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Chairman's report: Design methods

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1 THE SCOPE OF THE DESIGNER

A discussion on design methods must inevitably begin with a reconsideration of what we mean by design; there are three contrasting approaches.

Practitioners might like to reduce the scope of design to the basic decisions which must be taken on the materials to select, the manner in which they are to placed and connected, and the form and dimensions of the finished structure. So practitioners would wish to hear of new materials and fresh opportunities.

Research workers might like to focus more on the evidence which is available on the real performance of structures in comparison with the simplified mechanisms which have been developed to assist designers with their calculations. So research workers would wish to hear of new theoretical mechanisms which more closely reflect the truth.

Regulators might like to retain simplified analyses and approach real behaviour through the alternative device of probability theory and factors of safety. So regulators would like to hear of standard measures of acceptability in the face of uncertainty.

This session was organised in an attempt to satisfy all three desires - the practical, the scientific, and the philosophical. The broad range of the word "design" is reflected in the broad range of activities which may seem relevant. Consider the following list of ten things a designer might do:

- refine the project brief in relation to life-costs, completion date, robustness, and visual appeal;
- select the materials and methods of construction;
- dimension the structure using accepted calculation procedures;
- conduct a site investigation;
- requisition appropriate tests on materials;
- judge loads, earthquakes, floods and other site-specific hazards
- evaluate the software used in standard calculation procedures;
- commission expert FE analyses where conditions extend beyond current experience;
- monitor construction, and compare observations with expectations;
- develop a database of the long-term performance of structures in service.

The legal situation will generally dictate that the first five are essential, while the commercial situation may suggest that only well-established companies would entertain the last five (and demand higher fees accordingly). Public clients, and regulators, will be aware that they are taking a risk in permitting non-specialists in small companies to design reinforced soil structures, especially in relation to site-specific hazards. They may nevertheless wish to do this in order to keep design costs low.

The methodology generally chosen to reduce risk to an acceptable level is that of the Code of Practice. In the UK (with which the Chairman is familiar) a Code of Practice, such as the new British Standard BS 8006 for Reinforced Soils, has only implicit legal force unless it is mentioned in contract documents. In the UK system, Codes are quite likely to be incompatible since they are produced by independent committees, so the conflict in approach between BS 8002 Earth Retaining Structures and BS 8006 Reinforced Soils, for example, is not surprising. It may even be seen as valuable, since a monolithic and comprehensive set of interlinking Codes would undermine our perception of uncertainty and risk, and might lead to the collective loss of the professional judgements represented by the last five bullet-points for action which are listed above.

Our understanding is not perfect, and it is both honest and useful to display differences of opinion where they occur. That is what the succeeding discussion is for. It is the responsibility of Government ministries and regulators (or insurers) to assist the engineering community in maintaining a healthy proportion of independent experts and successful companies, who continually test their understanding and extend technology beyond current limits. It might be argued that a free market in design services will keep costs down while an intelligent regulatory mechanism, which recognises the different capacities of individuals and companies, is essential to keep quality up.

The tension between quality, cost and risk is reflected obliquely in discussions on Code requirements. Simple authoritative statements may lead to cheap designs today, but may block progress to cheaper or more effective designs tomorrow, or may even lead to rare but catastrophic failures if they introduce false concepts which conflict with scientific evidence or logic. More honest expressions of alternative possibilities in complex circumstances may lead to gross errors due to confusion on the part of the inexperienced. There is no monopoly of wisdom, either way.

However, research workers must recall that the final objective is the creation of new works. Wonderful analyses may be pregnant with conceptual insight but they will lead nowhere unless that insight is distilled into some simple rules for the guidance of decision-makers.

2 SOIL CHARACTERISATION

Geosynthetics are novel and exciting materials which have contributed most to the development of these continuing symposia in Kyushu. But soil is a complex material too, and it often seems to be neglected. There has been very little discussion about the level of strain at which the strength of soil is required, whereas there has been considerable anxiety about the admissible strain of reinforcement. Figure 1 points up the incompatibility between recording the ultimate tensile strength T_{ult} of steel reinforcement, which may occur at 20% strain, and the peak strength ϕ_{max} of the surrounding soil which may occur at only 1% strain. The ultimate strength of granular soils, which might logically be used with T_{ult} for safety evaluations at large strains, is the critical state friction angle ϕ_{crit}. The larger ϕ_{max} could be relevant to serviceability checks. The exceedence of 1% strains in service, for example, could be prevented by the designer checking that the 1% yield strength T_{yield} would not be exceeded due to earth pressures calculated from ϕ_{max}.

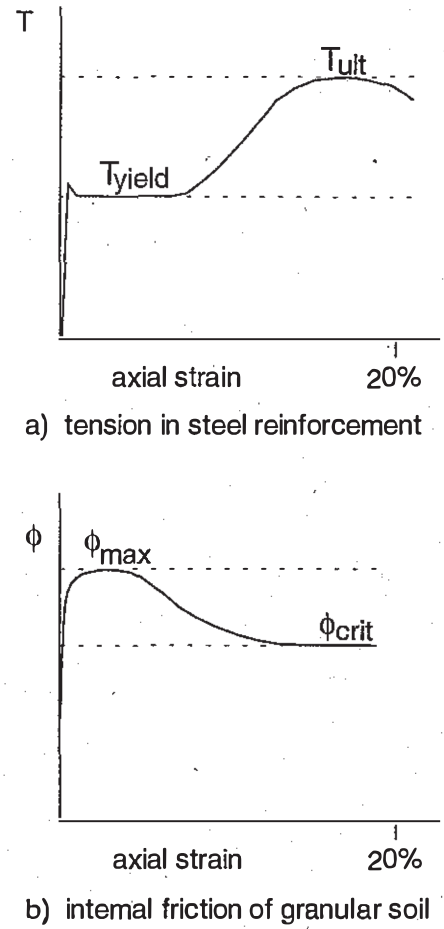

Fig 1 Strength and compatibility

There needs to be a much wider recognition that historic safety factors have had the effect of reducing mobilised angles of friction below the safety net of ϕ_{crit} and, indeed, below the level at which soil strains could be considered excessive. The new BS 8002 took the view that a mobilisation factor $M = 1.2$ was generally necessary to reduce the representative peak friction angles of moderate to good soils to a value which should not require any linear strain to exceed 0.5%. This limit was placed on the basis that traditional concrete or steel retaining walls would begin to show signs of distress at corresponding face rotations of about 1%. Reinforced soil structures should be more tolerant of deformation, and simple designs might best be calculated on the following basis:

- ${}_{design}\phi = \phi_{crit}$ to ensure safety;
- fill is to be well-compacted to limit strains;
- fill is to be free of fines to ensure drainage
- no further safety factor on earth pressures
- appropriate design strength for reinforcement.

At present, designers are often astonished to find that $\phi_{max} \approx 45°$ in low-stress tests on dense fill. The suggested approach leads them to use ${}_{design}\phi \approx 35°$, corresponding to the internal friction of loose angular fill, but then requires that it be properly compacted. The advantage chiefly lies in the justification of 35°, which shows that no other safety factors are needed.

A similar approach to design can be used with the undrained strength parameter c_u, except that a larger reduction factor of $M = 1.4$ was recommended in BS 8002 if strains were to be limited to 0.5% as before. The ${}_{design}c$ value of c_u/M can then be used, as before, without further factoring. For example, a well-reinforced block of width B placed on top of a deep clay layer should not settle immediately by much more than B /200 if the bearing pressure were limited to $5.14\,{}_{design}c$. Logically, larger M values should be used if tighter control of construction-induced strains is required. However, clayey soils can not escape consolidation or heave due to long-term changes of effective stress.

3 EARTH PRESSURE IS NOT A LOAD

The suggestion, made above, that earth pressure is not simply to be factored up for the purposes of selecting the required reinforcement, is difficult at first for structural engineers to accept. Engineers have become used to treating design live load values as though they were selected from probability-density functions. Stochastic live loads can logically be factored up, so that "collapse loads" in Ultimate Limit States (ULS) will be larger than "working loads" in Serviceability Limit States (SLS). Obviously, engineers would wish to assume more severe loading for ULS checks of very low probability events which have more severe consequences. Earth pressures are not stochastic live loads, however.

As a soil-structure deforms, and strains increase, the mobilised soil strength rises towards its peak and then falls to a critical state. Designs should be based on ${}_{design}\phi = \phi_{crit}$ mobilising critical state strengths, but on the way up to peak strength in Fig 1, not on the way down. In compacted granular fill, this guarantees acceptably small strain. If there were an exceptional live load on the fill, or an earthquake, or a sudden elevation of the water table, or some other loading event which causes the structure some distress, the mobilised soil strength would then increase towards ϕ_{max}.

The following logic applies:

- a safe structure in a working state mobilises an earth pressure coefficient K_{mob} based on ${}_{design}\phi < \phi_{max}$;
- if it is challenged by real live loads (such as a heavy vehicle) its earth pressure coefficient will fall as ${}_{design}\phi \rightarrow \phi_{max}$;
- the actual earth pressure may rise if the live load increases faster than the earth pressure coefficient falls, or if water pressure increases, for example;
- foreseeable live loads must be included in the analyses, and traffic loads may be factored if that is considered appropriate;
- earth pressures will fall from SLS to ULS conditions if the only cause is extension in the reinforcement, due to creep for example.

Estimation of initial earth pressures after construction may require further thought. The earth pressure coefficient behind a rigid face is often said to be $K_o \approx 1 - \sin\phi_{max}$, corresponding to conditions "at rest". It will be found that this is equivalent to mobilising an internal angle of friction $\phi_o = \phi_{max} - 11.5°$. This expression makes it easier to understand the consequences of subsequent lateral strains which erode the 11.5° margin between ϕ_o and ϕ_{max}. However, the K_o condition only applies to gentle pluviation. If the fill has been compacted to density γ by a roller of effective weight Q per unit width, extra lateral pressures up to $\sqrt{(Q\gamma)}$ may be locked in place. These larger pressures are relaxed, as explained above, if the wall face begins to move out.

The consequences of large strains also require some thought. The proposal made here is that the design point lies below ϕ_{crit}. The sudden imposition and removal of a live load, such as might be caused by an accident or explosion, should then leave the structure in equilibrium so long as the reinforcements have not also ruptured, or become disconnected from the facing panels, in the process. If design had been permitted with ${}_{design}\phi > \phi_{crit}$, however, an accidental load cycle could leave the structure generating mass-accelerations due to the imbalance between load and resistance; this would lead to catastrophic and sudden collapse in which lives could be lost.

DESIGN USING MOBILISABLE STRENGTHS

Limit Equilibrium (LE) and Finite Element (FE) analyses may seem to be in opposition, but they can come together most helpfully. LE analyses are based on equilibrium and ignore compatibility, whereas FE analyses take both into account. Failure in an FE analysis is usually recognised when the solver discovers equilibrium errors which it can not relax. At this point, there will be some failure mechanism developing in which strain is accumulating in the body without further resistance being developed. In a reinforced soil structure, some reinforcement will be extending plastically, some may be slipping relative to the neighbouring soil, and some soil zones will also be in some limiting state of stress which induces plastic shear strains, so that peak strength begins to fall towards critical states. In order for the FE analysis of failure to be reasonable, it will therefore be necessary to have a good non-linear solver, interface elements which permit sliding of extensible reinforcement, and a soil constitutive model which features peak strength dropping to critical states.

However, structures are also required to deform acceptably, and this criterion will be more demanding than the final collapse situation in a ductile body. If strains of the order of 1% are considered limiting, it might be assumed that granular fills will have reached peak strength but not yet deteriorated towards critical states, whereas reinforcing materials within the fill, and soft clays beneath, will have mobilised only a fraction of their peak strength. The ratio of the strain mobilised in these compliant materials will be some proportion of that observed in the structure as a whole, expressed perhaps as a displacement / height ratio. These strain ratios can be established for various classes of construction, and proportions of reinforcement, by FE analysis. These results can then be used in practice through LE analyses which account for both the mechanism and the strain ratios.

For example, if it happens that synthetic reinforcement in retaining walls usually extends, at the critical positions, by 2% in FE analyses when the wall displaces by 1%, the LE analysis can be based on the tension mobilised at 2% strain. Afterwards, the foundation can be checked for equilibrium under soil strengths mobilised at 1% strain. Here, the limit which is being addressed in the LE analyses is the 1% deformation limit (SLS) rather than the collapse limit (ULS). Instead of a "factor of safety", the designer would independently check that fully softened soil would not collapse, and would provide for ductility.

CONCLUSIONS

"Symposium" means coming together. It is time to try to weave together some of the separate strands of thinking about the design of reinforced soils.

Experts can contribute to practical design by transferring ideas from FE to LE analyses in the manner described above, and then by testing this strategy in practice. There is hope that deformations can be accounted for by LE analyses based on mobilisable strengths.

Much more needs to be spoken about ductility. We have participated in a development of a material concept which is widely regarded as capable of absorbing a great deal of coincidental deformation, of behaving plastically in other words. But reinforced soils techniques often seem to based on potentially brittle details, and to neglect the effects of local corrosion on the reduction of overall extensibility, for example. Are we prepared to declare some ductility allowance, such as a minimum 5% extension to rupture following the acquisition of the design working strength? Or do we prefer to admit that some designs are brittle?

Composite materials concepts such anisotropic cohesion, and structural concepts such as anchored bulkheads, also need to be related. Resources seem to be being wasted in walls or steep slopes with compliant facings, in the sense that large earth pressures are not found to act on the facing. The frictional interaction between reinforcements and soil in the "active" zone can certainly lead to the strong reduction of lateral pressure on the facing, as would be well understood by a composite approach. This understanding needs to be transmitted in a simple way to structural engineers who have only learned the bulkhead approach.

Finally, we must draw together the concerns of technology with those of the environment. In particular, we must not set rules which make it unnecessarily hard for designers to achieve "green" faces. Vegetation, and the avoidance of harsh external lines through sculptured faces, are to be welcomed. The use of reinforcement techniques to achieve efficiency in landfill design is also to be welcomed.