other than the lowest.

The outcome is a "permissible stress" approach to limit state design satisfying the partial factor format but never departing from an objective description of soil behaviour.

Shear strength of an interface

The available shear strength at the interface between the structure and the soil can best be determined by direct shear tests in which a sample of the steel or concrete material fills the bottom half of a shear box, and the appropriate soil is placed above and forced to slide over it. Where the surface texture of the structure is rougher than the d_{50} size of the soil, the interface will mimic the parent soil except that sliding at critical states is very easily induced: for these "rough" surfaces ϕ_{crit} for the parent soil may be used as the representative value in default of experimental measurements. Surfaces which are "smooth" relative to d_{50} may give lower values of ϕ dependent on the materials, which would then require measurement.

In total stress analysis, the shear strength of an interface may be affected both by friction and pore pressure deviations. Regarding friction alone, the representative undrained strength for sliding of a "rough" surface might be estimated as $(c_u)_{ult}$ for the parent clay, on the same grounds as before. However, where porous permeable materials are placed as drains between the structure and a clay soil, the undrained interface strength will *never* be relevant. The total normal stress σ in the clay will then be transformed into an effective normal stress at the interface, so that the shear strength becomes the lower of $\sigma \tan\phi_{crit}$ (for shear in the clay close to the drain) and c_u (for shear further from the drain). For all except the softest clays the former condition would be critical, and the equivalent α-value (former/latter) could be significantly less than unity.

It would clearly be impermissible to attempt to mobilize an interface strength either greater than the representative strength of "smooth" surfaces, or greater than τ_{mob} in the neighbouring soil. A design limit $(\tau_{mob})_{wall} = 0.75(\tau_{mob})_{soil}$ can be set for wall faces to allow for the possibility that the wall is not aligned with the planes of shear in the soil. Vertical equilibrium must be demonstrated in every case.

UK designers should note that taking $(\tau_{mob})_{soil} = \tau_{max}/1.2$ for granular soils, with $(\tau_{mob})_{wall} = 0.75(\tau_{mob})_{soil}$, gives a total reduction factor $(\tau_{mob})_{wall}/(\tau_{max})_{soil} = 0.63$. The alternative inequalities set out above to limit $(\tau_{mob})_{soil}$ will result in a somewhat smaller (variable) ratio. These new wall friction factors are similar in magnitude to those in current use, but are now expressed in objective terms.

METHOD OF EARTH PRESSURE ANALYSIS

The outstanding advantage of the concept of mobilizeable strength is that the usual plastic methods of analysis are available, except that no further "safety factor" is to be used. The reduced strength τ_{mob} can be expressed in terms of ϕ_{mob} or c_{mob} as appropriate. Mobilizeable active and passive pressures are then calculated as a function of mobilizeable soil strength and wall friction. In effective stress analyses, earth pressure coefficients K_{mob} are the usual functions of ϕ_{mob} and wall friction angle δ_{mob}. In total stress analyses, the limiting earth pressures (taking safety and serviceability into account) are functions of c_{mob} and α.

Convenience of analytical techniques

The theory of plasticity has been much extended in its application to soil mechanics problems since 1951. Whereas CP2 specified earth pressure coefficients based on Coulomb's assumption of sliding wedges, the draft EC7 contains explicit formulae based on the method of characteristics, which satisfies equilibrium and obeys the strength limitation at every point in the soil, not just on an assumed failure surface. These new earth pressure theories offer safe bounds to the plastic solution if the values of soil strength and interface friction are themselves on the safe side. The degree of under-estimation which they offer is much smaller than the over-estimation caused by assuming planar slip surfaces, for example. They can also account for geometrical parameters such as sloping fill or battered walls.

Bolton (1991b) used similar techniques to derive thrust coefficients for the effect of load patches on the surface of retained granular fills. Effectively, Prandtl's bearing capacity solution was extended downwards and outwards to produce earth pressures on a nearby retaining wall. The earth pressures were then integrated to provide the magnitude, inclination and location of thrust on the wall.

Water-filled cracks

Confusion often occurs regarding the possibility of cracking in active zones of lateral spreading, and the thrust which can be exerted on walls retaining clay if such cracks initiate and fill with water. Although CP2 warned of the effect, the detailed advice it gave is typical of the worst sort of code of practice, comprising a mis-understanding of the mechanism, an over-elaboration of a completely mistaken formula, and an overlay of arbitrary conditions.

First, it is necessary to recognize that the conditions for opening a crack are that tension is applied to a material which has finite shear strength but small tensile strength and small fracture toughness. These are fulfilled best on the interface between a wall and clay which it is retaining, where the wall is moving away from the soil. The interface has a much smaller fracture toughness than the parent soil, and offers the perfect crack-raiser, as described by Bolton and Powrie (1987). Second, it is necessary to appreciate that if a crack fills with water, the pressure can extend the crack beyond the "dry" depth $2c_u/\gamma$ down to $z_w = 2c_u/(\gamma-\gamma_w)$. The hydrostatic pressure $\gamma_w z$ replaces any active soil pressure, and tends to push the wall outwards. The water follows the wall out, keeping the crack full if there is sufficient inflow. Below depth z_w, the "active" soil follows the wall and keeps the crack shut, whereas above that depth the soil is supported by the water pressure in the crack.

The practical consequence of this is that in many overconsolidated clays the maximum possible depth of a flooded crack will exceed the depth of the wall. A crack will then open down to the point of zero lateral displacement, which is close to the foot of a rigid cantilever. Bolton et al (1989) documented centrifuge tests and charted the strength mobilized in a variety of cases. It is simply necessary to make a judgement of whether free water could be present to flood the crack at the wall/clay interface.

GEO-STRUCTURAL MECHANISMS

Plastic soil deformations around a stiff cantilever

Fig 4 shows an idealization for uniform soil shear strains around a free embedded cantilever wall, consistent with the usual assumption of uniform soil strengths. Wall rotation θ is shown to relate to mobilized soil shear strain $\gamma_{mob} = 2\theta$. The use of a mobilization factor $M = 1.2$ on $\tan\phi_{max}$ or 1.5 on $(c_u)_{max}$ is intended to restrict shear strains to within 1%, so that $\theta \leq 0.5\%$. If some different limiting displacement were desired, or if good quality stress-strain data were available, the factor M could be altered to suit.

Whatever value of ϕ_{mob} or c_{mob} is selected, the plastic equilibrium of the wall in terms of horizontal forces and moments provides the two conditions necessary to determine the point of rotation and the required length of the (assumed rigid) wall. This approach was used by Bolton et al (1989) to calculate and chart the minimum permissible lengths of a stiff wall consistent with the specified displacement criterion, under various soil conditions.

Walls rotating about props at their crest can be treated in the same fashion. A compatibility condition $\gamma_{mob} = 2\theta$ is maintained only when a frictionless horizontal surface is introduced at the foot of the wall: otherwise, an additional strain concentration propagates from the foot. In general, the region of soil immediately surrounding the wall mobilizes $\gamma_{mob} \geq 2\theta$, so that the assumption of equality causes the engineer safely to underestimate the degree to which soil shear strain (and therefore strength) will be mobilized.

Structural serviceability for a stiff cantilever

Design bending moments, shear forces and prop forces can be derived by equilibrium calculations from the linear design earth pressure distributions of the type shown in Fig 4, with c_{mob} or with ϕ_{mob} and pore water pressures. If the soil around the wall starts with zero mobilized strength (initial earth pressure coefficient $K_i = 1$), and if the same degree of shear strain is mobilized in both active and passive zones due to wall movement, the same strengths should be mobilized. This is equivalent to assuming a uniform mobilization factor $M_a = M_p = M_{target}$, 1.2 or 1.5 as appropriate.

Higher bending moments will be induced if it then transpires that $M_p < M_{target}$ or $M_a > M_{target}$ so that $K_{p,mob} \rightarrow K_{p,max}$ and $K_{a,mob} \rightarrow 1$. The two conditions of higher earth pressure either side of the wall inevitably go together since equilibrium demands they balance. This can happen where there is a high value of K_i, such as in heavily overconsolidated soil, or where clay swells on the passive side due to excavation. It can also occur where compaction increases soil pressures in retained fill, or where large prop forces are jacked into place.

In reality, the mobilized strength around a wall will vary with position and time in a complex way. The objective of the designer is not to predict such variations but to allow for their extreme values. The consequence of failing to predict enhanced lateral pressure may be the premature yield or cracking of the wall. If the wall material is ductile, local yielding will relieve pressures by creating wall movements sufficient to achieve target mobilizations of soil strength.

It follows that steel will present few problems. The wall could never collapse if it has been designed properly to a target value of M, nor can it rotate significantly more than the target value of θ (eg 0.5%). It may, however, have yielded unexpectedly at some locations. Similar problems afflict steel frames due to lack of fit in construction: steel designers generally ignore such self-stressing effects. In the design of steel retaining walls, mobilizeable earth pressure coefficients can be used in any arbitrary pattern, so long as the conditions of overall equilibrium are met.

Reinforced concrete retaining walls may be more troublesome if steel yielding causes concrete to crack which in turn hastens corrosion of the steel. It would be sensible, for example, to assume for the purposes of structural design that diaphragm or secant-pile walls in stiff overconsolidated clays ultimately develop $M_p = 1$, or $K_{p,mob} \rightarrow K_{p,max}$ beneath the excavation. The resulting bending moments could hardly be exceeded unless some gross error had been made, such as in the groundwater pressure distribution. No further "load factor" need be applied, therefore, prior to section design.

Wall flexibility effect

The relative bending deflections need to be calculated from the assumed earth pressure distributions. If they exceed 0.5% (or the selected target strain limit) the previous use of a constant M factor with depth needs to be re-examined.

Three stages of increasing flexibility might be recognised.

i) Stiff wall - any tendency to bend does not greatly influence the strains in the surrounding soil. Simple triangular distributions of earth pressures are unaltered.

ii) Flexible cantilever - greater soil strains are induced near the crest, reducing local earth pressures and thereby bending moments. Target mobilization factor M can not be reduced and the serviceability limit can not be relaxed, so account can not be taken of this effect.

iii) Flexible sheet with redundant supports - arching of earth pressures on to the supported sections, and encastré at the base with "fixed-earth" support. This class of soil-structure interaction may benefit from numerical analysis, leading to economies in the provision of steel in sheet pile wall design following Rowe's method, for example. One aspect of this is considered below.

Flexible cantilever with single level of support

If a high-level prop is provided, the simple "statically determinate" calculation corresponds to "free-earth" support with a rotation about the prop. This sets the minimum permissible penetration and a conservative value for the required bending resistance. A greater than minimum length of penetration may be provided but it might not all be effective due to excessive flexibility. The effective extension will be that part which can develop significant net earth pressures, and which can therefore contribute extra resistance. Hetenyi (1946) developed solutions for the bending of a beam on springs in terms of the relative stiffness of the beam and the springs. The diffusion along the beam of the effect of an imposed load or displacement can be generalized in terms of a characteristic length, which in plane strain for a retaining wall embedded in an elastic medium might be written $\lambda = (EI_{wall}/E_{soil})^{1/3}$. An extra penetration of up to λ beyond the "statically determinate" length may be regarded as simply extending the "free earth" solution, and therefore having little effect on the required bending resistance. An extension of 3λ, however, may be taken to provide full moment fixity and "fixed earth" support. Deeper penetrations will not be effective. A significant part of Rowe's moment reduction factor for walls with small EI values is captured by eliminating the wall below the effective depth, thereby mobilizing higher soil strengths, smaller active pressures, and larger passive pressures, over a smaller span.

CONCLUSIONS

New "limit state" codes are being written which set objective performance criteria for the safety and serviceability of retaining walls and other geotechnical structures. Critical events are to be forestalled by direct evaluation of the performance of the structure in certain well-defined design situations.

Eurocode EC7 is a design standard, specifying what must be checked without prejudicing the method. By avoiding calculation details as much as possible, it aims to cover the whole scope of geotechnical design in a single slim document. This effort deserves to succeed, and the BSI committee have aimed to make the new BS8002 compatible with EC7.

The concept of mobilizeable soil strength, c_{mob} or ϕ_{mob}, simultaneously satisfying safety and serviceability, is proposed. Careful attention to the selection of a strength parameter can lead to simple, conventional calculations of plastic equilibrium. This should be equally acceptable to "lump factor" or "partial factor" advocates, and to those who prefer to be guided to the direct selection of a design value taking both stiffness and strength into account.

If a more detailed analysis is sought, the method can be broken down into its basic components of plastic equilibrium ($K_{mob} = F(\phi_{mob})$) and plastic kinematics ($\gamma_{mob} \approx 2\theta$), and it can then be enhanced through the use of high quality stress-strain tests ($\phi_{mob}) = f(\gamma_{mob})$. The term "geo-structural analysis" has been coined to cover this simplified treatment of soil as a jigsaw pattern of structural elements.

The rate of advance in soil mechanics theory and geotechnical practice has been rapid. New textbooks, advanced courses, and computer programs will continue to emerge, as will new technologies in earth retention. The old BSI system of code writing has given way to the new process of European standardization. If European geotechnical engineers wish to have committees winnowing new ideas and practices, their professional bodies should establish them.

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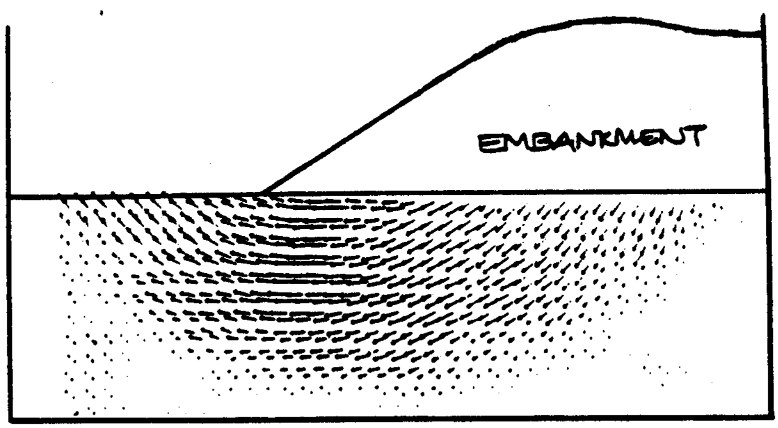
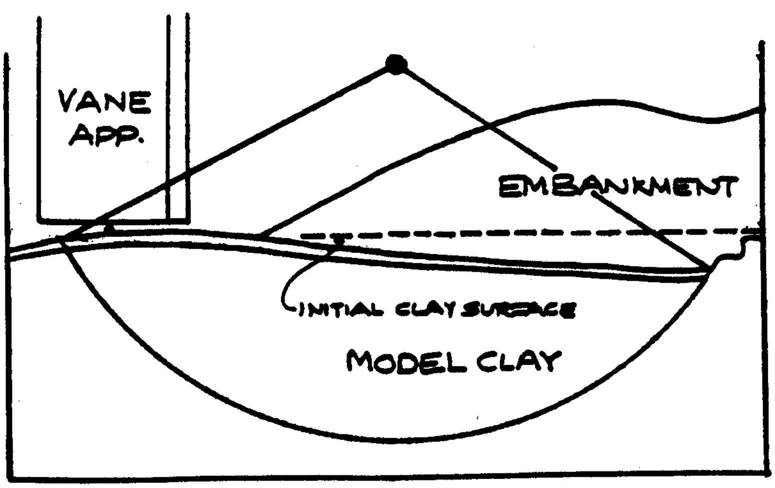
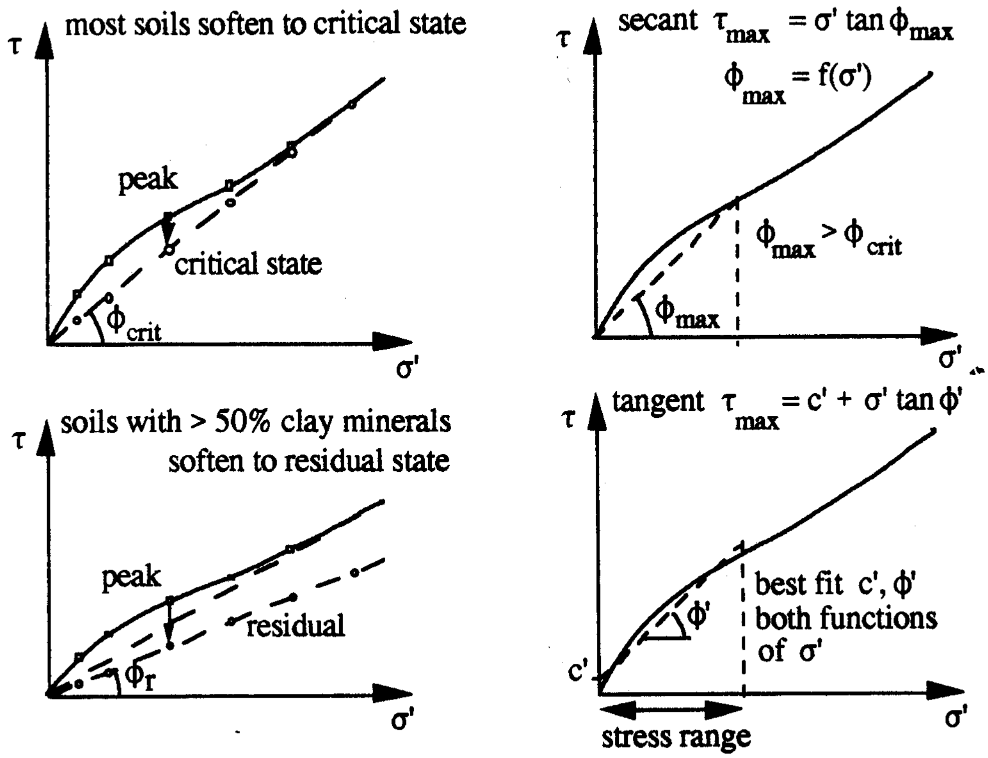


(a) Displacements increase smoothly towards clay surface, reaching circa 0.25 m prototype just prior to slip.



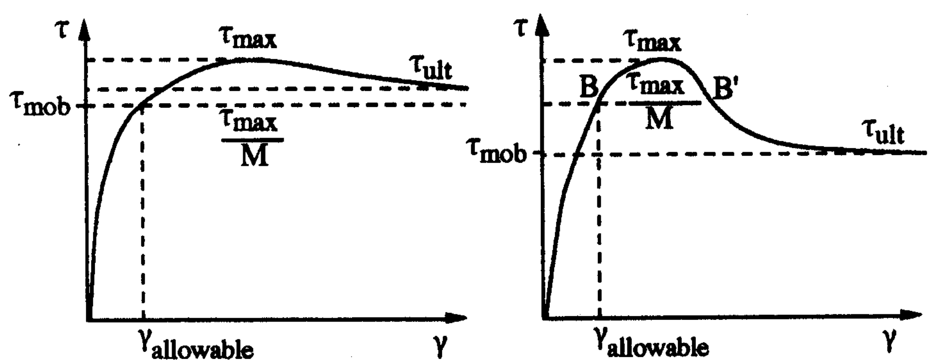
(b) Approximately circular slip surface, displacing circa 1.25 m beneath 6 m high embankment.

Fig 1 Centrifuge tests on embankments over soft clay



(a) peak and ultimate strength (b) secant and tangent parameters

Fig 2 Soil strength in terms of effective stress



(a) deformation limited (b) strength limited

Fig 3 Mobilizeable strength deduced from shear stress-strain curves

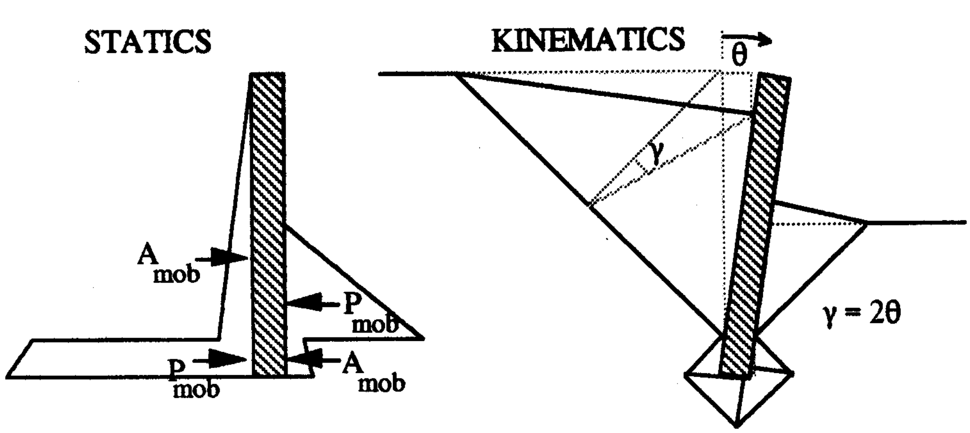


Fig 4 Geo-structural mechanism for embedded retaining wall