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REINFORCED SOIL RETAINING WALLS - AN OUTLINE OF DESIGN METHODS AND SOURCES OF CONSERVATISM

Michael Dobie[1] CEng FICE
[1]Tensar International Limited, Jakarta Regional Office, Wisma Sejahtera Room 407, Jalan Letjen S Parman Kav 75, Jakarta 11410, Indonesia; PH +62 (21) 5490016; FAX +62 (21) 5490170; email: tensar@attglobal.net

Abstract
Design methods for reinforced soil retaining walls consist of three main elements: method of calculation, material parameters and factors. Each of these is considered in turn to examine where conservatism may exist, so that there is scope for reducing the cost of a structure. The method of calculation, especially for internal stability, may result in considerable conservatism if many assumptions are required to carry out the calculation. This can be minimised by using more realistic methods of calculation, which make optimum use of the reinforcement, but at the same time identify the effects of any weak points (for example low connection strength between reinforcement and facing). Material properties are required for the fill soils, the reinforcement and the interaction between the two. Soil tests are required to define the fill shear strength and unit weight. The drained soil shear strength is required, therefore it is most important that the correct soil test procedures are used, especially for finer soils (quick undrained tests are not suitable). The reinforcement strength must be determined as suitable for the full design life of the structure, taking into account that degradation continues throughout its service life, but this must be assessed using accelerated test methods. Any idea of switching to a definition of reinforcement strength based on short term strength would be most unwise, resulting in a structure of unknown long term performance and inadequate serviceability. As regards the factors used, these are defined partly to take into account uncertainties, partly to ensure a safe structure and partly to ensure adequate serviceability during the life of the structure. With regards to reducing conservatism in reinforced soil design methods, the main elements to be considered are the method of calculation, and ensuring that appropriate and adequate shear strength testing is carried out on the fill material. There is also scope for choosing suitable factors. If design methods are developed to achieve less conservatism compared to current methods, then it is important that serviceability checks are included to ensure that post-construction strain in the reinforcement is not excessive.

1 INTRODUCTION

Methods of design for reinforced soil structures have been in use for more than 30 years. Reinforced soil structures are divided into two main categories: retaining walls when the face angle is steeper than 70° to the horizontal normally with a concrete facing of some type, and steepened slopes when face angle is less than 70° with a vegetated finish. Design methods used for these two cases are similar in principle, but differ in some of the detailed aspects of the methods. In both cases the aim of the design method is to achieve a structure which is fit for purpose over its design life, in terms of both stability and deformation, at the same time being an attractive solution from the point of view of cost. It is a common observation that, despite their popularity, reinforced soil structures are conservative in their design, and that there should be ways of reducing cost further. The aim of this paper is to examine the design of reinforced soil retaining walls, for which there are many published methods, in order to examine where conservatism may be present, so that there may be scope for reducing cost. The discussion is partly based on methods and practice currently observed or used in China.

2 OUTLINE OF THE DESIGN APPROACH

The reinforced soil retaining wall design methods discussed in this paper are based on limiting equilibrium principles, which are used for the majority of published methods. The design is carried out in two stages. Firstly an external stability analysis is carried out, which is used to determine the overall dimensions of the reinforced fill block, namely dimension B as shown in Figure 1. The external stability check is essentially a gravity retaining wall calculation, and is much the same in all codes and guidelines. This part of the design procedure fixes the length of the reinforcement, which contributes to cost. It should be noted that in many methods there is a limit on the ratio B/H as shown in Figure 1, and that this limit will often determine the reinforcement length rather than any other calculation, especially in the case of good quality fill and good foundation conditions.

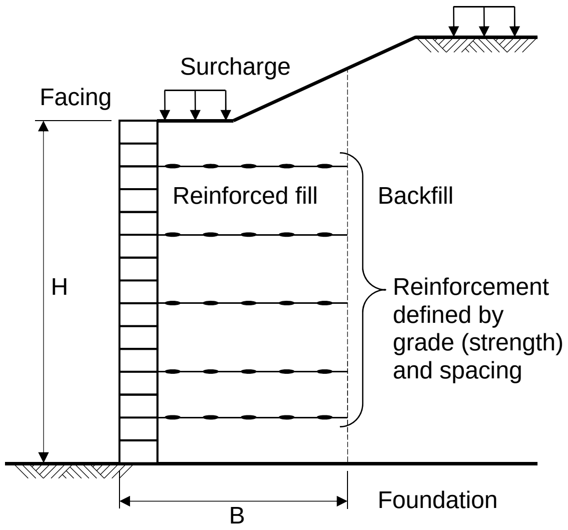

Figure 1. Reinforced soil structure main elements

The second stage of the calculation is to examine internal stability, to ensure that the layout of reinforcement (grade/strength and vertical spacing) is sufficient to meet the design requirements. The internal stability calculation should also take into account design features such as the connection strength between the reinforcement and the facing, variable reinforcement design strength and earthquake loading. Clearly the internal stability calculation has a significant influence on cost.

For both external and internal stability calculations, any "method" of design may be divided into three main elements: method of calculation, material parameters and factors. These are outlined in more detail in Table 1, and the discussion which follows considers each element in turn, especially in terms of conservatism which may exist either intentionally or possibly through poor practice, and therefore provide opportunities to reduce cost.

Element of design method	Details	Comments
Method of calculation	Method of calculating forces and stresses in order to make a design, covering both external and internal stability	For external design, most methods are the same, but for internal design there are significant differences
Material parameters	- Soil parameters - Reinforcement parameters - Interaction between soil and reinforcement	Material parameters should be measured using appropriate test methods and assessed as suitable for design
Factors	- Safety factors - Partial load and material factors - Wall friction angle on back of reinforced soil block - Inclination factors in bearing capacity - Soil strength definition	Factors ensure the margin against failure of the structure, and define some important design parameters

Table 1. The main elements of a reinforced soil design method

3 METHOD OF CALCULATION

The two main stages used for reinforced soil retaining wall calculations are outlined in Section 2: external and internal stability. External stability calculations consider that the reinforced fill block acts as a gravity retaining wall. Calculations are carried out to check sliding on the base, bearing capacity and eccentricity (or overturning). These calculation methods are well established, and there is little source for conservatism, especially if limits on the ratio B/H are defined or imposed by the method (typically in the range 0.5 to 0.7). However there is one important issue with regards to the bearing capacity calculation, namely the use of the inclination factor. This is outlined in Table 1 under "factors", and is discussed in Section 5.

3.1 Tie-back wedge method

The internal stability calculation is normally carried out by one of two methods: tie-back wedge or two-part wedge. The majority of published design guidelines use the tie-back wedge method, where design is generally based on assuming a single internal failure mechanism (see Figure 2), which requires many assumptions to be made as described by Dobie (2011). Inevitably assumptions result in conservatism, which is very much the case when designing for a connection strength (between the facing and the reinforcement) which is lower than the reinforcement strength, as well as adding earthquake forces to the method of calculation.

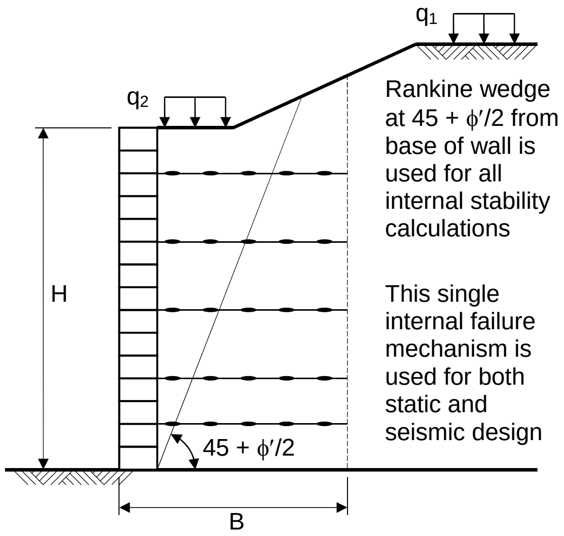

Figure 2. The tie-back wedge internal stability calculation method (as used in AASHTO)

3.2 Two-part wedge method

The basis of the two-part wedge method of analysis for internal stability is shown on Figure 3. The geometry is typical of reinforced soil retaining walls, but the method of analysis can incorporate all features shown without the need for any simplifying assumptions. The method of analysis is that of limiting equilibrium, but with the important requirement that any mechanism used should be admissible (ie. can actually happen) and that all forces associated with that mechanism should be taken into account. The aim of the calculation is to make sure that the resistance provided by the facing and reinforcement which is intersected by Wedge 2 ($T_1 + T_2 + T_3$ as shown on Figure 3) is sufficient to ensure stability of the two wedges.

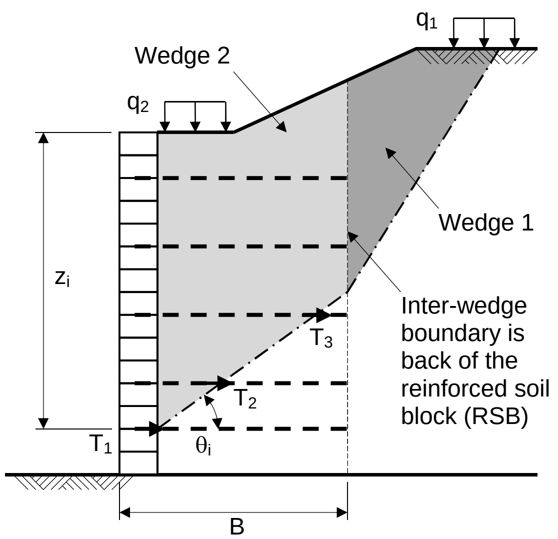

Figure 3. The two-part wedge internal stability calculation method (outline)

An important difference between tie-back wedge and two-part wedge is that there is no assumption of the wedges which give the critical case. Instead a search is carried out, looking at the stability of a large number of combinations of wedges. This is normally done as shown on Figure 4 (left). For a specific value of z_i, various values of θ_i are used so that a "fan" of wedges is checked. z_i is then adjusted and the fan of wedges repeated. Normally z_i is chosen starting at the base of the wall (where z_i = H, the total wall height), then at each elevation where reinforcement intersects the facing or if a water level is present.

Special cases of two-part wedges are checked, as shown on Figure 4 (right). The first are wedges defined by the maximum possible values of θ_i which do not intersect reinforcement. This check is normally carried out between all pairs of reinforcement layers and ensures that vertical spacing does not become too large. Generally the critical case is the lowest wedge, but higher wedges may be critical if vertical reinforcement spacing is increased or large surcharges are present behind the reinforced soil block. The second check is sliding over the reinforcement, which is generally critical for the lowest layer of reinforcement especially when the fill/reinforcement combination has a low sliding interaction factor.

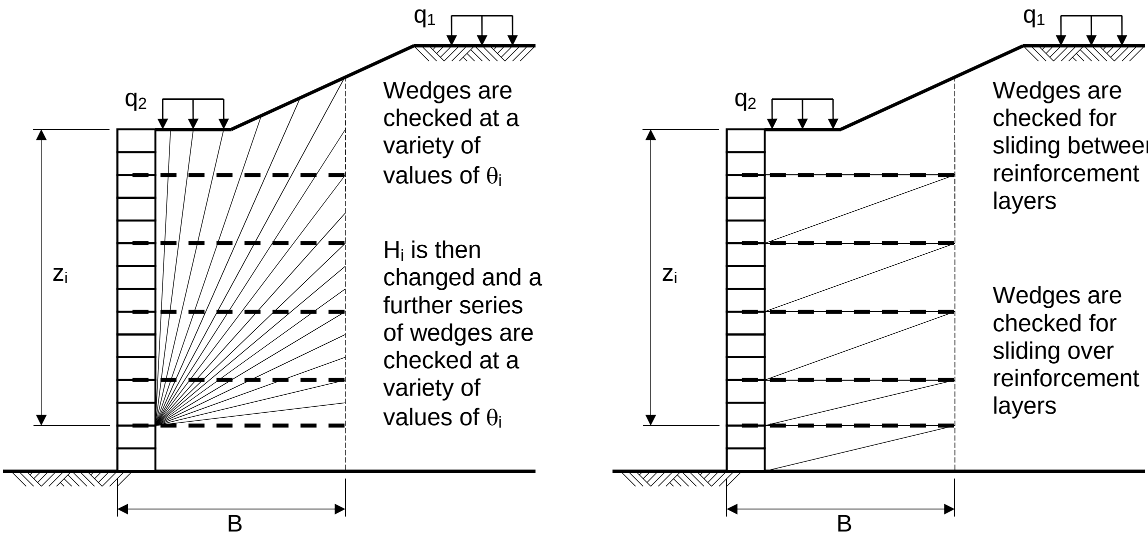

Figure 4.   Search routines used with two-part wedge method

The two-part wedge method as described above provides a comprehensive method of analysis of the internal stability of a reinforced soil retaining wall. To help visualise what might happen when a pair of wedges fail, the mode of failure is sketched on Figure 5 (left). As the wedges slide outwards, three layers of reinforcement are involved, each with a different failure mode:

Upper	Fails due to reinforcement pulling out of the fill
Middle	Fails by rupture of the reinforcement
Lower	Fails by pulling away from the facing combined with pull-out through the fill behind the facing

In addition to the three layers of reinforcement there is also failure through the facing, in this case by sliding between two of the facing blocks, which also provides resistance. However from the point of view of the reinforcement, it is necessary to assess the available resistance at three different locations, with three different failure mechanisms. This can be done by establishing a distribution of available resistance along each layer of reinforcement, as shown on Figure 5 (right). A full description of this method is given in Dobie (2011, 2012).

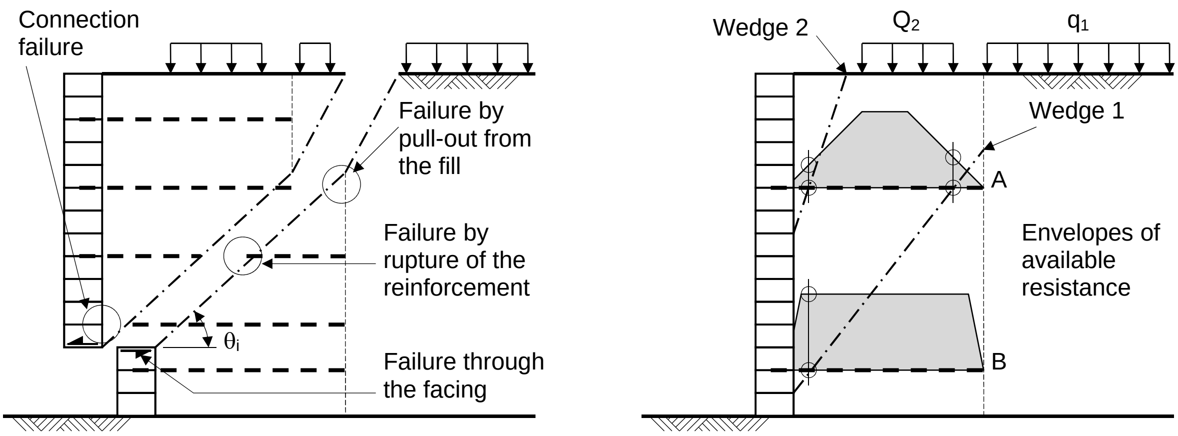

Figure 5.   Likely mode of failure of two wedges and definition of envelope of available resistance

The important result of using the two-part wedge method as described above is that less reinforcement will be required (for given conditions) compared to tie-back wedge, especially where the connection strength is less than the reinforcement strength, which is normally the case. The two-part wedge method may also be used to make an assessment of post-construction strain in the reinforcement.

4 MATERIAL PARAMETERS

The material parameters required for design are fill and foundation parameters, reinforcement strength and parameters which describe the interaction between fill and reinforcement in terms of sliding and pull-out.

4.1 Soil parameters

The soil parameters required for design are the soil strength and unit weight. The design of reinforced soil structures is carried out using the drained (effective stress) parameters, c' and ϕ'. This requirement is the same for all fill types, from coarse granular soils to fine cohesive soils. It is normal to assume that c' is either zero or very small (< 5 kPa). In order to establish suitable design values of c' and ϕ' it is necessary to obtain samples of the fills to be used, measured their index and compaction properties, then re-constitute test specimens at the expected on-site unit weight and water content in order to carry out strength tests. The method of carrying out the strength test should be appropriate for measuring c' and ϕ'. An example of test standards (JTGE40-2007) used in China is given in Table 2.

Test method	**Details**	**Suitable for c' and ϕ'**
T 0140-1993	Slow shear box test for clayey soil, rate of shear < 0.02 mm/min or time to failure > 50 $\times$ t_{50} from consolidation stage	Fine cohesive fill
T 0141-1993	Consolidated fast shear box test for clayey soil ($k < 10^{-6}$ m/s), rate of shear = 0.8 mm/min (especially for high fill slope which consolidates during construction)	
T 0142-1993	Unconsolidated fast shear box test for clayey soil ($k < 10^{-6}$ m/s), rate of shear = 0.8 mm/min (especially for cut slope)	
T 0143-1993	Direct shear test for sandy soil, rate of shear = 0.8 mm/min	Coarse granular fill
T 0144-1993	Unconsolidated undrained triaxial test (UU), strain rate 0.5 to 1.0 %/min	
T 0145-1993	Consolidated undrained triaxial test (CU), strain rate 0.05 to 0.1 %/min clayey soil & 0.1 to 0.5 %/min silty soil	Fine cohesive fill
T 0146-1993	Consolidated drained triaxial test (CD), strain rate 0.003 to 0.012 %/min	Fine cohesive fill/granular fill

Table 2. Soil strength tests to Chinese Standard JTG E40-2007

It is the author's observation that in China and many other countries, when testing granular fill, this is normally done with the T 0143-1993 method (direct shear test for sandy soil) using a small shear box (normally 60mm $\times$ 60mm maximum). This requires that particles larger than 4mm are removed, so if the fill has a significant gravel content, then the large particles will be removed, and the test will be carried out on a specimen which is not representative of the real soil, and generally this will result in a value of ϕ' being measured which is significantly lower than it should be.

In the case of fine cohesive fills, it is common for shear box tests to be carried out using the fast shear tests (either T 0141-1993 or T 0142-1993). These test procedures will generally give results somewhere between drained and undrained, typically with relatively high c and low ϕ. Such values are not suitable for reinforced soil design, and may well be highly conservative, especially for higher structures. The only shear box test suitable for measuring c' and ϕ' for fine cohesive fills is T 0140-1993, making sure that the specimen is fully saturated, then consolidated and then sheared very slowly. It is the author's experience that this test method is very rarely used in China, probably because it is very time consuming, and relatively expensive compared to the other shear box methods. In general incorrect shear test procedures are likely to result in low ϕ' values, and therefore conservative designs. By way of illustration, reducing ϕ' by 3° could easily increase reinforcement quantities by 10 to 15%.

4.2 Reinforcement parameters

Reinforcement strength is typically defined by a relationship of this type (following US practice), including some typical example design values:

$$T_a = \frac{T_{ult}}{RF_{CR} \times RF_{ID} \times RF_D \times FS} = \frac{100}{2.2 \times 1.1 \times 1.1 \times 1.5} = 25$$

where:

T_a	=	design strength	
T_{ult}	=	QC tensile strength	100
RF_{CR}	=	creep reduction factor	2.2
RF_{ID}	=	installation damage factor	1.1
RF_D	=	durability reduction factor	1.1
FS	=	factor of safety	1.5

The three reduction factors (RF) take into account three mechanisms of degradation which affect polymer geogrids: long term strength reduction (creep), installation damage and durability. The value FS is a general factor of safety to allow for uncertainties and risk. The European standard giving guidelines for the determination of the long-term strength of geosynthetics for soil reinforcement (ISO/TR 20431:2007) recognises three modes of degradation for geosynthetics, as given in Table 3.

Mode	Description	Example
1	Immediate reduction in strength, insignificant further reduction with time	Installation damage
2	Gradual, though not necessarily constant, reduction in strength	Durability
3	No reduction in strength for a long period; after a certain period, onset of rapid degradation	Creep

Table 3. Modes of geosynthetic degradation recognised by ISO/TR 20431:2007

In order to illustrate these three forms of degradation, Figure 6 shows all three, in terms of normalised load applied to the reinforcement versus time. The time axis starts at 0.000001 years (about 30 seconds) which is the typical duration of a short term quality control (QC) tensile test. The dotted line "creep rupture strength" is a typical relationship determined from a programme of creep testing, indicating creep rupture strength of 47% at 120 years. The solid black line above the creep rupture line is the "available resistance", which follows Mode 3 according to Table 3. This means that from the point of view of creep, the available resistance remains close to 100% until a short time before rupture, at which stage it reduces rapidly until the 120 year creep rupture strength of 47% is reached. So the question arises: "if 100% of the tensile strength is available until the end of the design life, then why not use this for design?". The reason that this is not done is that if a higher load is used, then the point of degradation will occur much earlier than the required design life. So if the reinforcement is loaded to 70%, then rupture would occur in about 0.0001 years, or about 1 hour. This is indeed the case; if a creep test is loaded to 70% of QC strength, then the rupture occurs very soon after applying the load.

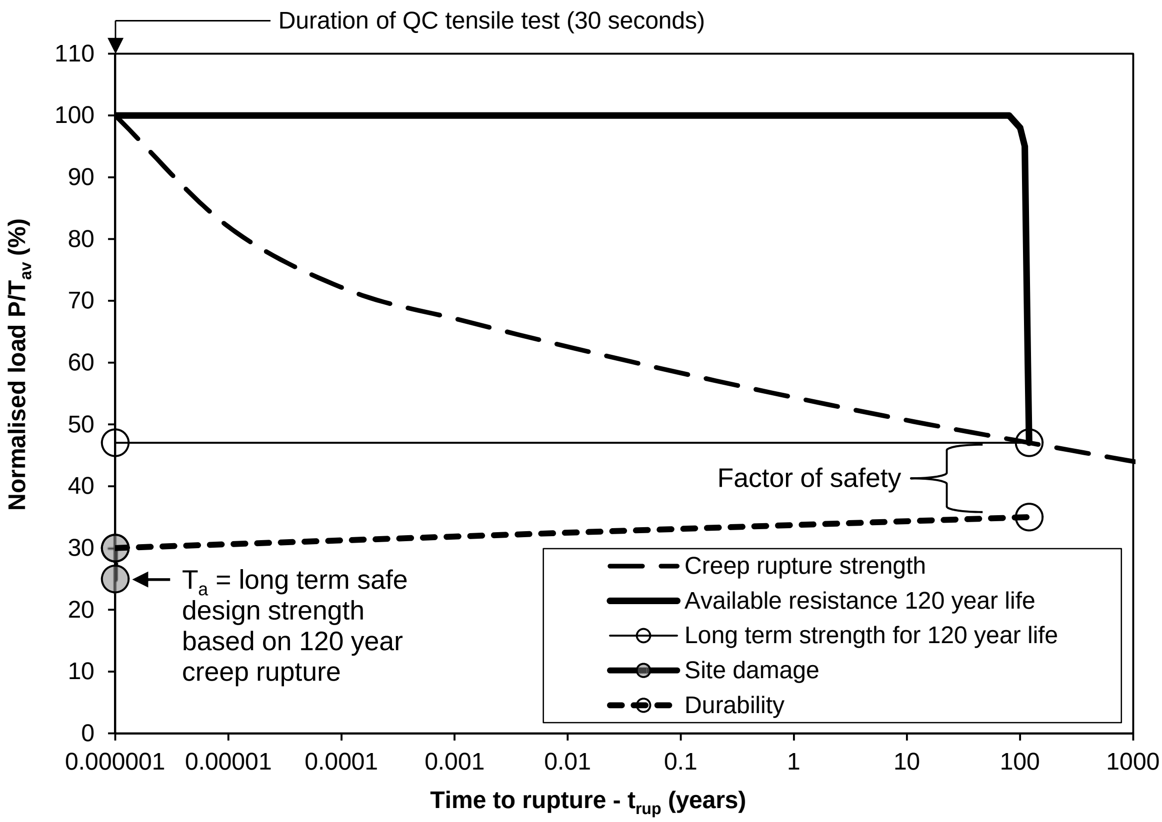

Figure 6. Degradation of polymer reinforcement retained strength with time

The behaviour outlined above can be seen in the data shown on Figure 7, based on testing carried out in the laboratories of Tensar International (TI). TI has a number of creep laboratories, at temperatures from 10°C to 50°C. Despite having a large number of testing locations, it is still necessary to terminate some tests before rupture occurs to create space for new tests. Over the last few years, TI has had a policy of carrying out a tensile test on any specimen taken down from the creep laboratory which has not ruptured. This tensile test result is then compared to the tensile strength measured before the creep test started. Comparison of the two values gives the "retained tensile strength". Figure 7 shows the retained tensile strength from 86 creep tests, plotted versus the maximum strain reached in the creep test (ie. at the point in time when it was terminated). The strain reached in the creep test is the best way of indicating how close the test was to rupture. There are different forms of geogrid included in Figure 7, with strain at long term rupture being anywhere from 15% to 35%. The longest duration test on Figure 7 was 49,000 hours, or 5.5 years. The majority of data plotted on Figure 7 show that, even at very high strains, the specimens retain around 100% of their initial strength. There are four tests with around 50 to 60% retained strength, but these were all very close to rupture when the tests were terminated (ie. well down the last part of the "available resistance" line shown on Figure 6).

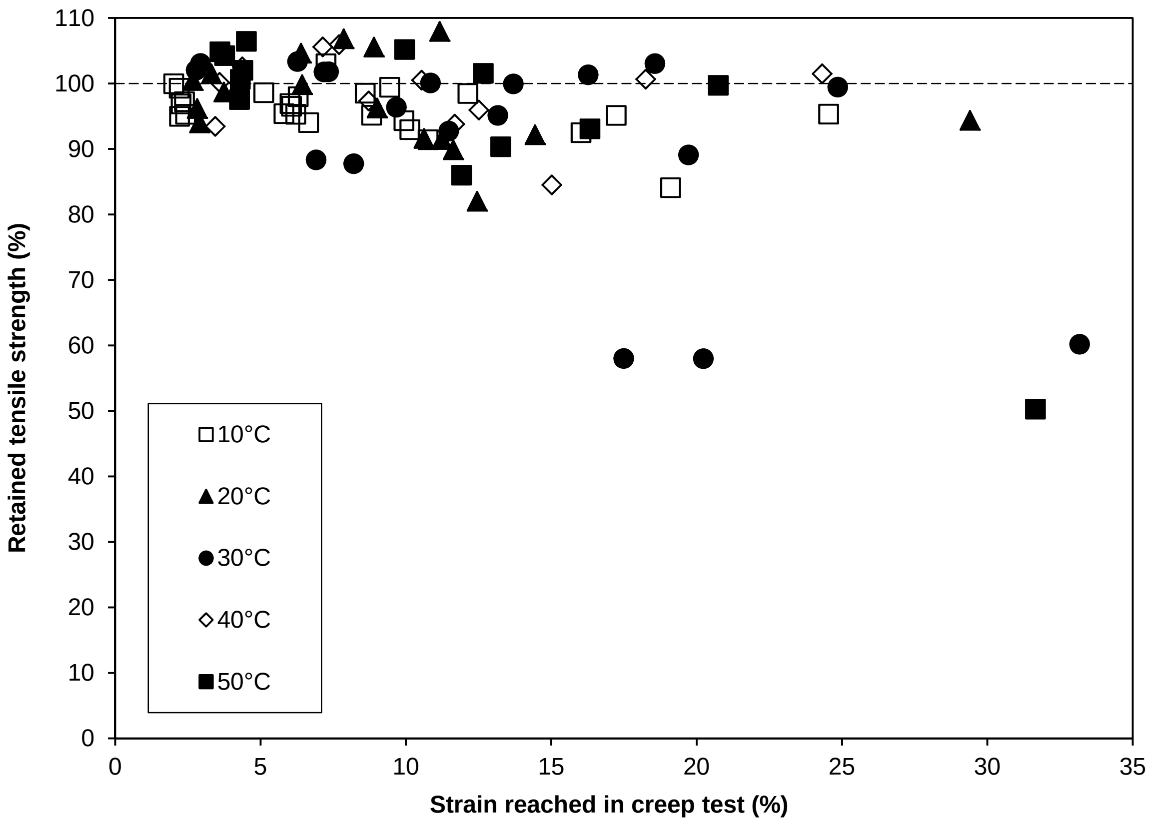

Figure 7. Retained tensile strength in creep test specimens in relation to the maximum creep test strain

Figure 6 also shows the design strength (T_a) at 25% on the y-axis. The short line moving upwards from this point indicates the immediate loss of strength due to installation damage (Mode 1 degradation), followed by the inclined line moving to the right indicating degradation due to durability strength loss (Mode 2 degradation). By the time this line reaches 120 years, there is still a vertical gap between the point at the end of the durability degradation line, and the creep rupture point. This gap represents the factor of safety (FS). In many design methods, there may be an additional safety margin due to either load factors used in calculating the load applied to the reinforcement, or material factors applied to the soil strength.

Returning to the question: "if 100% of the tensile strength is available until the end of the design life, then why not use this for design"; this can be explored further using the ideas shown on Figure 6. If the value of RF_{CR} is set to 1.0, then the calculated design strength becomes T_a = 55% (with all other factors remaining the same as given previously):

$$T_a = \frac{T_{ult}}{RF_{ID} \times RF_D \times FS} = \frac{100}{1.1 \times 1.1 \times 1.5} = 55$$

Figure 8 shows the same relationship as Figure 6, but with the design strength set at 55%, and assuming that the duration of service is now too short for significant degradation due to durability, and that the actual installation strength loss is negligible. It can be seen that 55% meets the creep rupture strength line at about 1 year. This indicates that it may well be possible to build the structure based on a design with reinforcement strength taken as the short term tensile strength, but time to failure has become very short, and there is no spare capacity to allow for installation damage or durability degradation. Therefore any attempt to reduce conservatism by basing designs on the use of the short term tensile strength is not an acceptable approach.

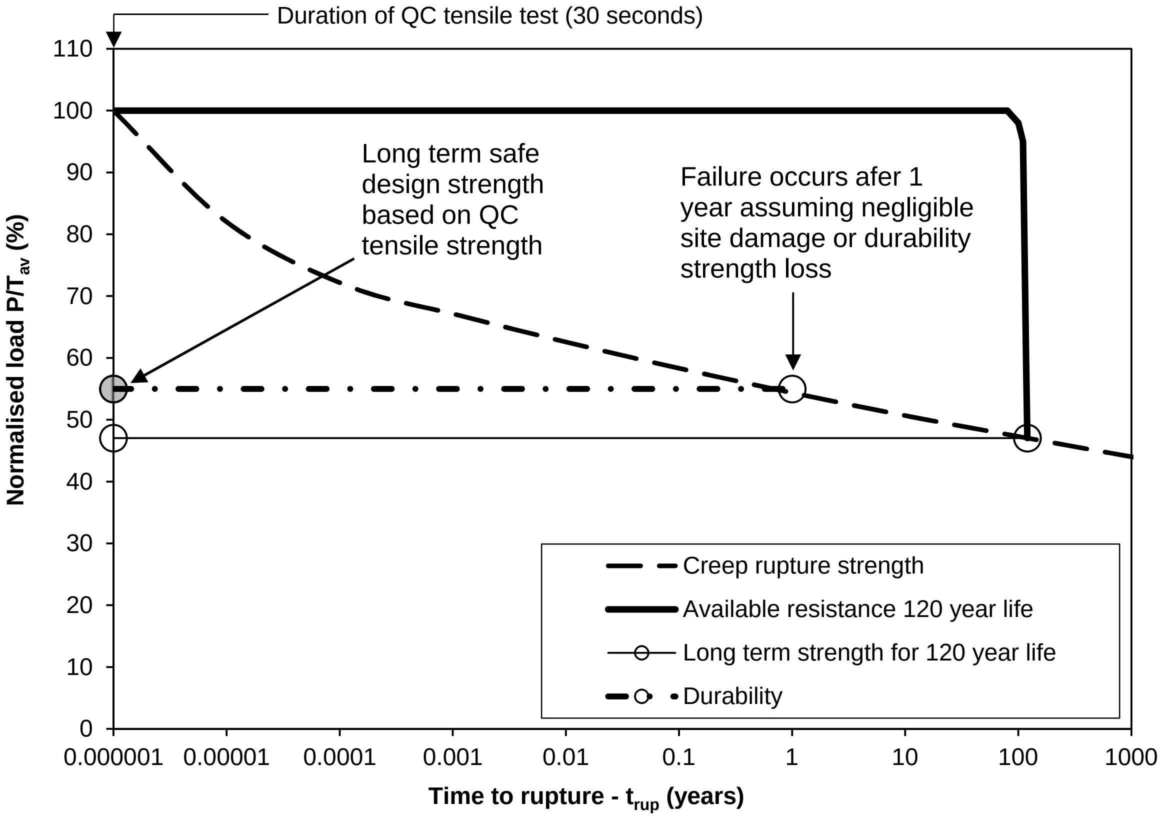

Figure 8. Degradation of polymer reinforcement retained strength with time omitting creep

4.2 Interaction parameters

Interaction parameters are required for design in two situations: mechanisms where sliding over reinforcement may occur, and mechanisms where pull-out of reinforcement may occur. Both are illustrated on Figure 9. Interaction parameters are defined as coefficients applied to the soil shear strength to take into account shearing between reinforcement and soil, rather than between soil and soil. Factors are therefore equal to 1.0 or less. Typically, measured values are close to 1.0 for very coarse soils, reducing to lower values for finer soils.

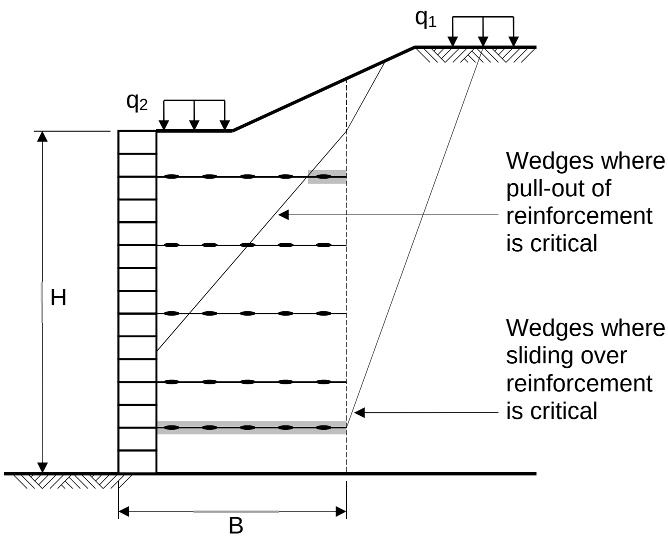

Figure 9. Interaction parameters for sliding over reinforcement and pull-out of reinforcement

With regards to affecting the cost (and therefore conservatism) of a design, the pull-out interaction factor is not important. This can be seen on Figure 9, where the length of upper layer of reinforcement, which may be critical in pull-out, is relatively short. Layers of reinforcement deeper in the structure will be critical in tensile failure. This means that using a pull-out interaction coefficient (reduction factor on shear strength) of 1.0 and 0.5 will give much the same design layout (especially when using the two-part wedge method, see Section 3).

The situation as regards the sliding interaction factor is quite different. Figure 9 shows how the sliding check along reinforcement involves its full length. This means that if sliding over reinforcement is critical (which is frequently the case), then changing the sliding interaction coefficient from 1.0 to 0.5 will double the length of the reinforcement, and therefore double the cost of the reinforcement. Fortunately measuring the sliding interaction factor is relatively easy, normally done by using a large shearbox (300mm × 300mm), adapted to incorporate the reinforcement on the sliding plane. However, because internal stability design is done in terms of effective stress, interaction testing must also be carried out in terms of effective stress, so that shearbox testing procedures may be relatively quick for coarse soils, following the T 143-93 method as per Table 2. But for finer soils, the T 140-93 method is required, which can mean that the duration of testing using a large shear box will be very long, to satisfy the requirements for full saturation, consolidation and slow shear. It is the author's observation that such testing is very rarely carried out for fine soils, so that assumed values of the sliding interaction coefficient may well be on the low side.

5 DESIGN FACTORS

By definition, limiting equilibrium design methods predict the point of collapse of a structure. This is the case for both the traditional lumped safety factor methods, and limit state methods. In order to ensure that failure does not take place, a margin against failure is built into the calculation. The obvious margins against failure are either the lumped safety factors for a particular calculation, or more recently, partial factors which may be applied either to loads, to material properties, or to the resulting resistances calculated. Some of these factors are listed in Table 4, and some examples of typical values are given for the cases of external sliding, bearing capacity and reinforcement strength. Two of the best known limit state codes are AASHTO (LRFD) and Eurocode 7 (EC7, which has three Design Approaches, DA's, although not yet including reinforced soil design, ie. internal stability). The list of values in Table 4 is far from complete, but gives some examples of typical values which generally indicate that the newer limit state methods are intended to reduce conservatism.

Factor type	Lumped factor of safety	Load factor methods	Material factor methods
Safety margin against external sliding	Typically 1.5	EC7/DA2 applies 1.35 (DL) & 1.5 (LL) to driving and 1.0 to resisting forces	EC7/DA3 applies 1.25 to soil strength combined with 1.3 to driving live loads
Safety margin against bearing capacity failure	Typically 2.0 to 2.5	AASHTO applies 1.35 to the soil mass, 1.75 to LL and a reduction factor of 0.65 to the bearing capacity	EC7/DA3 applies 1.25 to soil strength combined with 1.3 to driving live loads
Safety margin against reinforcement failure	Typically 1.3 to 1.75	Mostly use 1.0 (BS 8006-1:2010, AS4678 and AASHTO)	EBGEO uses 1.4
Wall friction on back of reinforced soil mass	Varies from $\delta = 0$ to $\delta = 1.0 \times \phi'$ BS 8006-1:2010 and FHWA/AASHTO both define $\delta = 0$ for level backfill NCMA defines $\delta = \phi'$, but sets the vertical component of force to 0		
Bearing capacity inclination	Included in non-US practice Ignored in US practice		
Soil strength definition (ϕ'_{peak} or ϕ'_{cv})	The majority of published design guidelines define the soil strength for design as ϕ'_{peak}, but some use ϕ'_{cv} In some methods (HA68/94 for slopes) ϕ'_{cv} is used to give the full margin against failure		

Table 4. Outline of typical factors which provide the margin against failure in static reinforced soil design

The last three items in Table 4 are additional "factors" which also affect the margin against failure in a design, and must be defined to create a complete design method. The first is the angle of wall friction (δ) on the back of the reinforced soil block, which affects the values of earth pressure calculated. This can vary from $\delta = 0$ (perfectly smooth interface) to $\delta = \phi'$ (perfectly rough interface). Bearing in mind the mechanism which will take place as the reinforced soil block slides forwards, sliding on the interface between the backfill and back of the reinforced soil block will take place, and because it is soil-to-soil sliding, it would seem that using $\delta = \phi'$ is justified. The effect is significant, and switching from $\delta = 0$ to $\delta = \phi'$ in a given design could easily result in a 10% reduction in reinforcement quantities.

The second item is the use of the inclination factor in the bearing capacity calculation. It is generally used in non-US practice and specifically ignored in US practice (AASHTO). This only becomes an issue in the case of low bearing capacity, where reinforcement length (B in Figure 1) may become large to create an adequate margin against bearing failure. Normal reinforced soil calculations indicate that a substantial outward lateral force exists at the foundation level, which would indicate that it should be included. The effect is significant, and for a typical reinforced soil structure, bearing capacity can easily be halved by including the inclination factor.

The final item is the definition of soil strength to be used in calculations, with a choice between ϕ'_{peak} (the peak value) and ϕ'_{cv} (the constant volume value). Figure 10 shows a typical shear box test on a sand fill, where ϕ'_{peak} = 38.9° and ϕ'_{cv} = 30.4°. The benefit of using ϕ'_{cv} is that it is reliable, and will always be present in the soil mass, even if density or compaction is below standard. With reference to Figure 10, the value of ϕ'_{cv} = 30.4° could be used directly as a design value. However if ϕ'_{cv} is combined with other load and material factors, then it is probably becoming over-conservative. Most published design standards for reinforced soil retaining walls use the ϕ'_{peak} definition. However, with reference to Figure 10, judgement is required when choosing a value of ϕ'_{peak} for design, and the measured value of 38.9° would be too high, because it only exists over a very small range of movement on the shear surface, and as mentioned already, it depends on density and the degree of compaction achieved during construction.

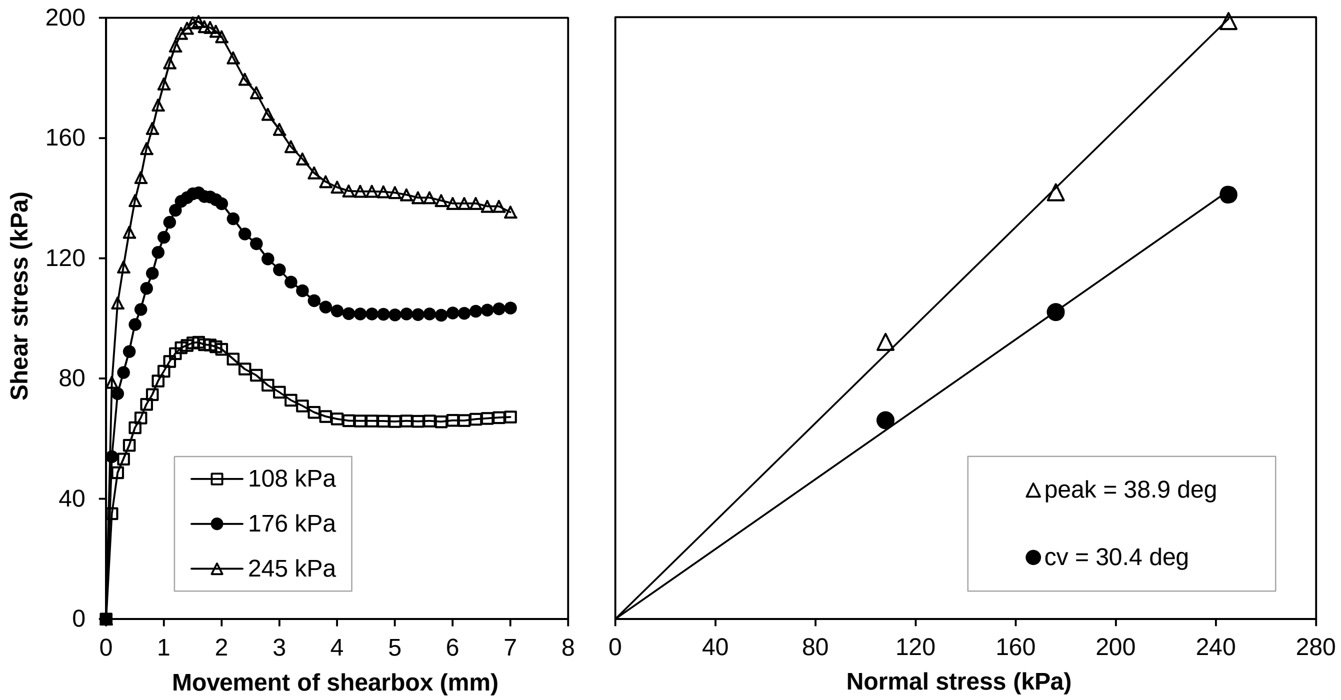

Figure 10. Typical shear box test on sand fill

6 CONCLUSIONS

When reviewing design methods for reinforced soil structures, from the point of view of conservatism and reducing cost, it is convenient to consider three elements of the design method: method of calculation, material parameters and factors used. Based on the discussion given above, the following conclusions are made in relation to design by limiting equilibrium methods:

1 Conservatism can be reduced by using a method of calculation for internal stability which models likely modes of failure as closely as possible, and requires as few assumptions as possible to make the calculation. A two-part wedge method is described which relies on searching a large number of potential failure mechanisms, but takes full advantage of the contribution of the reinforcement and facing in the stability analysis.

2 It is important that soil parameters to be used for design are measured on representative soil samples, reconstituted to the conditions expected during construction. Soil strength test methods used must provide the effective stress parameters c′ and ϕ' for all soil types used as fill materials. Consideration may be given to using peak shear strength, adequately assessed from measured data, provided that adequate margin against failure is provided elsewhere in the design method.

3 Reinforcement strength should be based on measuring the long term creep-rupture behaviour as appropriate for the expected design life of the structure. Suitable test methods, as well as methods of interpretation, are given in Chinese Standard QB/T 2854-2007 on "Creep testing and evaluating method on plastic geogrids". Design should not be based on short term tensile strength, because there will be no control over the margin against internal failure in design, with the likelihood of creating structures which are unsafe within a short time after completion.

4 Conservatism may be reduced by making accurate measurements of the sliding interaction factor, using appropriate drained soil test methods.

5 There is considerable scope for reducing conservatism by investigating the factors of safety used in design. Recently published methods, such as EBGEO (based on EC7) and the AASHTO (LRFD) approach applied to the two-part wedge method of calculation, already provide designs which are significantly less conservative than older methods.

6 Consideration may be given to using a wall friction angle of $\delta = \phi'$ when calculating earth pressures on the back of the reinforced soil block. This will provide a significant reduction in conservatism.

7 If any design method is created based on the ideas presented here, which results is a significant reduction in the quantity of reinforcement required, then it is important that the method includes a serviceability check. The two-part wedge method of calculation may be used to assess post-construction strain in the reinforcement as defined by BS 8006-1:2010.

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