Wroth Memorial Symposium

DESIGN METHODS

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DEFINITION OF THE DESIGN PROCESS

We must begin, especially considering the man in whose memory this Symposium is held, by clarifying the question and defining our terms. To design may be to "indicate, draw, form a plan of, contrive, or intend" but most civil engineers are acutely aware of the difference between intentions, plans, and achievements. Executable plans need to be robust enough to cope with uncertainties and deviations, but sufficiently detailed to permit resources to be allocated to specific construction activities. For the purpose of this report "Design methods" will be taken to mean "Techniques contributing to the creation of executable plans".

Four components can be recognised in the creation of a design. Firstly there must be a clarification of the ultimate goal expressed in terms of specific performance requirements, secondly an attempt to synthesize a complete solution, thirdly an evaluation of whether the proposed solution would fully meet the requirements, and fourthly the revision and refinement of the proposal to obtain such compliance in an optimum fashion, such as at minimum cost. A possible fifth stage, which might be called "design as you go" or "the observational method", following Peck's Rankine Lecture, might involve the extension of the refinement phase to include construction control to ensure continuing compliance so that the goal is achieved in the face of all uncertainties and difficulties.

Not all ways of writing performance specifications, or of performing soil mechanics analyses, are equally useful in the creation of a geotechnical design. In particular, it will be suggested that the final phases of refinement and control have often been neglected in engineering – in teaching, research and practice – and that undue emphasis has been placed on analytical techniques which are too inflexible to be of much assistance in decision-making. Prediction (*pace* the organising committee) is not enough.

Performance Requirements

Many attributes of a constructed facility – size, function, appearance – are obvious or fall outside the focus of the geotechnical engineer. An objective statement of the geotechnical problem can be provided by a list of performance criteria. Such criteria can best be expressed in terms of the avoidance of any *critical event* comprising the activation of any *limit mode* in any of a specified set of *design situations*: Bolton (1991). The limit modes must cover the various independent mechanisms by which the facility could come to grief, while a small number of design situations are intended to be sufficiently onerous to encompass all the combinations of loads, environmental agencies, and soil-structure parameters which could occur during construction, normal working, and any foreseeable class of accident. Checks must be made of each limit mode in each design situation, unless some of these events can be seen to be inherently less critical than others. These dual definitions clarify the otherwise intractable problem of attempting to forestall every conceivable *limit state*, formed as a conjunction of every possible variable in some complex probability space.

Modern "limit state design" methods, as exemplified by the Eurocodes currently being drafted by CEN, incorporate distinct criteria for safety and for distortion expressed respectively in terms of the prevention of "ultimate" and "serviceability" limit states. Ultimate (or, more descriptively, collapse) limit states endanger people; unserviceability implies excessive distortion which compromises the efficiency or economy of the facility. Bolton (1989) proposes that just five independent limit modes be recognised in geotechnical design:

i) unserviceability arising through soil strain
e.g. differential settlement of a bridge abutment causing jamming of the deck bearings

ii) unserviceability arising through structural deformation
e.g. compaction of fill causing cracking in the concrete at the base of a retaining wall

iii) collapse arising through soil failure
e.g. catastrophic translation or rotation of a retaining wall as a monolith

iv) collapse arising through structural failure
e.g. collapse of a sheet-pile wall following progressive rupture of the anchors

v) collapse of a structure arising without soil failure
e.g. differential settlement of a bridge abutment causing the deck to fall off its ledge

Various design situations may need to be checked. Geometries, loads, water tables, and soil conditions, may each be a function of circumstances; consider over-dig during excavation in the construction phase of a retaining wall, the effects of abnormally heavy loads on the back-fill in service, or the raising of water levels due to the accidental bursting of a main. Clear statements of the standard combinations of loads and ground conditions have to be devised. Society may demand more severe situations to be checked for collapse than those used to evaluate serviceability. Designers may be asked to select "worst credible values" of loads and resistances in collapse checks, but be permitted to employ less severe assumptions in serviceability checks if there is a safe and economic remedy should the worst happen.

Pessimism over resistances should generally lead engineers to use critical state soil strengths rather than peak strengths in collapse analyses. Even smaller material resistances, mobilizeable at small strains, may have to be used in serviceability checks. For example, lateral earth pressure coefficients used in structural serviceability checks of retaining walls must logically be greater than the active pressure coefficients used to check against soil collapse, since the strength of the soil will not yet have been fully mobilized. A medium-dense granular fill behind a retaining wall might be capable of mobilizing a peak angle of shearing of 42° ($K_a = 0.20$), a critical state angle at large strains of 35° ($K_a = 0.27$), and a value consistent with acceptable displacements in an adjacent structure of only $\phi_{mob} = 33°$ ($K_a = 0.29$). Many engineers checking for cracking of a reinforced concrete retaining wall would use the peak angle to determine coefficient $K_o = 0.33$, even if they made no other allowance for locked-in stress due to compaction, which would be equivalent to mobilising the even more conservative value $\phi_{mob} = 30°$. It is therefore likely that the critical calculations for the integrity of structures are those to limit deformation and cracking in serviceability limit states.

Synthesis

Before any evaluation can be made, there must be something to evaluate. At any level of detail, the designer should perceive options and be prepared to compare their positive and negative attributes. Taking a geotechnical perspective, a road in cutting could be created with in situ techniques (diaphragm or bored-pile walls, steel sheet-pile walls, soil nailing system), or with a variety of back-filled structures constructed in front of temporary earth supports, or simply by designing stable side slopes using drains as required. An experienced engineer would have developed prejudices regarding many of the alternatives, and might make a selection based on a qualitative assessment of the local circumstances. If the decision on structural form is effectively final, any subsequent calculations will serve mainly as a check on reliability. If the designer is inexperienced, or the task is unusual, the detailed evaluation of a number of alternatives may be thought necessary to find the optimum.

If new technology becomes available, such as soil nailing or reinforcement, some additional incentive may be necessary before designers are prepared to educate themselves in its use. The relative infrequency of novel solutions in the UK compared with other countries suggests that such incentives are absent. Fee competition can hardly assist, since it draws attention away from the cost-effectiveness of the completed facility. Perhaps the current enthusiasm for design-build contracts is explained by the recognition by some clients that there needs to be an incentive to design efficiently.

The original synthesis of a complete proposal is clearly the sort of creative act classed by the dictionary as design, but any such conception then requires evaluation, revision and refinement if it is to attain specified performance targets. The writers of codes and standards tend to ignore the decision-making which leads to an original synthesis, and to describe as a "design method" the system of check calculations to which the proposal is then subjected. This can, perhaps, be traced from the schism between architects who propose and structural engineers who evaluate. The geotechnical designer needs to do both, and must therefore be capable of imagining working models of reality rather than simply a balance sheet of calculations.

Evaluation by Geo-Structural Analysis

The evaluation of the effects of forces on any deformable body is carried out with respect to three conditions: equilibrium, compatibility, and the material's stress-strain relations. Most soil mechanics evaluations are based on plasticity, where sufficient strain is invoked to permit every point within the zone of plastic deformation to develop its ultimate strength. This leaves the engineer to estimate an appropriate value for soil strength and imposed load, and then to solve for equilibrium alone. There are two fundamentally different methods of performing the calculations which offer upper and lower bounds to the theoretical collapse loads derived on the assumption that soil is ideally plastic and that the engineer's estimate of its strength is correct. Since they both rely on the structural engineers' approach of discretizing the continuum according to the function of the parts, and of assuming that the elements behave in certain simple ways, it may be appropriate to call this approach Geo-Structural Analysis.

An upper bound to the collapse load is provided by the kinematic method in which a collapse mechanism is assumed. Either global equilibrium, or an equivalent balance between work, potential energy and plastic dissipation, then guarantees that the soil body will find a way of collapsing at or before the calculated value of applied load. A lower bound to the collapse load is provided by the statical method in which equilibrium is shown to be satisfied at every point within the soil body without having to mobilize more than the plastic strength available at that point. These two approaches can best be appreciated if their results are expressed in terms of the plastic strength which needs to be mobilized in the soil body to carry the weights and external forces demanded by the design situation. Kinematic and statical methods then provide lower and upper bounds to the required strength, respectively.

A great deal of importance is often attached to the question of whether some plastic calculation method invokes a kinematically admissible mechanism which satisfies the plastic compatibility rules at every point, or a statically admissible stress field which can be extended to infinity or beyond the region of any possible collapse mechanism. Failure to observe these strict conditions leads to the method being described as a limit equilibrium analysis, the bounding nature of which is unknown. Of course, if the method is based on observed mechanisms of soil behaviour, the scope for error should be small.

A much more significant question concerns the possibility of progressive failure, and the status of peak versus ultimate strength. Unless there is strong evidence to the contrary, the collapse design situation must be assumed to be one in which all dilatancy has been progressively eliminated within fully softened shear zones mobilizing critical state strengths. Although residual strengths on polished slip surfaces in pure clays are known to be even smaller than the critical state strengths of random aggregates, there is no evidence that in-tact soil bodies suffer deterioration which reduces their average angle of shearing below that of critical states, especially if additional steps are taken to limit strains to the pre-rupture regime.

It has recently been pointed out by Bolton and Sun (1991) that a combination of kinematic and statical methods can offer the designer a simple way of assuring against unacceptable deformations. The mobilization of soil strength is typified by plastic hardening along a loading curve, with much increased stiffness observed on unloading-reloading cycles. A similar situation obtains for annealed copper. If, in either case, the objective is to design structures which will be serviceable in that they distort with no more than about 1% shear strain (say) at any point, a plastic design can be carried out with the strength at the "1% proof shear strength" rather than at either the peak or ultimate shear strength. The usual plastic equilibrium equation will relate loads to required shear strengths (bearing capacity factors, stability numbers etc). The hardening curve will permit the mean shear strain to be deduced from the mobilized strength. The ratio between normalized surface displacement (settlement / width, translation /

height etc) and mean mobilized shear strain can then be found from some appropriate plastic deformation mechanism which satisfies the kinematics: see Fig 1.

Taking this view, it can be shown that mobilization factors – reduction factors on peak strength – of about 1.5 on the undrained shear strength, or about 1.2 on the drained shear strength, are necessary to reduce shear strains in moderate to strong soils to within about 1% and typical displacement ratios to less than 1%, which might generally be regarded as on the limits of unserviceability. The evaluation of the safety and serviceability of soil bodies can then be treated in a unified fashion using simple plastic analysis with the soil strength set at the lower of two values: the ultimate strength at large strains, and the strength mobilizeable at permissible strains.

It must, of course, be recalled that the undrained shear phase of fine-grained soils will be followed by a transient flow phase in which the soil will consolidate (or swell) as positive (or negative) excess pore pressures dissipate as the ground comes into hydrostatic equilibrium or adopts a regime of steady seepage between the boundaries. It would be consistent with the simple plastic calculations described here to analyse the final, drained, soil condition in terms of the development *ab initio* of drained strength with shear strain, if such stress paths had been investigated in element tests. Equally, an approximation of one-dimensional compression might be used for the transient flow phase, so that the increment of consolidation settlement could be calculated if the pore pressures at the end of the undrained phase could be estimated, and appropriate oedometer tests had been carried out. Centrifuge tests have proved valuable in testing simplified procedures of this sort: more needs to be done.

Evaluation by Continuum Mechanics

It is usually thought better in principle to attempt a solution of the equilibrium, compatibility and constitutive equations at every point within the body, rather than rely on local mechanisms, of whatever sort. Algebraic solutions are possible to a restricted range of boundary value problems for ideal elastic material, and well documented in Poulos and Davis (1974) for example. The real problem is that soil is very non-linear even when it exhibits recoverable strains in executing a hysteresis loop on a stress-strain diagram. On taking the soil state beyond previous limits, irrecoverable plastic deformations occur. In performing such tests in other than a monotonic fashion some generalization of stress states is called for. The 5-parameter Cam Clay model, for example, is capable of representing isotropic hardening with elastic unloading and reloading inside a nest of yield surfaces. Even such a simple soil model can be used effectively only in the context of a numerical solution, probably using a finite element discretization of the continuum.

Failure by shear localization in heavily over-consolidated soil, the shaping of yield surfaces to account for inherent anisotropy, the introduction of hysteresis to replace elasticity, and the additional production of permanent plastic strains due to cyclic stress changes are essential extra components in some investigations. These greatly increase the numbers of parameters necessary to specify the pertinent model, and still leave some aspects of soil behaviour undescribed. Whittle and Aubeny, for example, in their paper "The effects of installation disturbance on interpretation of in-situ tests in clays", describe the use of the 15-parameter MIT-E3 model which is capable of representing some, but not all, of these additional facets.

There is increasing anxiety about the increasing number of parameters required to specify soil models. On philosophical grounds, Popper suggests that the best scientific theories generate the greatest number of testable propositions based on the smallest quantity of input data. On practical grounds, the designer will wish to limit the expenditure on soil testing to that which is truly necessary to guarantee economy and reliability. Note, however, that the number of model parameters is not the most appropriate measure of the complexity of input data. It is the diversity of soil tests demanded by the soil model, and the unambiguity of the means of fitting parameter values to this data, which is relevant. What should be sought is the soil model which makes maximum use of the information available from standard tests carried out on test paths which relate to the proposed facility.

The first essential feature of the Cam Clay family of soil models is their capacity to scale automatically for different preconsolidation pressures, using data of plastic compression on a plot of v versus p'. Their additional capacity to relate gross plastic behaviour at different over-consolidation ratios – through boundary surfaces particular to the variant – can easily be calibrated by performing shear tests at a range of initial OCRs. The remaining feature of stiffness inside the boundary surface can be fixed in a number of ways, as long as sufficient basic data of unloading and reloading has been acquired. The inevitable hysteresis can either be modelled through piecemeal alterations of element stiffnesses in incremental analyses, taking cognisance of strain history, strain path, and stress level, or by a more thorough-going "constitutive relation" fitted at the outset to the same data.

Refinement

If it is accepted, following Wroth and Houlsby (1985) that the efficient selection of continuum soil models is to be based not on their capacity to predict everything but their performance on the particular behaviour which is thought relevant, it might follow that the alternative geo-structural approach to evaluation will be even more efficient. If the mechanism is properly selected, the types of soil test (compression, extension, simple shear etc) which must be conducted on soils representative of different zones will immediately be apparent. The outcome will be in terms of sketches of soil movement patterns, simple equations between dimensionless groups, and corresponding charts. The consequences of parameter variations (i.e. uncertainties) and design modifications (i.e. revisions and refinements of the first guess) will be immediately obvious. To characterize the choice as between "design charts" and "doing the analysis properly" would severely mis-represent the situation, therefore.

Consider as an example the question of designing an embankment over a stratum of soft clay. The engineer should have in mind the following options:
a) to spread the embankment with slopes and, if warranted, berms
b) to use geotextiles (or other reinforcement) within the fill to retain vertical, or steep, faces
c) to reinforce the base of the embankment to resist spreading
d) to build the embankment in stages to capitalise on consolidation
e) to use vertical drains in the clay to accelerate consolidation

Whatever synthesis the designer first chooses will be arbitrary, and unlikely to reflect the optimum solution which will depend on relative land values, the urgency of completion, the tolerance of settlement in service, and aesthetic considerations, in addition to the more obvious matter of construction cost. What is wanted is a quick method of assessing the viability and effectiveness of each possible combination. Continuum analysis is not relevant at this stage: what is wanted is a structural engineer's mechanistic perception of the costs and benefits of the various components listed above. The author has been working with Dr Wing Sun to derive and validate plastic mechanisms applicable to this class of problem, using both statical and kinematic solutions based on the method of characteristics.

Sun (1990) showed that, whether plastic mechanisms or finite element solutions were applied to the analysis of these situations tested in centrifuge models, careful account had to be taken of strain-history and future strain path in determining appropriate stiffnesses "inside" the yield surface. Geo-structural calculations based on the same data gave answers that were just as reliable as those of the FE analysis. Extensions into the drained phase of behaviour based simply on Terzaghi's 1D consolidation theory were somewhat *more* successful than FE predictions. Modified Cam Clay assumes isotropic elastic soil behaviour inside the yield surface which generates volumetric soil consolidation in the absence of shear strain: consolidation settlement vectors point to the region of greatest consolidation, inwards, beneath the embankment. The centrifuge models, and real embankments of course, continue to shear the subsoil as it consolidates, due to the influence of kinematic plasticity. The net effect can be that displacements during consolidation are vertical, or inclined outwards as with the earlier shear displacements. It is sometimes easier to guess an appropriate mechanism than the soil properties which are necessary to induce it.

Objective and Subjective Design Methods

Within living memory geotechnical design was based on subjective judgements. A client would request a facility, and the consulting engineer would recall recent solutions for similar problems, and select a candidate solution on the basis of pattern recognition. A single bad experience with a particular material or technique would effectively banish it from use, irrespective of the reasons for its failure. A costly technique which was easy to design and which "worked" in practice could soon be copied by other designers, become a standard solution written in to Codes of Practice, and thereby hold off any challenge from more efficient, novel solutions. Even when soil mechanics calculations came into use, the specified "factors of safety" were ambiguously couched in terms of parameters which were not clearly defined, tending to obscure the fact that any such factor could be shown, at whim, to be either satisfactory or unsatisfactory depending on the assumptions. The evaluation process has largely remained normative, therefore, rather than objective.

Structural design broke free of conventional thinking about a generation ago when the analysis of structures could account properly for the interconnection of members. Not only could the elastic analysis of frames be carried out, their eventual plastic collapse could also be predicted. Although stability and buckling problems can hasten flexural plastic collapse, and membrane or dome action can delay it, designers were able to devise geometrical rules based on simplified treatments of these effects, which were successful in enabling quick decisions to be taken during the elaboration of structural form. At the same time, finite element analyses had been developed which permitted an analysis to be performed of the proposed structure making far fewer assumptions regarding its behaviour and the interactions between its components. Large scale physical tests, following the Merrison Report on box-girder bridge failures for example, were used to validate the methods of analysis.

It is now possible to design structures objectively to meet their performance targets. Such targets have to be specified, of course. Whilst "collapse" ought to be unambiguous, the criterion of "unserviceability" requires additional information. Burland and Wroth (1974) introduced simple mechanistic models of superstructure behaviour to define and set limits for the tolerable deformation of foundations. They treated whole buildings as being capable of characterization as beams deforming either in shear or flexure depending on their structural type, linked foundation distortion to tensile strain in the equivalent beam, and reported tests associating average tensile strain with the onset of visible cracking in panels, for example. This procedure for setting serviceability criteria for foundations embodies the scientific approach to design. Unserviceability is defined in terms of lack of fitness for purpose; unfitness is defined as including visible cracks in partitions; cracking is correlated with foundation distortion through a structural model. The procedure is objective because the terms are unambiguous, and it is scientific because every step of the process is open to challenge and reinterpretation.

It now seems that the techniques of Burland and Wroth can be carried below the superstructure into the foundations. The soil continuum itself can be declared to consist of geo-structural components. Experience shows that foundation distortion can be linked with soil strains in significant soil zones, and that the limitation of these strains can be achieved through the limitation of soil stresses to those mobilized at the limiting strain magnitude in appropriate stress-strain tests. Although much remains to be done, the technique of centrifuge modelling has been shown to provide – at moderate cost – the essential scientific feedback by which deformation mechanisms can be justified.

Fig 2 illustrates a procedure for geotechnical design which would permit creativity full reign while preserving an objectively scientific approach. The heart of the design method is the cyclic review of alternative schemes, and the successive modification of each scheme currently under consideration, leading to the proposal of a particular scheme with a particular configuration. This "final" scheme can be subjected to a rigourous finite element analysis if desired, but it will be earnestly hoped at that stage that no flaws will be found, and that no fundamental changes will be made. Of course, the final construction will not exactly follow the final design, and construction monitoring may be used in Peck's sense to "design as you go".

At the centre of the creative cycle in Fig 2 is the geo-structural analysis of limit modes in design situations. It is suggested that this can best be fulfilled by geo-structural mechanisms based on plastic methods, as shown in Fig 1. These can be framed to be sufficiently simple to enable comparisons of different techniques and geometries, descriptive enough to enable spatial problems of interaction to be recognized or avoided, and close enough to the truth to guarantee that any "final" analysis will be unlikely to overturn their conclusions.

EXAMPLES FROM THE SYMPOSIUM

Earth Retention and Soil Reinforcement

Jewell, Burd and Milligan raise some important issues of design, analysis, and scientific evaluation in their paper "Predicting the effect of boundary forces on the behaviour of reinforced soil walls". Their contribution is based on their earlier work which established an understanding of the action of soil reinforcement through the use of limit equilibrium analyses. Clear mechanisms emerge from these studies, derived via the equilibrium of wedges, and enhanced by introducing consistent arguments related to deformations and kinematics. The pictorial presentation of reinforcement action, and the exposition of results in terms of dimensionless groups of parameters, fulfils exactly the conditions laid down above for the type of geo-structural mechanism which is capable of being used creatively in design, whilst remaining open to further scrutiny.

The prediction of wall displacements and reinforcement extensions is under review at a number of research centres. In order to test the success of such predictions, Bathurst conducted large scale tests at the Royal Military College, Canada. The reinforcement tensions, and consequent wall displacements, were much smaller than had been anticipated. Subsequent investigations showed that this was due, in the main, to side-wall friction influencing the results of the 3m high and 2.4m wide wall. Jewell et al produce a variant of their limit equilibrium analysis to take account of boundary forces, following an analysis by Bransby and Smith which was developed 17 years ago in identical circumstances, to salvage the data of reinforced earth models tested in a narrow chamber with insufficient side-wall lubrication. Jewell et al confirm that their mechanism is capable of explaining the results of the RMC trials, by including the bearing thrust on the bottom wall panel, and friction on the sides of the test section.

Boundary friction will continue to challenge experimental workers, and 3D effects will continue to puzzle geotechnical designers. Jewell et al point out, however, that it may well be inconsistent to include in design calculations each source of strength monitored in pilot tests. The design situation would have to assume minimal bearing capacity under the front panels unless particular trouble had been taken to assure its presence, and the effect of side-wall friction could not be relied upon unless the relative immobility of the side walls could be guaranteed. Even more uncertainty currently exists regarding the advisability of adopting the peak angle of shearing in well-compacted backfill (over 50°, probably, in plane strain), as opposed to fully dilated critical state state angles (perhaps 35° to 38°). The opportunity exists for research workers to demonstrate – if it is the case – that some reinforced fills do not strain enough to soften, and for engineers in the field to investigate whether a high degree of compaction in fills containing relatively extensible geotextiles is either possible or desirable. Until that is clarified, it is necessary for designers to take the pessimistic view – the worst credible scenario – consistent with the precepts of limit state design.

Excavations and Tunnels

Sloan and Assadi describe an attractive numerical package for plastic analysis in their paper "Stability of shallow tunnels in soft ground". They have developed two finite element techniques which satisfy, separately, the conditions for statical and kinematic solutions to bound the equilibrium of perfectly plastic bodies – here taken to be undrained clay with any linear variation of strength with depth. In each case, the closest bound obtainable within a specified family of mechanisms is found through linear programming optimization. Statical solutions search through equilibrium solutions which can satisfy body forces and include any permissible stress-discontinuities at the boundaries between elements. Kinematic solutions search through mechanisms which involve uniform strain within elements and compatible displacement-discontinuities on their boundaries.

These solutions are therefore automated forms of the geo-structural analysis referred to earlier, and with sufficient degrees of freedom to be capable of approaching the "correct" answer – as the authors demonstrate in the case of plane strain cavity collapse. Although tight bounds had already been produced manually by Davis et al for the case of constant strength with depth, Sloan and Assadi show that efficient solutions can be generated for strength varying with depth. Bounds relative to their mean value were generally ±5% and not worse than ±10%, based on the ratio of the strength required in the soil at the depth of the cavity centre, and the vertical stress deficit at the cavity centre (i.e. the stress in the free field minus the cavity pressure). Although there are many dimensionless ways of presenting stability data, the key parameter is always the strength required for equilibrium divided by the stress difference causing distortion. These values, written perhaps as a mobilization ratio $c_{mob}/\Delta\sigma$ and calculated at collapse using Sloan and Assadi, fell between 0.6 (for a cavity with cover/diameter ratio 1, in soil which lost half its strength between the crown and the ground surface) and 0.2 (for a cavity with cover/diameter ratio 5, in uniform soil), approximately.

Mair and Taylor in their paper "Predictions of clay behaviour around tunnels" show that there are significant advantages in the medium strain range in modelling the soil movements around tunnels in terms of spherical cavity collapse into the approaching heading, followed by cylindrical movements onto the tunnel supports once they are in place. This geometrical understanding is enhanced by relating the magnitude of deformation to the stability number $\Delta\sigma/c_u$, using cavity expansion theory. The result is an admirable physical clarification, embodied in dimensionless charts of meaningful parameters, of the soil movements due to tunnelling. The authors use a linear-elastic perfectly plastic relation for the soil, but remark on its inability to characterise some significant elements of the data they present. This aspect of the problem is carried forward in another paper to the Symposium.

Mair et al (1981) presented their results for "ground loss" above shallow tunnels in terms of a "load factor" which could be interpreted as c_{mob}/c_u. Gunn recalls Mair's centrifuge test data, and earlier finite element analyses based on Cam Clay with simple κ-behaviour inside the yield surface, in his paper "The prediction of surface settlement profiles due to tunnelling". Gunn shows, as Bolton and Sun (1991) also remarked, that power curves fit the medium strain range of the shear stress-strain data of clays rather well. He uses an FE analysis with a soil model based on Tresca with a power law hardening curve to demonstrate that higher small-strain stiffness causes the soil deformations to localize, in better agreement with the physical evidence. The magnitude of "ground loss" - proportional loss of cavity volume, expressed as a subsidence trough – corresponded with the earlier findings, namely that between about 1.5% and 3% ground loss might be expected at $c_{mob}/c_u = 2/3$.

This finding conforms with the spirit of the geo-structural approach described earlier. The stress-strain function which Gunn used gave $c_{mob}/c_u = {}^2/_3$ at an "axial strain" of about 0.75% which presumably relates to a shear strain in plane strain of 1.5%. The simplest geo-structural mechanism which may convey something of the kinematics, at least for deep cavities, is simple cylindrical contraction for which proportional volume change equates to shear strain mobilized at the cavity boundary. Crudely, therefore, a 1.5% shear strain in the soil near the cavity boundary might have been expected relate to 1.5% ground loss. If it is generally essential to mobilise no more than ${}^2/_3c_u$, that is to have a mobilization factor of at least 1.5, to limit ground deformations in undrained clay, Sloan's ±10% uncertainty regarding the bounding of the "correct" value of c_{mob} is seen to be of negligible practical importance.

Hight et al in their paper "Predicted and measured tunnel distortions associated with construction of Waterloo International Terminal" were clearly commissioned, in the terminology of Fig 2, to perform a "final analytical check" rather than a recursive design routine. Nevertheless, it is interesting to compare their computations and observations with the estimates available through geo-structural thinking. The objective was to determine the changes in tunnel diameter on the Bakerloo line due to an average 180 kPa of unloading due to excavation above. This is a reversed bearing capacity mechanism with a bearing capacity factor of about 6, so the average change in mobilized shear strength must be about 30 kPa. The authors went to immense trouble to establish the *in situ* stresses ($K_o \approx 1$ near the tunnels) and the small strain stiffnesses of the soils respecting their strain history – the essential input to any method of analysis.

The tunnels in question were close to the weathered/unweathered boundary for the London Clay. Since the data was presented (inconveniently for these purposes) as normalized secant stiffness versus log strain, rather than normalized shear stress versus log strain, it was necessary to back-calculate a few points to find the strain required to mobilize 30 kPa. For an initial effective stress $p_o' \approx 190$ kPa, the authors' data imply that a shear stress of 30 kPa can be mobilized at triaxial extensions of about 0.08% in the weathered clay and 0.22% in the unweathered clay: they remark on the strange drop of apparent stiffness with depth. However, a tunnel lying in the passive unloading zone will tend to take up the vertical extension of the soil surrounding it, so the increase in vertical diameter of a 5m diameter tunnel might have been guessed, without further analysis, to lie in the range 4mm to 11mm, depending which value of extension was selected. The observed range of extensions was apparently 5mm to 8mm. This simple geo-structural analysis confirms and explains the overwhelming importance of stress-strain data in the first 0.25% of strain following some new excursion. In critical cases, especially of soil-structure interaction, a non-linear finite element solution respecting measured soil hysteresis will be indispensable.

This was demonstrated in the paper "Prediction and performance of ground response due to construction of a deep basement at 60 Victoria Embankment" by St John et al. The authors were able to create a numerical simulation of the excavation process, including intermittent changes in the conditions of fixity of the secant-pile wall as floor slabs were cast. In cases such as this, where possible damage to neighbouring property was a concern, strain-history effects in the soil had to be accounted for in the stress-strain response anticipated in active and passive zones.

The authors conclude by showing that maximum wall displacements induced during the construction of various excavations in London Clay: 30mm due to 10m of excavation was typical. Following the geo-structural approach of Bolton et al (1989, 1990) the lateral displacement should be divided by the effective depth of the wall in order to obtain an estimate of the principal strain in a plane element test on the surrounding soils. Although the depth of excavation, rather than the depth of the wall, was quoted, it seems likely that the induced principal strain is about 30mm on 20m, or about 0.15%. It is the soil strength which can be mobilized following construction of the wall, and after a further 0.15% principal strain, which could be entered into preliminary design calculations. The authors do not comment on whether this represents a carefully considered serviceability limit or whether, in some cases, more displacement could have been tolerated and a thinner wall or cheaper support system could have been provided to support soil which was mobilizing a greater proportion of its own strength.

Shallow Foundations

Two papers on spud foundations for jack-up rigs reflect precisely this reporter's preference for self-contained approximate mechanisms of behaviour to clarify the task of design. Dean et al present "The bearing capacity of conical footings on sand in relation to the behaviour of spudcan footings of jack-ups", while Houlsby and Martin offer "Modelling the behaviour of foundation jack-up units on clay". As a conical footing penetrates a soil bed it continuously shears the soil through a succession of self-similar states scaled by the instantaneous depth of penetration which is used as a sort of hardening parameter for the soil-structure system. Both papers offer an interpretation of the plastic yielding of the soil bodies beneath the spuds in terms of an interaction between the vertical force, horizontal force, and moment applied by the structure. Both draw attention to the similarity of the emerging system model to the Critical State material models, with the shapes of yield surface defined empirically rather than theoretically.

Houlsby and Martin aim to determine the penetration of a single spud in soft clay, first under surcharged vertical load on placement, then on recovery to working load, and then under storm conditions with shear and moment interactions. The self-consistency of interaction envelopes was demonstrated at 1g in very soft clay models subject to strain-controlled loading, with control of the three degrees of freedom of the spud. They point to the greater uncertainty on the "dry" side where, just as with material models, sliding and cracking rather than plastic deformation must be expected. Here, under a lifting leg, the resilience of the clay is significant and the authors may like to consider the use of power curves fitted to hysteresis loops rather than the identification of an equivalent-linear shear modulus.

Dean et al consider the behaviour of a three-legged jack-up unit on sand. They develop plastic yield envelopes as described above, and also develop possible paths in load space through a simple structural analysis. This simple geo-structural idealization was validated through a sequence of ingenious model tests carried out in a drum centrifuge. The normalization of vertical force with penetration depth was achieved through an apparently constant $N\gamma$ value corresponding to quite low values of ϕ, at or below critical state values. Only 34° was mobilized in fine, medium dense Leighton Buzzard sand, for example. It is interesting to speculate on whether higher ϕ values would be mobilized in dense sand, or whether shear strains during placement are effective in causing dilation to critical states. The authors may need to consider the influence on ϕ of dilation or contraction at different levels of mean effective stress, especially since the stress under the 1.6m diameter prototype spuds reported here are an order of magnitude smaller than those applicable in the much larger field-scale structures. The installation behaviour would, in any event, permit the re-evaluation of storm effects using the interaction diagrams presented here. The possibility of taking commercially valuable decisions regarding the moment-fixity of spuds, and therefore of the working envelope of particular rigs, is a good example of the virtue of simplified, wholistic, simulations for design and operation.

Schnaid et al in their paper "An investigation of bearing capacity and settlements of soft clay deposits at Shellhaven" provide an example of "design as you go" in planning a haul route for an exceptional load. The success of the exercise demonstrates the value of careful investigation, testing and observation. A geo-structural analysis might have been performed with the following effect. The measured bearing capacity was 84kPa from a load test, but this was reduced to 80kPa as a correction for shape effect. However, the settlement at failure was of the order of 0.25m which must have created a relative depression of about 0.3m, giving a buoyancy effect of perhaps 4.5kPa, so the true bearing capacity might have been about 75.5kPa. The bearing pressure finally adopted was 37kPa giving a mobilization factor of 2.04 against collapse. A triaxial sample with a compressive strength of 29.5kPa mobilized a deviatoric stress of 29.5/2.04 = 14.5kPa at an axial strain of about 0.19% in a triaxial test, corresponding to a shear strain of 1.5 x 0.19% or $\gamma = 0.285\%$ which would have occurred in a plane test at a principal strain of 0.14%. Following Bolton and Sun (1991), the settlement of a 6m wide strip on 5m to 7m of the soft clay would have been predicted to be simply 0.14% of 5m to 7m, or 7mm to 10mm. The average settlement during passage was inferred to be 8 to 10mm under the edge of the load.

Deep Foundations

Mandolini and Viggiani present a paper "Settlement predictions for piled foundations from loading tests on single piles". Their objective is to demonstrate that linear elastic interaction coefficients can be used with an empirical correction to predict settlements of flexible piled rafts from single-pile tests. It becomes clear from their data, however, that the stress-strain data of the sands through which the piles penetrated must have been highly non-linear. Suppose, for example, that the vertical shear stress could be represented by a power-law

$$\frac{\tau}{\tau_m} = \left(\frac{\gamma}{\gamma_m}\right)^b$$

Now let the shear strain in the soil be γ_0 at the pile interface radius r_0, reducing to γ at radius r. Vertical force balance means that $2\pi r\tau$ must remain constant at any radius, neglecting end effects. Equating the drag-down slope (dw/dr) to the shear strain γ, and integrating, it can then be shown that the settlement of a single friction pile is

$$w_o = \gamma_o r_o$$

while the drag-down at radius r can be written

$$\alpha = \frac{w}{w_o} = \left(\frac{r}{r_o}\right)^{1-1/b}$$

and the development of settlement of the pile itself mimics the original power-law

$$\frac{Q}{Q_m} = \left(\frac{w}{w_m}\right)^b$$

where Q_m and w_m form a point on the load-settlement curve representing ultimate failure. Fig 3a shows the comparison between the expression for interaction factor α and the authors' results from dual-pile tests, and Fig 3b shows the single-pile settlement curve, fitted for exponent b = 0.565. The fit is excellent. This shows that the authors' intention to extrapolate from single pile tests may be possible, but that the analysis should be non-linear. This in turn means that more thought should be given to integrating effects over a number of loaded piles, since superposition will not apply. The authors are, however, quite right to say that deformations are much more highly localised around piles than linear-elastic theory suggests, so the integration need not involve piles at large separations.

Lehane and Jardine show data of pile installation by jacking, and later loading behaviour, in their paper "The behaviour of a displacement pile in Bothkennar clay". The skin friction was found to depend on residual friction within a superficial zone of disturbance within which anomalous pore pressures must have developed during jacking. Excellent data of transient radial stress and pore pressure at the pile, at various points along its length, presently defy simple interpretation. The effective radial stress is difficult to predict.

A similar observation is made by Fahey et al in their paper "Parameter selection for pile design in calcareous sediments". They show a complex behaviour in terms of local dilation or contraction at the pile interface being offset by the response of the surrounding soil in cavity expansion or contraction. Constant normal stiffness tests in direct shear provided an analogous interaction, with the pile material sprung against the soil in the laboratory. Once again, the highly non-linear stress-strain relation of the soil at small shear strains proved essential in understanding the results. The higher the shear modulus of the soil, the faster the radial effective stress can be lost due to contraction. There is clearly some hope that extrapolations of pile field tests, and the use of CNS or other novel laboratory tests, will soon prove adequate for the more accurate and economic design of friction piles.

CONCLUSIONS

Far more than a method of prediction, the designer needs a descriptive mechanism which embodies the working of any geotechnical facility. The mechanism will need to represent the equilibrium and displacement of the various elements of the facility, especially at their boundaries. Overall function is much more important than local detail: in that sense, the mechanism may be like the structural engineer's idealization. Engineers' beam theory fails to deal with stress concentrations at joints, and ignores shear deformation in the axiom "plane sections remain plane": it is invaluable, however, in the design of buildings. The symbolism of a geo-structural model should ideally have a closed algebraic form, or at least offer an unequivocal dimensionless scaling from measurable parameters to desired output.

Error is qualitatively different than approximation. What is desired is a soft-focus perspective of the workings of the system, (with optional zoom!). In that sense, the full complexity of a finite element program or a centrifuge model test simply takes the engineer one step of the way: the next step must always be to attempt to produce a simplified model of the complex reality. Geo-structural models which have proved useful include: active and passive triangles, Prandtl's bearing capacity mechanism generalized for inclined loads, and the equilibrium of a cylindrical insert expanding, contracting, or translating in an infinite medium.

The non-linear relationship between stress and strain in soil is central to a correct understanding of soil deformations and ground displacements. Soil hardens through plastic strain in a fashion not unlike that of many metals or polymers. Whereas geotechnical engineers have been used to applying plastic theory to collapse at peak strength, structural engineers perform their plastic calculations at a yield stress far below ultimate tensile strength, because they wish to limit plastic deformations and avoid progressive failure. Geotechnical engineers should perform safe analyses of collapse with fully softened critical state strengths. They should, in this reporter's view, use elementary plastic calculations to limit deformations to the medium strain range (<1%, perhaps).

It is well known that significant non-linear behaviour is exhibited in all soils beyond about 0.01% strain, so linear elastic calculations are difficult to apply. What value of modulus should be taken? Evidence is growing of the success of an approach treating a stress-path test, on an element representative of some soil zone, as a curve of plastic mobilization. Plastic equilibrium relates applied stresses to mobilized soil strengths. Strains are deduced from the test. They are then entered in to a plastic deformation mechanism to predict boundary displacements. The approximation is good if the mechanisms are appropriate: hence the need for physical or numerical modelling. Various levels of sophistication are called for in fitting curves to stress-strain data: a simple power curve has proved very valuable.

We correctly use simple models to teach young engineers how the world works. We can extend this thinking now in to most areas of geotechnical engineering. This must lead us to provoke more engineer-like behaviour in our students by setting exercises which go beyond prediction (what is the settlement?) to refinement (what width of foundation would limit settlement to 25mm?), and then to synthesis (do I need piles here, or not?).

Research projects often emphasise scientific exploration followed by numerical prediction, and fail to follow these up with the creation of simplified idealizations – "seeing the trees as a wood". The responses of grant-giving bodies and academic referees often show a surprising ability to miss the point. One of the first referees to look over the reporter's plastic mechanism approach to predicting the settlement of foundations in two lines, taking non-linearity in to account, wrote that "all this could be done in an FE analysis"! Perhaps that is why, in the UK especially, practising engineers have undervalued research. We need to persuade industry to support our best post-doctoral workers, those people who have done the science and the numerical analysis, while they undertake the essential task of digesting and clarifying their understanding. This must be the best way of bridging the gap between "research" and "design" to the mutual advantage of both cultures.

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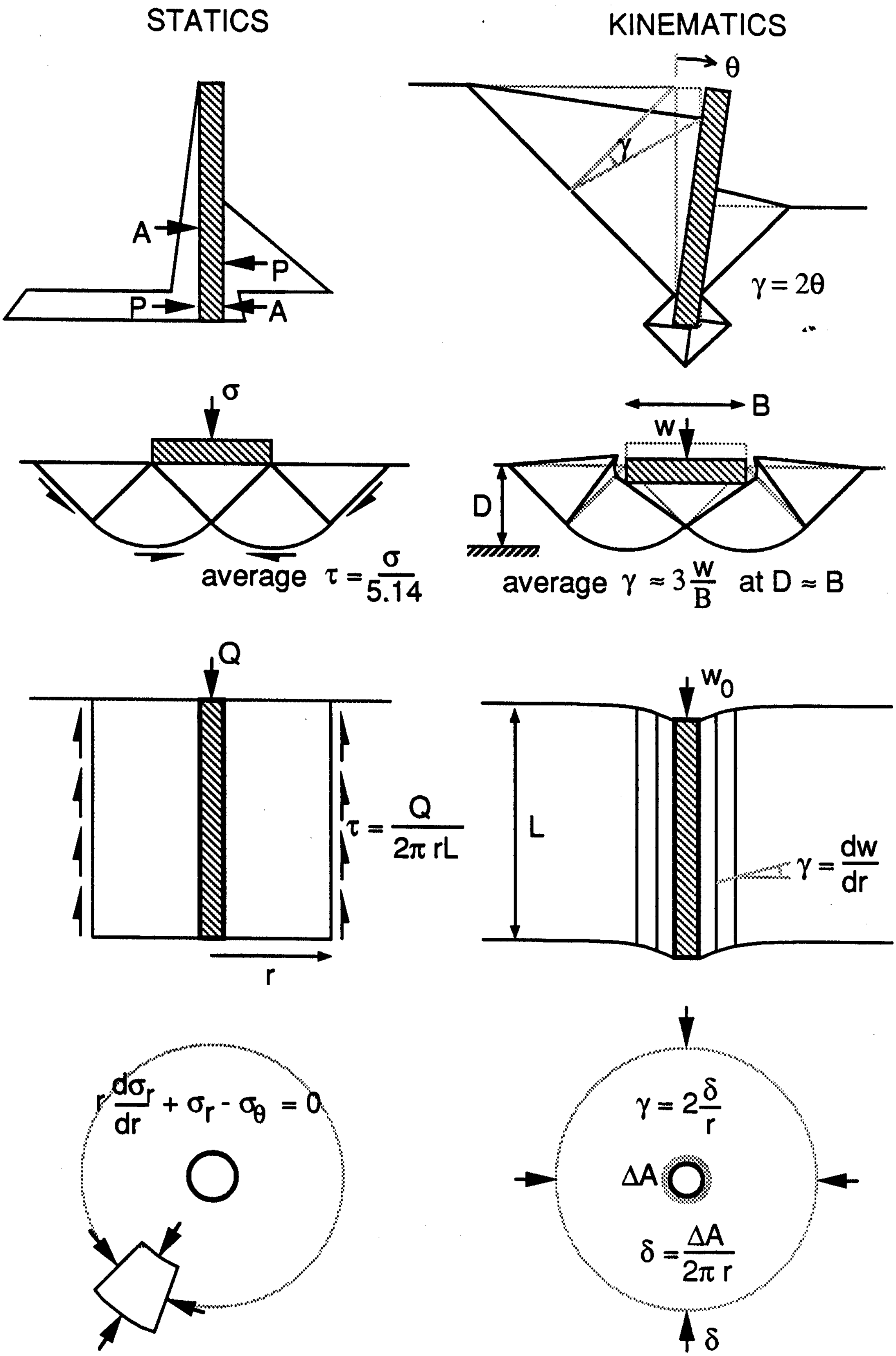


Fig 1 Geo-Structural Mechanisms

Fig 2

DESIGN PROCESS

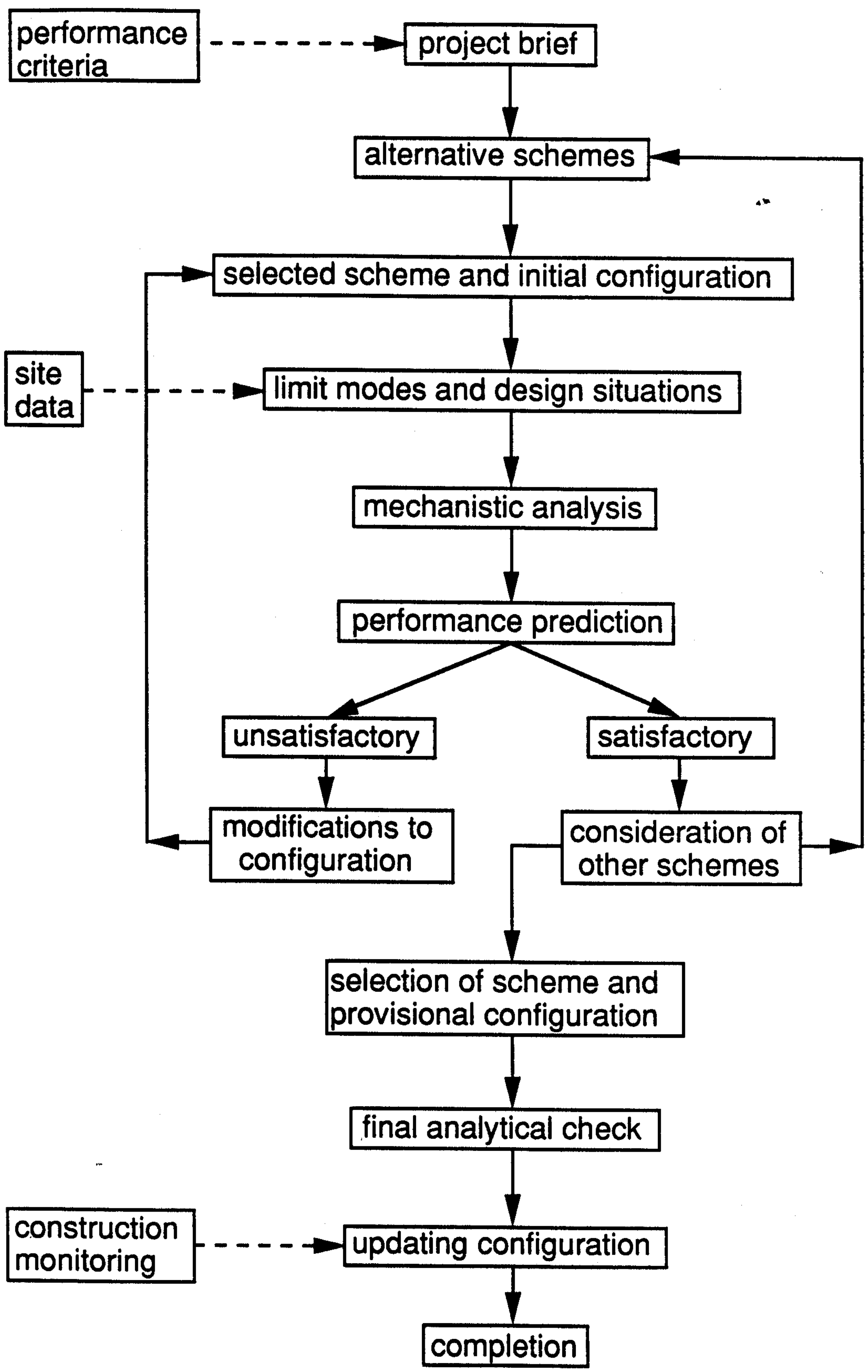
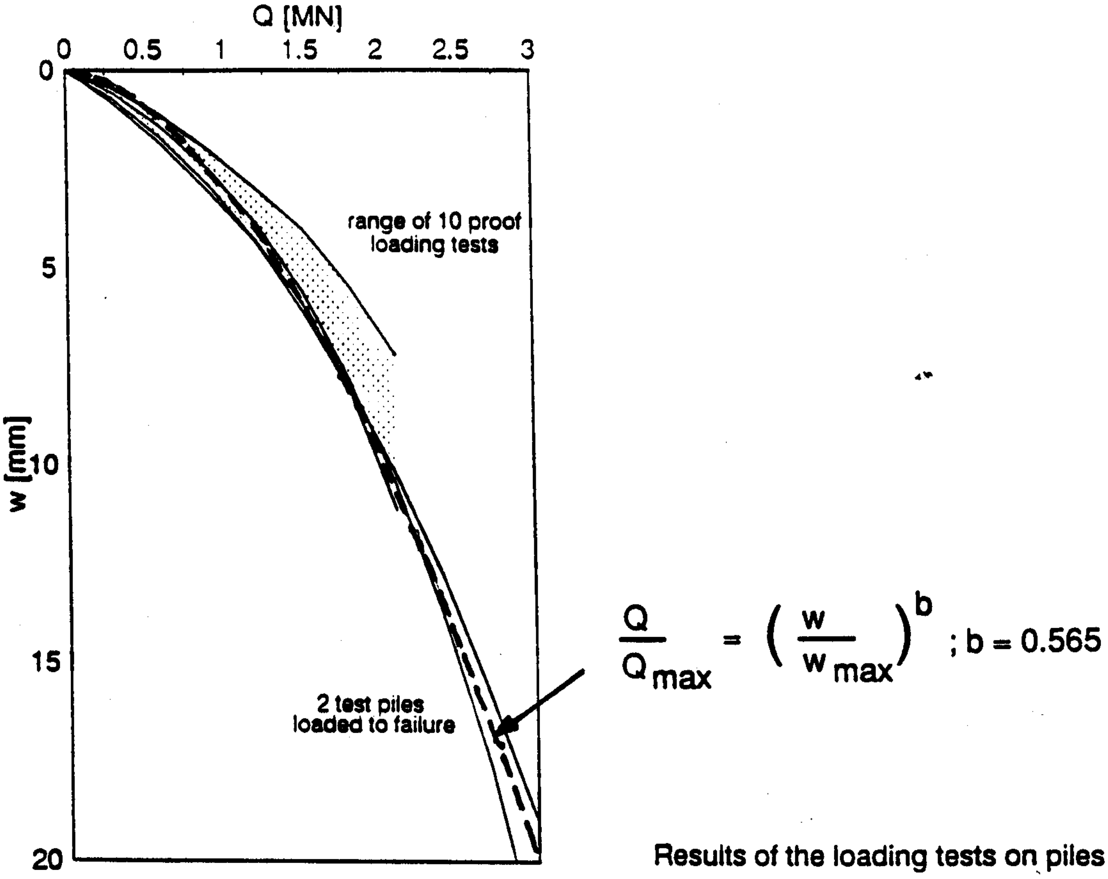
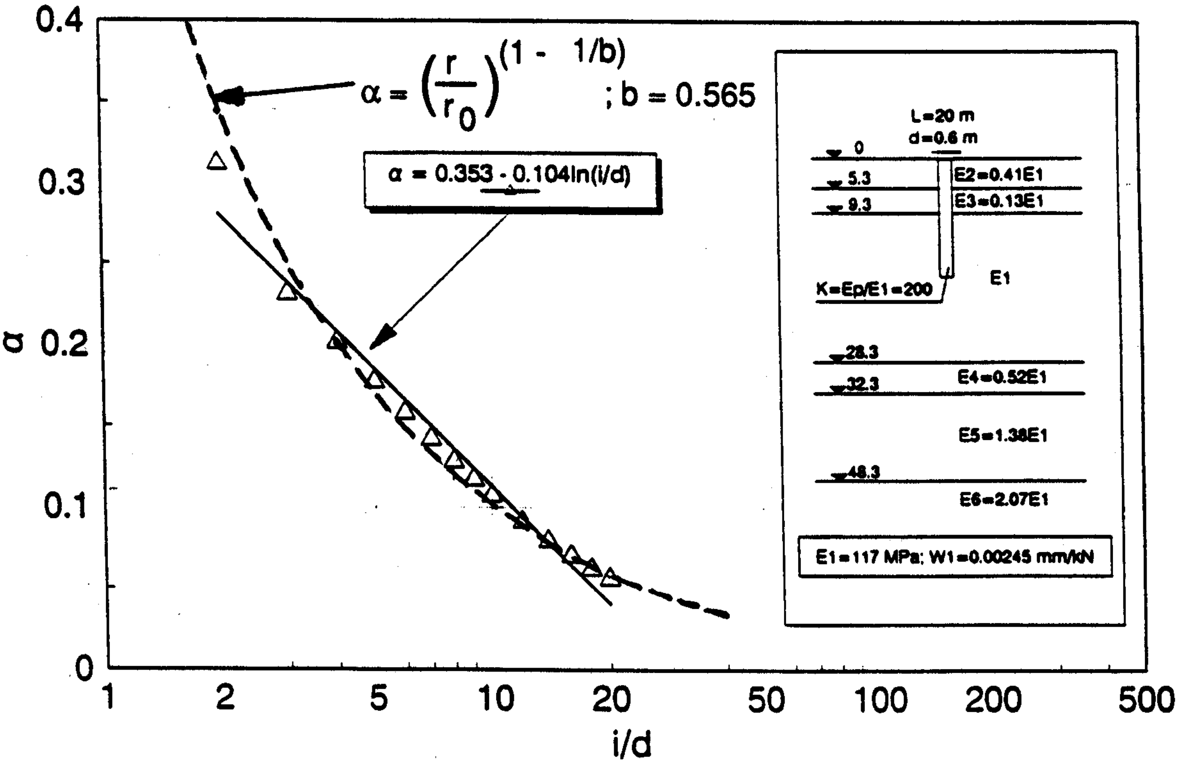


Fig 3a



Results of the loading tests on piles

Fig 3b



Computed values of the superposition factors and subsoil model

Taken from Mandolini & Viggiani