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Towards a limit state design specification for reinforced soil walls

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ABSTRACT: To achieve economy, reinforced soil wall projects undertaken by the Roads and Traffic Authority of New South Wales have been based on Design and Construct Contracts. This changing environment has prompted the RTA to develop a Limit State Design Specification which provides a rational and equitable framework for specialist contractors to produce designs with consistent safety margins for a range of conditions. Due to the complexities of this exercise, this design specification only covers reinforced soil walls with vertical or near-vertical facing. Calibration studies have been conducted to assess parameters that have significant impact on the cost of constructing reinforced soil walls. Reliability analyses were conducted to ensure that the partial factor on soil shear strength can lead to a consistent reliability. The potential problems of using a statistical definition of characteristic values are addressed.

1 INTRODUCTION

Over the last few years, various proprietary reinforced soil wall systems were built in New South Wales, Australia. These include conventional metallic strip and bar mat reinforcements as well as geosynthetic strip and geogrid reinforcements. Precast concrete panel and modular block facings have been used.

To achieve innovative and cost effective designs, construction of reinforced soil walls let out by the Roads and Traffic Authority of New South Wales (hereafter abbreviated as RTA) is often based on Design and Construct Contracts. This requires the contractor to select one of the pre-qualified reinforced soil wall systems, and to design and construct the reinforced soil walls to meet all the overall prescribed requirements.

In many cases, reinforced soil walls are constructed to support bridges which are designed to limit state structural codes. To achieve a harmonised framework between the design philosophy of bridges and reinforced soil structures, a limit state design specification of reinforced soil structures is desirable. Furthermore, the limit state design approach is most suitable for dealing with a range of systems and for providing an analysis framework covering a number of potential failure mechanisms.

The RTA limit state reinforced soil wall design specification is still in a draft state, and will be referred to as the draft RTA Specification in this paper. This paper presents the framework for formulating the draft RTA Specification, which adopts a wholistic approach to the design process. Critical and debatable issues related to the implementation of limit state format in the design of reinforced soil structures are discussed in the paper.

2 LIMIT STATE DESIGN

2.1 Limit state principle

Limit state design of reinforced soil structures requires the consideration of two main classes of limit states, namely ultimate limit state and serviceability limit state.

An ultimate limit state is a state at which failure mechanism can form in the ground or in the reinforced soil structure, or severe structural damage (e.g. yielding or rupture) occurs in principal structural elements. A serviceability limit state is a state at which movements of the reinforced soil structure affect the appearance or efficient use of the structure or nearby structure or services which rely upon it.

For each limit state under consideration, the design resistance (R*) shall be greater than or equal to the design action (S*), that is

$$R^* \geq S^*$$

Traditionally, design actions are factored up values relative to the working values whereas the design resistances are factored down values. This factoring is done using partial factors.

2.2 Integrated design procedure for reinforced soil structures

Limit state design loads and design strengths are the most severe values that can credibly occur in the limit state under consideration. The four main components making up these are: (1) soil parameters, (2) load factors, (3) calculation model and (4) material factors.

Selection of soil parameters is critical in design of reinforced soil structures. Measurements of soil frictional angles at constant volume condition and at peak resistance will lead to largely different results in medium dense (or denser) soils.

The internal design of reinforced soil walls is often based on the classical assumption of earth pressures varying between at rest and active conditions, Mitchell and Villet (1987). Since this assumption does not explicitly take into consideration all the factors that influence the horizontal earth pressures in a reinforced system, a load factor significantly greater than unity has to be applied to give the design limit state loads.

On the other hand, if a calculation model which explicitly takes into consideration all the factors that influence the horizontal earth pressures and automatically gives the most severe values that can credibly occur, the corresponding load factor could be just unity.

An integrated set of soil parameter selection, load factors, calculation model and material factors is adopted in the RTA Specification.

3 SOIL RESISTANCE

3.1 Soil parameters

As relatively large soil shear strains would occur at ultimate limit state (especially in the case where instability is governed by pull out failure), mobilised angle of friction is likely to approach the constant volume value. On the other hand, the use of constant volume angle of friction may result in some conservatism for in-situ soils because soil fabric strength would be destroyed in the measurement. As such, the use of constant volume angle of friction for fill materials and peak angle of friction for in-situ soils is being considered.

The design value of angle of friction is the factored characteristic value as follow:

$$\phi_{design} = \mathrm{atan}\left(\tan\left(\phi_{characteristic}\right) \cdot \Phi_m\right)$$

The characteristic value is taken as the prudently selected value that represents the average properties of the mass of the soil affecting the occurrence of the particular limit state under consideration. In the presence of sufficient, representative and reliable test results, characteristic may be derived from statistical methods such that the calculated probability of a worse value governing the occurrence of a limit state is less than 5%. As such, it should be recognised that the mean of test results from samples is not the characteristic value.

The use of definition based on statistical methods presents problems as adequate data is seldom available for a rigorous statistical calculation. Although one may argue that Baysian statistics may be used to make up for the lack of data, the calculated results are very dependent on the assumed prior knowledge of comparable experience with ground properties.

There is also the important issue of spatial averaging. Unlike structural member, the performance of a geotechnical system, or an important part of it, may be affected by a relatively large volume of soil. Therefore, the variability of the performance is not governed by the variability of individual soil element, but by the variability of the spatially averaged property, where spatial averaging is taken over a domain that governs the particular performance criterion. For reinforced soil walls where the occurrence of a limit state involves a large volume of soil, the variability of the spatially averaged property is in general significantly less than that of individual soil elements.

As the assumption of simultaneous mobilisation of constant volume angle of friction through out the soil mass is often conservative, the draft RTA Specification permits the use of a material factor Φ_m equal to unity. For peak angle of friction, the assumption of simultaneous mobilisation of these values is somewhat optimistic. Limited reliability analyses presented in Section 8 below suggest that a Φ_m equal to 0.8 may be appropriate.

3.2 Pull-out resistance

At ultimate limit state, there would be significant relative movements occurring between the soil reinforcement and the reinforced soil. Pull-out resistance at ultimate limit state could therefore be conservatively taken as the residual value. However, it is recognised that the practicability of measuring residual pull-out resistances often depends on the design of the pull-out box. Options of using residual resistances measured in direct shear boxes and large deformation resistances measured in pull out boxes are being considered.

4 LOAD FACTORS ON SOIL WEIGHT AND EARTH PRESSURE

4.1 Serviceability limit state

Maximum reinforcement tensions are required to be less than some critical values in the draft RTA Specification to ensure that excessive post-construction movements and yielding of reinforcement would not occur under serviceability conditions.

An examination of the measured reinforcement tensions of twelve reinforced soil walls presented by Christopher et al (1990) indicates that the measured tensions were often higher than that given by the calculation model. As shown in Figure 1, a load factor of about 1.25 has to be applied to give a reasonable maximum tension envelope for metallic reinforcements.

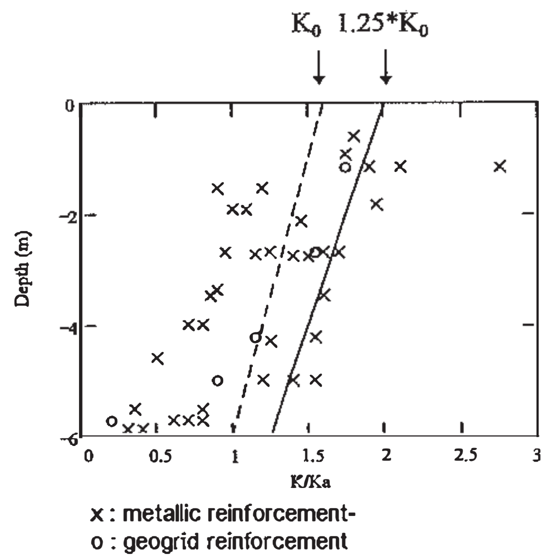

Figure 1: Field measured coefficients of earth pressure, Christopher et al (1990). Note values measured during constuction excluded.

A dead load factor much greater than unity is inconsistent with the variability of soil dead weight, which is relatively small. An updated study by Enrich and Mitchell (1994) have shown that these higher tensions could be explained by locked-in of tensions due to compaction stresses.

This seems to justify the use of a load factor of unity on soil dead weight and a load factor of 1.25 on the coefficient of earth pressure calculated from classical calculation model, Mitchell and Villet (1987). A load factor on the coefficient of earth pressure with a value less than 1.25 may be considered if a higher chance of exceeding serviceability limit state criteria can be justified.

Full scale field data for walls reinforced with geosynthetic reinforcements were not sufficient to derive a meaningful maximum tension envelope.

4.2 Ultimate limit state

The relevance of locked-in tensions due to compaction stress at ultimate limit state depends on whether the reinforced soil structure is ductile enough to relief the locked-in tensions. At the present moment, available information is not conclusive.

Despite excessive post constuction lateral deformations had occurred in a number of walls with mild steel strip reinforcements (Blight and Dane 1989), reinforcement tensions (on average) were more or less similar to the design maximum tension envelope. Blight and Dane (1989) have also shown that the breaking strain of steel reinforcements could be greatly reduced as a result of localised pitting corrosions.

Breaking strains of a number of geosynthetic reinforcements are less than 10% under long term sustained loads. Installation damage would further reduce the breaking strain. The limited breaking strain of geosynthetic reinforcement may pose a constraint in relieving the locked-in tensions (if any) completely at ultimate limit state.

The relevance of locked-in tensions at ultimate limit state is not yet fully resolved. In the absence of sufficient full scale field measurements at ultimate limit state, the draft RTA Specification takes the prudently approach of making allowance of locked-in tensions.

5 LIVE LOAD AT SERVICEABILITY LIMIT STATE

One important feature of many soil structure interaction problems is the difference between live load and live load effect. While live load (like traffic load) may be transient in nature, some of its induced effects may be permanent because of the non-elastic nature of the soil-structure.

Bastick et al (1993) has shown significant tensions were locked-in upon unloaded from a surcharge of 200 kPa applied on a full scale reinforced soil abutment. The RTA Specification requires allowance of live load effectas in the serviceability limit state checks.

6 MATERIAL FACTORS FOR GEOSYNTHETIC REINFORCEMENT

In the draft RTA Specification, material factors are multiplied to the intact short term tensile strength of geosynthetic reinforcement for deriving long term design strength. These material factors are to account the effects of long term creep deformation, stress rupture, chemical degradation, biological degradation, installation damage and connection efficiency. Some of these material factors are discussed in details in the following paragraphs.

6.1 Stress rupture

Although geosynthetic reinforcement will not corrode, its load carrying capacity reduces with load duration. This is referred to as stress rupture. The reinforcement tensions at ultimate limit state shall not be larger than the stress rupture strength under a load duration equal to the design life.

The mean stress rupture lines of four geosynthetic reinforcements are presented in Figure 2. Note the load duration in Figure 2 is plotted in a log scale. It is important to note that both polyester and HDPE reinforcements have significant strength reduction for long load duration.

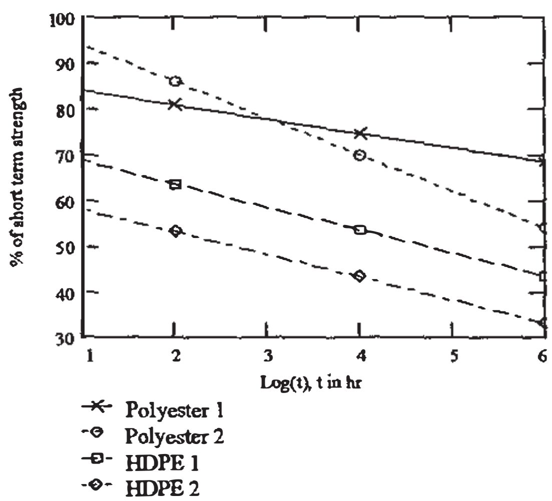

Figure 2: Mean stress rupture curves at 20°C of some geosynthetic reinforcements

The approached adopted is to determine the material factor to account the long term stress rupture strength of the geosynthetic from stress rupture data with at least one year load duration. However, it is still necessary to extrapolate test results to a design life of 100 years and there are uncertainties involved in such an exercise.

Extrapolation uncertainty requires further reduction in the long term rupture strength as suggested by BSI 8006 (1995). However, the stress rupture effect may cease when the applied load is less than a critical value also needs to be recognised. Further works are required to establish a more realistic and yet safe extrapolation technique to achieve economy.

6.2 Post construction creep

A material factor for post construction creep is applied to the geosynthetic reinforcement short term tensile strength to limit the maximum post construction creep to less than 1% and 0.5% for reinforced soil walls and reinforced soil abutments under serviceability load respectively.

This material factor is derived by extrapolating strains measured in a number of sustained load tests carried out near working stress level. An extrapolation factor is required as in the case of deriving the stress rupture strength.

6.3 Hydrolysis of polyester

To control hydrolysis of polyester to an acceptable value, pH value of the environment should be less than 10. Hydrolysis of polyester can be expressed by classical chemical reaction equations, McMahon (1959):

$$\frac{d}{dt}x = C\cdot e^{-\left(\frac{E_a}{RT}\right)}\cdot(A-x) \quad (1) \qquad x = \frac{M_0}{M_t} - 1 \quad (2)$$

where C, E_a and A are material constants; R is the gas constraint; T is temperature (may be a function of time); M_0 and M_t are the polyester initial and residual molecular weight at time t.

By integrating equation (1), the polyester residual molecular weight at any elapsed time and over any temperature variation can be calculated from equation (2). Residual short term strength can in turn be found by establishing a correlation with the polyester residual molecular weight.

The percentage strength loss against ambient temperature of two polyesters is presented in Figure 3. For an ambient temperature of less than 20°C, the strength loss due to hydrolysis is very small; however, for higher temperature, the strength loss is not insignificant and needs to be factored in the design process. Soil temperature measurements in a reinforced soil wall have been carried out by RTA in Sydney, located on the coast of New South Wales (Australia). The test data indicates that the maximum soil temperature at about 300 mm from facing was 35°C decreasing to 26°C at 1 metre from the facing. In hotter regions of New South Wales, other data indicate that soil temperatures could be about 30°C even at 1.2 m ·om the facing.

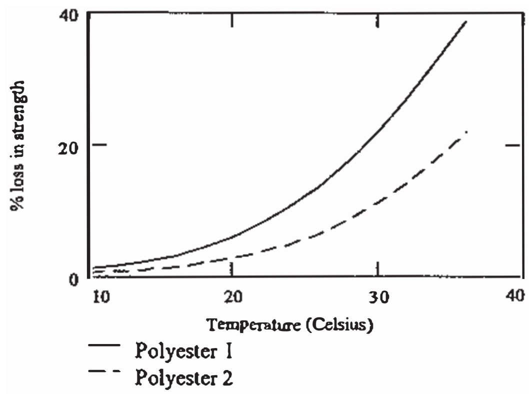

Figure 3: Loss of tensile strength due to hydrolysis for 100 years

Variation of temperature over time can be taken into account by expressing temperature as an appropriate function of time (e.g. sinusoidal variation) and used in conjunction with equations (1) and (2) above.

For Sydney conditions, an equivalent constant temperature of 32°C was found to be appropriate for determining the material factor for strength loss due to hydrolysis near the concrete facing. Scatterness of test data and uncertainty in the extrapolation would warrant the need for an additional uncertainty factor.

6.4 Connection strength

For modular block walls reinforced with geosynthetic reinforcements, connection test results often revealed that short term connection strengths were less than the short term tensile strengths.

In many cases, connection failure was initiated by physical rupture of the geosynthetic reinforcement rather than governed by pull out failure. This premature rupture was due to the longitudinal members being unevenly loaded because of the wedge-shaped space between some modular blocks. A material factor is multiplied to the long term stress rupture strength to take account of the uneven stress distribution.

For HDPE geogrid reinforcements, connection failure was often initiated by rupture of the traverse members which were bearing on the modular block alignment pins to resist the applied tension. As the long term rupture strength of the traverse members of mono-orientated HDPE geogrid reinforcement is not yet well established, its contribution to the long term connection strength is neglected in the draft RTA Specification.

7 OVERALL SLIP ANALYSIS

Overall rotational stability is often analysed using limit equilibrium methods. Experience has shown that the most critical potential slip surface is not necessarily circular in some cases. An example of this is shown in Figure 4. The draft RTA Specification requires searching of both circular and non-circular potential slip surfaces.

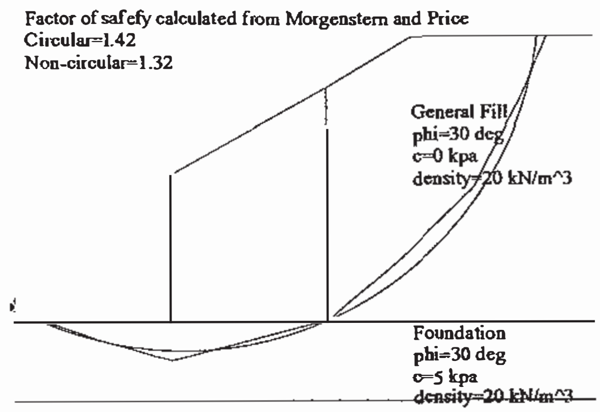

Figure 4: Comparison of factors of safety for most critical circular and non-circular slip surfaces

8 RELIABILITY ANALYSIS

Reliability analyses were conducted to investigate whether the limit state approach could produce more consistent designs as compared to the conventional working stress approach. Bearing failure and sliding failure were considered in the analyses.

Soil parameters adopted are shown in Table 1 below. Characteristic values of the soil parameters were calculated from the Student-t density function. The material factor Φ_m on peak angle of friction was taken as 0.8 and the load factor on soil density was taken as unity.

For each failure mechanism, peak angles of friction of foundation varying from 20° to 45° were considered. The base lengths of the walls were sized such that the walls were marginally stable under ultimate limit state. As shown in Figure 5, the required base lengths (and hence the overall cost) increased as the foundation frictional angle decreased.

The reliability index of each wall was then obtained by direct integration over the probability occurrence functions (Student-t distribution) of the three variables: foundation frictional angle, foundation density and fill frictional angle. Variability of the fill density was assumed to be negligible as compared to the other quantities. Modelling uncertainties have not been allowed in the analyses.

Table 1: Parameters for reliability analyses (Wall height = 6 m)

Property	Mean of samples	cov of samples	No. of test	Chara. value	Design value
ϕ, fill	30 deg	10%	7	28 deg	23 deg
γ, fill	20 kN/m^3	n.a.	n.a.	20 kN/m^3	20 kN/m^3
ϕ, foundation soil	20,25,30,35,40, 45 deg	10%	7	19,23, 28,32, 37,42 deg	15,19, 23,27, 31,35 deg
γ, foundation soil	18 kN/m^3	5%	7	17.7 kN/m^3	17.7 kN/m^3

As the assumption of simultaneous mobilisation of peak angle of friction through out the whole soil mass would be somewhat optimistic, a mobilisation factor (mb) multiplied to the soil frictional angle was incorporated in the reliability analyses. Values of mb equal to 1.0 and 0.9 were considered. The calculated reliability index for bearing failure and the overall factor of safety calculated from working stress approach based on the characteristic soil parameters are shown in Figure 5 below.

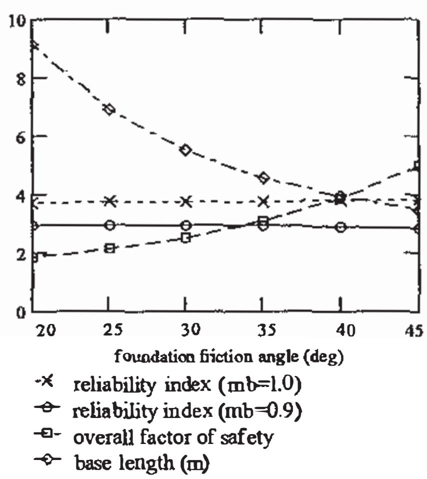

Figure 5: Reliability index for bearing failure

The first thing observed in Figure 5 is that a Φ_m of 0.8 on peak angle of friction was fairly consistent with an overall factor of safety between 2 to 3 normally adopted in working stress analyses for the range of foundation soil frictional angles varying between 25° to 35°. Secondary, the calculated reliability index was almost independent of foundation soil frictional angle for a particular value of mobilisation factor (mb). This suggests that the limit state approach can produce consistent safety margins over a range of foundation conditions. Thirdly, the wide range of overall factors of safety (between 2 to 5) highlights the deficiency of the working stress approach, which will not produce designs with consistent safety margins by using a unique value of overall safety factor. However, it should be noted that the calculated reliability index was largely influenced by the mobilisation factor and the assumed density function. The absolute value of the calculated reliability index may not be truly correct and further work is required to quantify this.

9 CONCLUSION

A RTA limit state design specification is currently under development. Critical issues including determination of long term design strength of geosynthetic reinforcements, load factors, material factors and soil parameter selection were identified. A draft document was issued to major suppliers of reinforced soil wall systems in New South Wales and the feedback from these organisations is currently being assessed in preparing the final specification.

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REFERENCES

Bastick, M., Schlosser F., Segrestin, P., Amar, S., Canepa, Y. (1993). Experimental reinforced earth structure of bourron marlotte. Soil reinf.:full scale experiments of the 80's, Paris, France, p. 201-228.

BSI 8006 (1995). Strengthened/reinforced soils and other fills.

Christoper, B.R., Safdar A.G., Giroud J.P., Juran, I., Mitchell, J.K., Schlosser, F. & Dunnicliff, J. (1990). Reinforced soil structures, Federal Hwy. Administration Rep No. FHWA/RD/89/043.

Ehrlich, M., Mitchell, J.K. (1994). Working stress design for reinforced soil walls. J. of Geot. Engg, ASCE, Vol. 120 No. 4, p.625-645.

McMahon W., Birdsall H.A., Johnson G.R. and Camilli C.T. (1959). Degradation studies of polyethylene terephthalate, physical properties evaluation of compounds and materials, 1/4 57-59.

Mitchell, J.K., Villet, W.C.B. (1987). Reinf. of earth slopes & embankments, NCHRP Report 290, National Research Council, Washington, DC.