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The paper was published in the proceedings of the 20th International Conference on Soil Mechanics and Geotechnical Engineering and was edited by Mizanur Rahman and Mark Jaksa. The conference was held from May 1st to May 5th 2022 in Sydney, Australia.

Global failures of certain reinforced earth slopes

Ruptures globales de certains talus en terre armée

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ABSTRACT: Relatively steep manmade slopes and embankments are a common feature of modern developments, maximising available land use in constrained sites and minimising land takes for transportation links. The British Standard Code of Practice for Strengthened/Reinforced Soils and other fills, BS 8006, underwent a revision in 2010, with guidance on soil strengths for use in design of those slopes. Since that time a number of global failures have occurred in reinforced earth slopes, where either peak strengths have been used to assess potential global instabilities in cohesive soils, that were prone to softening and progressive failure, or pore water pressures have not been adequately considered. A number of case studies of such failures will be presented, with recommendations for appropriate ground models for assessment of potential global instabilities.

RÉSUMÉ : Les pentes et les remblais artificiels relativement raides sont une caractéristique commune des développements modernes, maximisant l'utilisation des terres disponibles des sites limités et minimisant l'occupation des terres pour les liaisons de transport. Le British Standard "Code of Practice for Strengthened/Reinforced Soils and other fills", BS 8006, a subi une révision en 2010, avec des directives sur la résistance des sols à utiliser dans la conception de ces pentes. Depuis lors, un certain nombre de ruptures globales se sont produites dans des pentes en terre armée, où soit les résistances maximales ont été utilisées pour évaluer les instabilités globales potentielles dans les sols cohésifs, qui étaient sujets à un ramollissement et à une rupture progressive, soit les pressions interstitielles n'ont pas été suffisamment prises en compte. Un certain nombre d'études de cas de telles ruptures seront présentées, avec des recommandations pour des modèles de terrain appropriés pour l'évaluation des instabilités mondiales potentielles.

KEYWORDS: Reinforced earth slope failures.

1 INTRODUCTION

The use of strengthened or reinforced soils and other fills has become widespread over the recent decades, allowing relatively steep slopes/retaining walls to be formed without using traditional retaining wall construction, on both constrained sites and for transportation links, by allowing reduced land takes.

Specifically, we are primarily concerned with reinforced soil or earth slopes effectively formed as embankments with soils placed and compacted as fill, reinforced with geogrids or similar. The reinforcement allows a steeper angle of slope to be formed than could be safely achieved with the soil alone.

In terms of stability, the design of such slopes has to consider both the potential for internal failures through the reinforced soil but also the potential for external instability. The potential global failure modes that should be considered include those effectively around the block of reinforced soil, such as sliding at the base of, or rotational slips around the reinforced soil.

In the UK, particular guidance on the design of reinforced earth slopes is provided in the British Standard Code of practice for strengthened/reinforced soils and other fills, BS 8006-1 (2010) (with Amendment 1 published in 2016) (BS 8006). BS 8006 uses a limit state approach to design, with partial factors applied to external disturbing forces and material strengths to provide a margin against instabilities that are ultimate limit states of collapse.

1.1 Failure states

BS8006 recommends that both internal and external stability are considered, commenting that the assessment of external stability involves consideration of the stability of the reinforced soil mass. Further, the overall, rotational or global stability of the reinforced soil mass has to be checked using slope stability procedures as described in Eurocode 7: Geotechnical Design - Part 1: General Rules, BS EN 1997-1:2004 (Eurocode 7).

In respect of reinforced slopes, BS8006 defines shallow slopes as having a face angle of less than or equal to 45° from horizontal and steep slopes as having a face angle of greater than 45°. Specific guidance on the ultimate limit states that should be considered for external stability of steep slopes include forward sliding and a slip failure around the reinforced soil block

(see Figure 1). That said, BS 8006 comments that all potential slip surfaces should be considered, including those passing wholly external to the structure using appropriate analysis methods. The length of the reinforcement back into the slope should be sufficient to ensure such failures do not occur.

1.2 Design strength

In respect of design strength, in general BS 8006 comments that resisting forces will be a function of several variables including pore water pressure and soil shear strength. Their characteristic values are determined as a cautious estimate of the value affecting the occurrence of the limit state. These are reduced by a material factor, of prescribed value, to produce the design strength. As with any other geotechnical problem, due account should be taken of any variation of soil shear strength with time over the selected design life.

In respect of reinforced soil slopes, BS 8006 notes that in general soil shear strength is defined by the effective shear strength parameters of the angle of internal shearing resistance (ϕ') and apparent effective cohesion (c') or the undrained shear strength parameter (c_u), although the effective cohesion is only applied to cut slopes formed in overconsolidated clays.

The mobilised shear strength also depends on the strain properties of the fill/soil and reinforcement. In many soils, increasing strains under compressive loading result in increasing mobilised shear strength up to a peak angle of internal shearing resistance (ϕ'_p), generally involving dilatant behaviour. However, in strain softening soils where strains are increased above that required to mobilise such a peak strength, softening can occur and the available shear strength can reduce so the soil shears at a constant volume. A similar reduction in available shear strength can also occur in certain soils prone to softening, independent of strain. The mobilised angle of shearing resistance in these conditions can be referred to as constant volume, critical state or softened (ϕ'_{cv}). An example of the stress-strain relationship for a soil exhibiting a peak and constant volume shear strength behaviour is included in Figure 2.

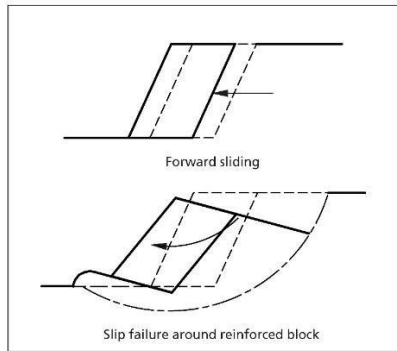


Figure 1. Examples of certain ultimate limit states (BS 8006).

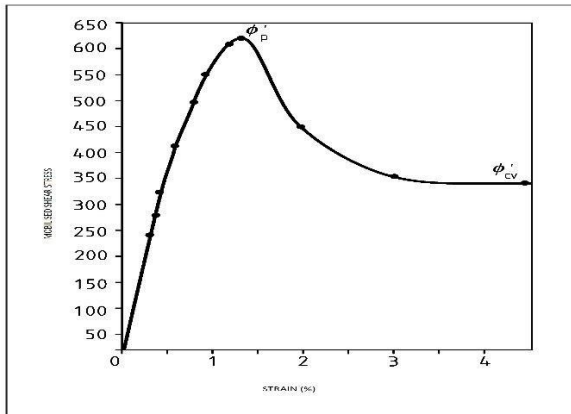


Figure 2. Example of a stress/strain relationship for a soil (BS 8006).

For the design of steep reinforced soil slopes incorporating multiple layers of reinforcement that are considered to be purely frictional and where axial strains are generally 1% or less, BS 8006 generally recommends use of the peak angle of internal

shearing resistance, but comments that for greater strains the mobilised angle of shearing resistance may be assumed to tend towards the constant volume value, such as in shallow slopes.

In summary, the use of peak strengths is generally appropriate for the internal design of steep reinforced soils slopes. However, in respect of external slope stability assessments that are effectively not in the reinforced soil, the design should consider larger strains and adopt the constant volume angle of internal shearing resistance of the soil being considered.

1.3 Drainage

The design of reinforced earth slopes also has to consider the pore water pressures that could occur during the life of the slope. BS 8006 comments that for slopes, the loads imposed on the soil reinforcement will be increased if positive pore water pressures are allowed to develop, but notes that the development of adverse pore water pressures in reinforced fill slopes can be prevented by the installation of appropriate drainage.

In respect of the drainage of reinforced soils slopes, BS 8006 recommends that either measures should be taken to ensure that the fill does not become waterlogged, or that any water pressures assumed in design are not exceeded. BS 8006 comments that where the fines content of a fill is greater than 10% the effects of pore water pressure both during construction and during the service life of the structure should be considered. Consideration also has to be given to pore water pressures, including those arising below embankments from the construction of slopes.

1.4 Instability

Where the external stability assessments recommended by BS 8006 indicate that one or more modes of potential collapse exist, a number of alterations to the design can be considered including:

- reducing the slope face angle;
- increasing the length of the reinforcement into the slope;
- using better quality fill of higher strength; and
- introducing drainage to reduce pore water pressures.

2 CASE A

Case A is a steep reinforced earth slope failure that we investigated the cause of. This involved a constrained city site where a redevelopment required the construction of an embankment, using site won material. On one side the embankment was to be formed as a steep reinforced earth slope, due to space constraints on this site.

The design height of this slope was 11m and the length was over 50m. This slope was reinforced with geogrids at the base of the embankment and at 1m centres of height that were 9m long, save for the final 4 layers, that were shorter (see Figure 3). The front face was formed at around 50° from horizontal.

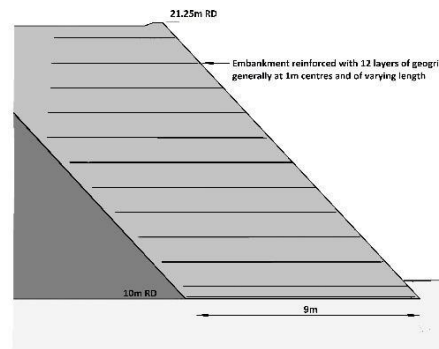


Figure 3. Design cross section of the reinforced earth slope.

The site of this reinforced soil slope was underlain by around 1m thickness of River Terrace Deposits of Taplow Gravel (very dense, very sandy Gravel) over London Clay (stiff, heavily overconsolidated silty Clay). The groundwater was found essentially at ground level with a stream known to have formerly run across this site, which was essentially level.

This slope was formed of selected wet cohesive material or selected stoney cohesive material derived from reworked London Clay. The cohesive fill was compacted to at least 95% of the maximum dry density (heavy rammer method to British Standards), and benched into general cohesive fill placed previously behind the reinforced soil slope. The cohesive fill was compacted within moisture limits of the plastic limit -4% up to plastic limit or plastic limit +2%, with an undrained shear strength of 120kN/m².

Some 6 months after construction was completed, this reinforced earth slope suffered a global failure following a period of heavy rainfall.

A review of the analysis used in the design of this reinforced earth slope prepared by others showed that all the cohesive fill had been modelled for ultimate limit state global failures assuming a peak angle of internal shearing resistance of 26° with an apparent cohesion of 2kN/m². Further, the Taplow Gravel was assumed to have a peak angle of internal shearing resistance of 35° and the underlying London Clay a peak angle of internal shearing resistance of 23° with a cohesion of 2kN/m². The reinforced soil was assumed to have no pore water pressure. With these parameters, the designer modelled potential circular slips passing below the reinforced earth block and considered all to have an adequate margin against collapse following BS 8006. The available evidence indicated that when the embankment was placed all the earthworks conformed to the earthworks specification set down by the designer.

The slope failure comprised a global rotational slip with a bulging at the toe sliding forwards and a considerable back-scarp forming around 10m back from the crest of the slope, with ongoing movement at a relatively slow rate for around 2 months, before the slope was removed. The failure was apparently wholly within the cohesive fill with the shear plane passing between the basal reinforcement and the next layer up.



Figure 4. View of back-scarp.



Figure 5. View of toe bulge.

We assessed the stability of the reinforced slope applying no partial factors on the input parameters using limit equilibrium methods. With no groundwater in the slope, this indicated a factor of safety of the order of 1 where a constant volume angle of shearing resistance was adopted (21°) with no cohesion in the cohesive fill, for a critical slip starting close to the toe, passing between the lower two layers of reinforcement and up through an arc point about 10m back from the crest. If any allowance for groundwater pressure is included then the factor of safety was reduced to less than 1.

The critical slip was consistent with the slope failure that occurred and the conclusion was the failure that occurred was expected based on the reinforced earth slope design that had been constructed. The adoption of relatively high peak strengths in the design for the cohesive fill was not considered to be appropriate when assessing potential global instabilities where the soil would not be restrained by the geogrid reinforcement and strains could be expected to exceed those required to mobilise the peak strength. This cohesive fill would also be expected to be prone to softening upon inundation with water. The vertical spacing and length of the geogrid reinforcement adopted in the design effectively allowed a slip surface to form that was not intersected by the reinforcement. In the circumstances the design should have adopted the constant volume angle of internal shearing resistance of the cohesive fill, when assessing potential global instabilities.

Further, notwithstanding the likelihood of some pore water pressures developing in the reinforced earth slope at times of significant rainfall, in light of the stream at this site, without drainage of the foundation the embankment could be expected to have some pore water pressures developing within the slope and this should have been considered in the design.

3 CASE B

This involved a further reinforced earth slope failure that we were instructed to investigate the cause of. The project of relevance involved a substantial transportation development where an attenuation pond was to be formed with reinforced earth slopes, due to limited land availability. This pond had an area of some $3,000\text{m}^2$ and was formed in an oversized open cut excavation, with permanent side slopes then formed using geogrid reinforced fill.

The reinforced earth slopes were about 5m high with a total length of some 320m and a face angle of 45° . The main geogrid reinforcement comprised 9 layers at a vertical spacing of 0.6m and extending back up to 6m from the face of the slope and then a further 8 secondary layers of geogrid extending 2m from the face, located between the main geogrids (see Figure 6). In the permanent state, the pond was to be lined with a waterproof liner.

The natural ground at this site was Glacial Till (generally firm to stiff silty slightly sandy Clay with thin layers of loose Sand) to up to 10m depth, over Mudstone bedrock, with groundwater found about 1.5m below ground level. The site was relatively flat, prior to the development.

The original ultimate limit state design analysis for this slope prepared by others assumed that the reinforced earth slopes were to be formed with granular fill with a peak angle of internal shearing resistance of 35° with no pore water pressure in the fill. The underlying natural Glacial Till was modelled with a peak angle of internal shearing resistance of 28° .

In the event, the reinforced earth slopes were constructed with site won Glacial Till. Laboratory tests carried out during the construction on samples of remoulded Glacial Till found a peak angle of shearing resistance of about 25° . The construction records indicate general conformance with the earthworks specification with the construction completed over a period of some 4 months.

Around a month after the whole construction was completed, a back-scarp appeared about 5m behind the crest of part of the reinforced earth slope that was first constructed. The movement of this slope was monitored for the following two months with a total toe bulging of about 700mm occurring with a back-scarp of around 500mm high forming (see Figures 6 and 7). This was a rotational failure of the natural soil around the block of reinforced soil, with the slip surface passing through the unreinforced ground behind and below the reinforced earth slope.

We assessed the stability of this slope using limit equilibriums but with no partial factors in this model. This indicated that if the global stability of the ground around the reinforced earth block was considered with groundwater at 1.5m depth using the peak strengths adopted in the original design, then the margin of safety of this slope against rotational slips reduced towards 1. On this basis, any reduction in the strength of the soil post-peak, would be expected to lead to a failure of these slopes. Even with the lower peak strengths reported from tests during construction the slope would be expected to fail.

We consider that the design of these slopes adopted overly optimistic shear strength for the primarily clay subsoils. The assessment of potential global instabilities around the reinforced earth block in the design of these slopes should have adopted a constant volume angle of internal shearing resistance for the Glacial Till (likely to be of the order of 20° to 23° , depending on plasticity) and made reasonable allowances for groundwater in the reinforced soils slope and surrounding ground.

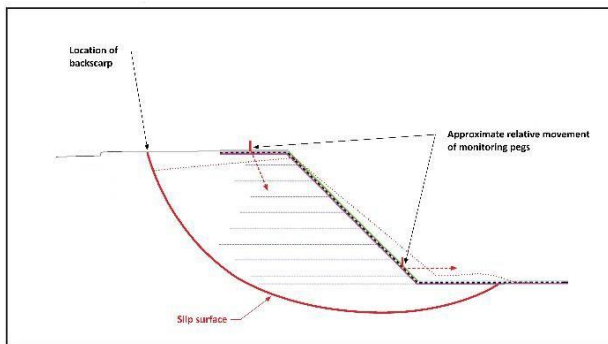


Figure 6. Section through the reinforced earth slope showing slip surface.



Figure 7. View of back-scarp and toe bulge.

4 CASE C

This project involved the development of a waterfront site where there was a global instability of the reinforced earth slopes formed to provide level platforms on this sloping site. This slope was approximately 400m long and up to 6.3m high, and formed with lime stabilised cohesive fill and coarse stone fill. The average face angle of the tiered face was 45° with geogrid reinforcement provided at 600mm vertical centres extending 6m back into the slope.

The site was underlain by some 5m of Alluvium (interbedded layers of very soft to firm silty Clay) over a about 4.5m of Marl Member (firm to hard Clay) over Limestone. The reinforced earth slope was founded on a thin layer of granular fill. The undrained shear strength of the Alluvium ranged upwards from 19kN/m² with a constant volume angle of internal shearing resistance of 20°. The angle of shearing resistance of the lime stabilised cohesive fill was measured as 34.5°. The design of this reinforced earth slope assumed a peak angle of internal shearing resistance of 26° in the Alluvium.

Construction of the reinforced earth slope took place over a period of some 4 months, with further terraces placed up to a height of 6m. Within a week of the final terrace having been placed a tension crack was observed in the ground behind the reinforced earth slope with a level difference of 200mm. Monitoring of the ground over a period of around a month reported up to 800mm lateral movement with back-scarp around 400mm high forming and uplift beyond the toe of the slope of over 100mm over a length of 30m (see Figure 8).

In the short term, the construction of the reinforced earth slope would generate significant pore pressures in the essentially incompressible pore water in the saturated low permeability Alluvium. Over time, as consolidation occurs, these pore pressures would reduce to equilibrium levels.

The relatively rapid construction of the reinforced earth slope would not be expected to allow drainage of the underlying Alluvium, leading to elevated pore water pressures reducing the effective strength of the Alluvium, and the available resistance to potential global slip surfaces.

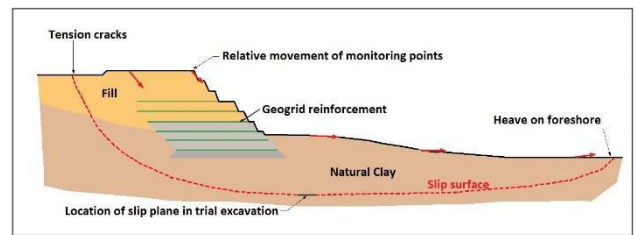


Figure 8. Section through failure of reinforced earth slope.

Even allowing for a peak strength in the underlying Alluvium, a limit equilibrium analysis of the global stability of the reinforced earth slope with no partial factors and with short-term elevated pore water pressures showed the slope would be expected have a factor of safety against instability of less than 1. Further, the softened layers of Clay in the Alluvium are likely to have a reduced peak strength tending towards the constant volume strength of this soil, which would further reduce the global stability of this slope.

In respect of the longer term design of this slope, with a reasonable allowance for groundwater, and the constant volume angle of internal shearing resistance, the factor of safety of the reinforced earth slope would be less than 1 and a failure could be expected to occur during the life of this slope.

We consider the design of this slope should have considered the elevated pore pressures to be expected in the Alluvium, and probably provided additional drainage of the Alluvium to relieve these pressures over appropriate construction periods. The longer term design should also have made appropriate allowances for groundwater and adopted constant volume shear strength in the natural subsoils below the reinforced earth slope.

5 CONCLUSIONS AND RECOMMENDATIONS

When designing reinforced earth slopes it is as important to assess potential external instabilities as the internal design of the reinforced soil. These assessments should adopt appropriate strength parameters for the soils, such as constant volume strengths for potential external instabilities, which may include failures through the fill within the reinforced earth that are unrestrained by the reinforcement. This may require suitable laboratory testing to assess the reduction in strength with increasing strain to ensure compatibility and also the consideration of the potential for softening of cohesive soils resulting in loss of strength.

This is particularly important when computer based models are being utilised to assess such potential failures, where appropriate parameters should be adopted for internal and external failures independently of each other.

It is also of key importance that the designs of reinforced earth slopes adopt appropriate pore water pressures in the underlying or surrounding ground, and in the reinforced soils as appropriate. This should also include consideration of the pore water pressures during construction.

Some designers have adopted peak strengths when assessing potential global failure mechanisms not involving reinforcement, where stains are relatively larger, soils can soften and the effective angle of internal shearing resistance of the soil at constant volume would have been the appropriate strength to use. Similarly, inadequate allowances have been made for pore water pressures, including during construction. These shortcomings have resulted in the global failures of a number of reinforced earth slopes.

6 REFERENCES

- British Standards Institution. 2016. Code of practice for strengthened/reinforced soils and other fills, BS 8006-1:2010+A1:2016.
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