Earth Reinforcement, Ochiai, Yasufuku & Omine (eds) © 1997 Balkema, Rotterdam, ISBN 90 5410 833 9

Limit state design of reinforced soils

Malcolm D. Bolton
Cambridge University Engineering Department, UK

INTRODUCTION

Limit state design codes should aim to assist the designer in preventing failures by:
(i) defining the envelope of hazards which the structure must survive,
(ii) clarifying the various modes of failure which should be checked,
(iii) adopting a robust and holistic approach to the engineering, based on sound principles such as equilibrium but which also includes good connection details, construction practices, and quality control procedures.

The new British Code of Practice for Reinforced Soils, BS 8006, claims to have achieved this. It has made a valuable contribution, but it has also introduced some features which are inconsistent and inadvisable. This is discussed below.

DESIGN SCENARIOS

A design code should generate design scenarios which are both onerous and believable, e.g. severe loads, high water tables, poor quality fill, aggressive groundwater, corroded or degraded reinforcement. This requires judgement, both from the code-drafter and the code-user. High standards demand more than a check-list of actions. Guidance must be given on the precise design conditions. This is sometimes presented as a matter of selecting "load combination factors", but it is more helpful to create "design scenarios".

For example, consider the question of whether a reinforced soil structure should be designed on the basis of the worst believable degradation of the reinforcement, or the worst credible surcharge, or the 1000 year earthquake, or all three simultaneously. It is expensive in materials (but cheap in design fees) to design every structure for the worst corrosion ever seen anywhere, *and* the biggest earthquake ever experienced anywhere. It saves in construction costs (but is more expensive in design time) to take a more scientific view based on data relevant to the particular site.

The aim should be to ensure firstly that material deterioration does not cause failure until the design life of the structure has been exceeded, and secondly that when failure does occur it will not be catastrophic. A more accurate (less conservative) design always demands more information - on groundwater chemistry, and the actual deterioration of similar materials in the ground, for example. However, some essential aspects will remain a matter of judgement - the likelihood of road salt or spillages reaching vulnerable materials, or the future performance of a new polymer. The same qualifications apply to the information which can be obtained on seismicity or other live loading for a structure at a given location.

BS 8006 issues a check-list of actions together with some strong prescriptive rules, such as the required thickness of sacrificial metal to compensate for corrosion in various steels, the presumed efficiency of different classes of joint, and the electrochemical properties of fill in relation to the selection of reinforcement materials.

It does not deal with seismicity. It is, however, interesting to reflect on the general principles which might apply to the specification of design scenarios which could include such exceptional environmental actions. Consider the following proposal.

In terms of loading, a retaining structure faces hazards of three main types - compaction during its construction, live load surcharge in normal service, and rare environmental incidents such as a severe earthquake or erosion at the base due to flooding. In terms of material properties, a structure tends to deteriorate during its design life. The designer should derive design values of superimposed surcharge and material deterioration, each of which could reasonably be expected to represent the limit of the hazard during the design life of the structure. A design flood leading to erosion, and a design earthquake will also be specified which could reasonably be expected to occur only once in the design life of the structure.

These influences should be considered to co-exist in certain patterns as exemplified below. The structure should be able to survive construction and service life loading together with lifetime deterioration without exhibiting a serviceability failure. It should not collapse suddenly due to a design earthquake *or* a design flood even while carrying design surcharge and after design lifetime deterioration, but either of these conditions may cause permanent deformations which represent a serviceability failure. The structure, its materials and connections must, however, be designed to maintain sufficient ductility and continuity to survive all credible design scenarios without collapsing suddenly.

Although this particular wording may be contentious, it should be clear that it does set a standard. It draws attention to the avoidance of sudden collapse, which will lead the designer to consider how to maintain continuity and ductility of the structure. It specifies believable but extreme events with return periods equal to the design lifetime of the structure, rather than extraordinarily severe events which almost never happen. It then asks for the design to assure survival under a simple combination of thcsc cxtrcmc cvcnts. If thcrc is no relevant database from which probabilities of failure could be derived, they can play no part in the process of decision-making. However, the recommended approach demands that sudden collapse through loss of stability (ULS failure) should always be preceded by obvious deformations (SLS failure) which would give the owner time to evacuate those in danger.

MODES OF FAILURE

It has become natural in limit state design codes to discriminate between deformation (serviceability limit states, SLS) and collapse (ultimate limit states, ULS). Unfortunately, the original understanding of the distinction has been eroded. Many structural engineers actually design to limit deformations in steel and concrete, but erroneously say that they are "designing against ULS". It seems important for the designers of reinforced soil structures to be very clear about the difference.

Soil is inherently brittle, especially when it is well-compacted, stiff or dense. It will mobilise a peak strength (expressed as a secant angle of internal friction) ϕ_{max} at about 1 or 2% axial strain in a triaxial test, which will then drop to the ultimate strength ϕ_{crit} as the soil ruptures on slip bands. Only about 5 particle diameters of slip will be sufficient to "fully soften" the soil in this way. If a soil structure has been designed to mobilise $\phi > \phi_{crit}$ at collapse, then if it collapses it will collapse violently with mass-accelerations, since its resistance will be dropping as it deforms. The new British Code of Practice on Earth Retaining Structures, BS 8002, regards this as inherently dangerous: it demands that ${}_{design}\phi \leq \phi_{crit}$.

BS 8006, takes a different approach - it was written by a different committee, with no liaison. It permits ϕ_{max} in ULS calculations for reinforced soil zones on the grounds that the reinforcement will prevent slip bands forming, and ϕ_{crit} in unreinforced zones. But it also factors up all soil densities and live loads so that in reality much smaller angles of friction would be mobilised at failure if failure actually occurred. It is self-evident that soil can not ever weigh 30 kN/m^3, which is the typical unit weight derived for granular fill after applying a partial factor of 1.5 as BS 8006 requires for retaining walls. BS 8006 can not therefore be described strictly as a limit state design code. By the time the partial factors have been applied, the design scenario is not believable; the code has introduced arbitrary safety factors which are unrelated to the parameters to which they have been applied.

Soil is also inherently compliant; it needs to suffer large strains if it is to mobilise even its peak strength. BS 8002 took the view that a displacement / height ratio of 0.5% was as large as the owners and users of retaining walls would generally accept. It deduced that a 0.5% strain in a triaxial test, or a 1% shear strain in a plane test, represented the serviceability limit for soils. It went on to propose for moderate to good soils that the angle of internal friction at 1% shear strain could be presumed to be $\phi_{1\%} = \tan^{-1} (\tan \phi_{max}/M)$ where the mobilisation factor M = 1.2 would be sufficient to guard against such strains. In that case, we can apply the constraint ${}_{design}\phi \leq \phi_{1\%}$. What has then been derived in BS 8002 is a design soil strength which satisfies both safety (ULS: ${}_{design}\phi \leq \phi_{crit}$) and serviceability (SLS: ${}_{design}\phi \leq \phi_{1\%}$) constraints. It is a mobilisable soil strength.

Interestingly, BS 8006 adopts a similar approach to the selection of a mobilisable reinforcement strength. The design tensile strength is taken to be the lower of the ultimate (or creep rupture) strength factored down only for uncertainty, and the tension mobilised at the serviceability limit of extension in service (typically 0.5% to 1%). The BS 8002 approach to the mobilisable strength of soil, and the BS 8006 approach to the mobilisable strength of reinforcement are, most appropriately, compatible.

Finally, bond failure must be averted. If the roughness of the reinforcement exceeds the mean particle size it may be taken that the interface friction matches the internal friction of the soil, so that $_{design}\mu = {}_{design}\tan\phi$. Otherwise, shear box tests have to be performed sliding soil against a block of reinforcement fixed in the lower half of the box, to give a smaller value of interface friction. This approach is set out in BS 8006, which recommends factoring down the interface friction by 1.3, bringing the friction parameter down almost exactly to the $_{design}\tan\phi$ value which could be derived via BS 8002. BS 8006 also permits pull-out tests to determine an equivalent coefficient of interface friction μ^* which can take advantage of arching of stress onto the reinforcement due to dilation around strips or rods. It does not, however, seem to be certain that an isolated pull-out test can fairly represent the degree of arching in the field, especially in earthquake loading.

It is interesting to compare earth pressures generated by the alternative design approaches in BS 8002 and BS 8006. Consider a well-compacted fill with typical properties $\phi_{max} = 44°$ at $\gamma = 20$ kN/m^3, and $\phi_{crit} = 35°$. BS 8002 would derive $_{design}\phi = 35°$, and $_{design}K_a = 0.27$. The mobilisable active earth pressure at 5 m depth would then be 27 kPa. BS 8006 would use $\phi_{max} = 44°$ to get $K_a = 0.14$ but would apply this to soil with $\gamma = 30$ kN/m^3 to give an earth pressure in reinforced fill at 5 m depth of 21 kPa. This would be later factored up again by 1.1 if the consequences of failure would be life-threatening, so reinforcement capable of withstanding 23 kPa would be provided. This is not very different from the BS 8002 result.

A much larger discrepancy is revealed when dealing with the unreinforced backfill. BS 8006 then imposes $\phi_{crit} = 35°$ to get $K_a = 0.27$, whilst still factoring up the density as before; the design earth pressure at 5 m becomes 45 kPa which would correspond to an earth pressure coefficient of 0.45 in real soil. BS 8002 would derive the same earth pressure as before, 27 kPa. Considering the lack of clarity which already exists regarding the correct earth pressure coefficients to use with real soil, it seems a little unfortunate that BS 8006 chose to factor up soil density instead of factoring down soil strength.

BS 8006 shows a complete compendium of limit equilibrium analyses, with valuable references. It seems to demand some redundancy in the making of equilibrium checks, but perhaps this is no bad thing. Its main weaknesses seem to be:

- soil strength data may be fitted either by tangent parameters (c', ϕ') or by a secant peak angle of friction ϕ_{max} - different partial factors are proposed for these alternative approaches, and this is inconsistent;
- soil density is strongly factored up to an unbelievably high value - this undermines reliability, produces unbelievably high earth pressures, and causes difficulty in dealing with effective stress analyses under water where the unit weight of water must equally be factored up (though this seems not to have been recognised);
- there is no recognition that soil also needs to strain to mobilise strength - peak soil strength is used in design calculations for earth pressures and a small strength reduction factor of 1.35 is advocated in calculating the bearing pressure of soft clays whereas deformation considerations might have led to a mobilisation factor between 1.5 and 2.0 (this unconservatism presumably compensates for the excessive conservatism referred to immediately above);
- there is neglect of the well-known question regarding the impossibility of mobilising bond on both top and bottom surfaces of sheets of reinforcement;
- although certain ductile metals are listed as usable, and there is an admirable exposition of connection details for example, the vital importance of ductility and continuity are under-emphasised.

A very simple design approach now suggests itself, however, based partly on BS 8002 and partly on BS 8006. Let us design so that the strength of soil nowhere exceeds $_{design}\phi$ and so that the strength of reinforcement nowhere exceeds $_{design}T$, each of which satisfy criteria both of safety and serviceability. Reduce the frictional bond to $_{design}\mu$ where measurements on smooth materials can be used to define a lower bound. Satisfy rigorous but believable design scenarios, but do not apply any other factors of safety. Instead, simply demonstrate in each scenario that an equilibrium free-body diagram can be constructed for the reinforced soil zone acted upon by backfill pressures and surcharges, and held in place by mobilisable bearing pressures on its foundations. Then, demonstrate that internal stresses could be mutually in equilibrium without exceeding the design strengths discussed above. Wedges or other failure mechanisms, earth pressure diagrams and stress analyses - indeed any validated limit equilibrium analysis - could then be used to verify that the structure has internal equilibrium. If there is a stress distribution inside the structure which satisfies equilibrium under these conditions, the structure can neither collapse nor deform excessively.

ENGINEERING

It is very much to be welcomed that BS 8006 contains a good account of connection details, construction methods and quality control procedures. The reliability of the finished product is strongly dependent on the intelligent management of the design process and the skill of site personnel. Two practical issues, which are fundamental to reinforced soils, are not given sufficient significance: compaction pressures in fill, and the ductility of reinforcement and connections.

Reinforced soil structures work by trapping high earth pressures in a zone which is laterally confined by reinforcements which must extend; they thereby develop tension which maintains stability. This fundamental mechanism comes into play whether the structure is resisting gravity, surcharge or earthquake. Compaction pressures locked into retained fill during construction are not inherently a bad thing. They "proof-test" the structure against some future surcharge in service, or an earthquake, either of which could create identical earth pressures in a quite different way. Only if the earth pressures during construction far exceed those predicted for design live loading or earthquake loading will it be advisable to consider how to reduce them.

Ductility and continuity are essential to a safe structure. There is danger in permitting sacrificial steel around connections which are already weak; the ultimate tensile strength of the connection might then be smaller than the yield strength of the parent reinforcement. Such a tie would fail without much elongation. Progressive failure has been observed in centrifuge tests on reinforced soils with brittle ties. It seems highly desirable to set a ductility target in terms of the extension to failure of reinforcement / connection / panel systems. A stretch of 5% to fracture of deteriorated materials, following the achievement of the design value of reinforcement tension, would alert the authorities and avoid catastrophe in the event of an unforeseen overload.

CONCLUSION

Limit state design offers a flexible framework within which clear standards can be set. The new UK Code, BS 8006, has made some useful contributions. It is much preferable to use the concept of mobilisable strength rather than adopting arbitrary partial factors. Mobilisable strength is simpler to understand, easier to apply, and more faithful to the fundamental principles of mechanics. It encompasses both deformation and strength criteria. All that is then required are realistic but severe design scenarios, and a rigorous check for global and local equilibrium.

BIBLIOGRAPHY

1. BSI: *'Code of Practice for Earth Retaining Structures'*, BS 8002, 1994.
2. BSI: *'Code of Practice for Reinforced Soils'*, BS 8006, 1996.
2. Bolton, M.D.: 'What are partial factors for?', *DGF Bulletin* 10 / 3, 565-583, Danish Geotechnical Society, Copenhagen, May 1993.
3. Bolton, M.D.: 'Codes, standards and design guides', *Retaining Structures*, Thomas Telford, 387-402, 1993.
4. Bolton, M.D.: 'Design methods', *Prediction and Performance in Geotechnical Engineering*, 50-71, Thomas Telford, 1993.