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Discussions: Foundations

• DISCUSSION BETWEEN BOLTON AND LESHCHINSKY

M. Bolton
(*Cambridge University, U.K.*)
D. Leshchinsky
(*University of Dalaware, U.S.A.*)

Dr. Bolton expressed doubt about the capacity of the longitudinal strain gauge shown by Dr. Leshchinsky to measure tension accurately in the geotextile. The rubber-gauge-rubber casting might either measure the strain in the overlying soil, or in the underlying textile, or an average of the two. In any event, the measurement of strain was not easy to relate to tension, due to creep.

Dr. Leshchinsky replied that the gauge was intended for situations in which there was no slip between the textile and the soil, so that the strains in each would be similar. The casting of a stiff lateral band of epoxy, as shown in the paper by Springman et al. to this conference, clearly provided a good load cell for their plane strain model, but he pointed out that further thought must be given to the required width of the stiffening band in a field application which would be very wide.

Dr. Bolton then responded to the comments which Dr. Leshchinsky had made about scale models, particle size effects, and centrifuge models. Dr. Leshchinsky had raised doubts about the zone of influence of full-scale geogrids in relation to particle size. Large displacements were evident in the soil above and below the specimen in laboratory pull-out tests, both near to the grid and at separations much more that the five to ten particle diameters which were frequently assumed. These measured displacements comprised both rigid body movements and deformations. The deformation zone had been observed at normal stress levels ranging from 25 kPa to about 100 kPa. This had led Dr.Leshchinsky to question the use of field-scale soil particles in model-scale experiments, which might lead to undue arching effects and strengthening.

Dr. Bolton doubted whether a pull-out test was relevant to this issue, since the soil riding above the sheet would always tend to translate as a rigid block except where it would form a passive wedge against the front face of the box. The five diameter transition zone referred to the thickness of a shear band. He recalled that the ratio of particle size to grid aperture was mentioned as a parameter to considered in the paper by Springman et al. to the conference. He also felt that it was important to make scaling observations at field-scale stress levels, since it was known that angles of shearing resistance and dilatancy could reduce by the order of 5 to 10 degrees for every factor ten in mean effective stress. Far too much dilatancy would be evident in reduced scale models unless they were centrifuged, and aberrant arching mechanisms might be observed.

Dr. Leshchinsky reiterated that he had advocated centrifuge testing in his paper. He simply wished to warn against the uncritical use of soil particles in models which would scale as boulders in the field. He was pleased to see that the issue of particle size was being addressed.

• COMMENT

J.T.H. Wu
(*University of Colorado, Denver, U.S.A.*)

Firstly, I wish to congratulate Professor Dov Leshchinsky for delivering an excellent keynote lecture in which he addressed some important issues on GRS structures. And, I would like to voice my concurrence with Dov in his statement that finite element analysis and limit equilibrium analysis should be used to augment each other and that we need to validate analytical models with full-scale tests.

However, I want to point out that, with the

exception of being relatively more complicated in formulation, finite element analysis CAN be made to share all the advantages which Dov cited in his lecture. Design charts can certainly be developed from results of finite element analyses. These design charts can also be used to double-checked finite element analysis results for design purposes. Input for performing a finite element analysis can be simplified to a level comparable to that of a limit equilibrium analysis. For example, the program GREWS for design of GRS walls and slopes, which I presented last week in the Seiken Symposium (Wu, et al, 1993), allows the user to simply specify wall height, soil classification symbols (for backfill, foundation and retained soil), and the degree of compaction to obtain a rational design of a GRS wall using the finite element method of analysis. A smarter user, of course, may specify more project-specific information to make the design more discriminating. As for the output, it is a simple matter to suppress information which the user considers redundant. One may choose to print out, for instance, the minimum local safety factor and the maximum lateral wall deformation for a design obtained from finite element analysis.

On the issue of "black box," I would argue that any computerized design tools, including those based on limit equilibrium analysis, are somewhat of a black box to most users. In that case, why don't we provide the designer with the best black box available--a black box based on finite element analysis, which is capable of simulating constitutive behavior of each component (the backfill, the foundation, the reinforcement, and the facing) and the interactive behavior among all the components, including the sequential construction operation. Not that many years ago, use of the black box programs which are based on limit equilibrium analysis is far more economical; however, this advantage is fast diminishing with the rapid advancement of computer technology. It is true that if a designer understands the formulation of a computer program better, he/she may feel more comfortable with the computer program. However, it is really not difficult for a designer to understand the concept of finite element analysis and its limitations. I think we should not paint finite element analysis as a difficult tool and thus discourage its use in routine designs. We as researchers could certainly make a conscious effort to simplify the usage of reliable finite element models for the design of GRS structures.

REFERENCE

J. T, H. Wu, R. K. Barrett, and N. S. Chou (to be published in 1993). Developing Cost-Effective Geosynthetic-Reinforced Soil Walls: Recent Efforts in Colorado, U.S.A. Balkema Publisher.

• COMMENT

J.T.H. Wu
(University of Colorado, Denver, U.S.A.)

I would like to comment on the method of attachment of strain gages on a geotextile to measure its deformation. Some of you probably realize that the difficulty arises from the fact that most geotextiles are weaker than the adhesives which are commonly used for attaching strain gages. This is especially true for low and medium stiffness/strength geotextiles.

When the adhesive is stronger than the specimen (i.e., the geotextile), the deformation characteristics of the adhesive will dictate the strain measurement. The deformation characteristics of the adhesive depend, among others, the thickness of the adhesive, which is very difficult to control in typical applications. On the other hand, the use of a weak adhesive is not advisable as it is generally not adequate for secure mounting to the specimen.

At the University of Colorado at Denver, we have been using a unique attachment method for mounting high elongation strain gages to geotextiles. The method has been used in model tests (Billiard and Wu, 1991; Helwany and Wu, 1992), full-scale tests (Wu, 1992) and field tests (Chou, 1992) with very satisfactory result.

The attachment method employs an epoxy glue which is applied only at the two extremities of a high elongation strain gage, in stead of the conventional method of applying a special adhesive over the entire area of the strain gage. It is to be noted that, before the glue is applied, a removable adhesive tape is emplaced over the strain gage covering the deformable portion of the strain gage (i.e., the area over which glue will not be applied). The adhesive tape is removed after the glue is set. This is to ensure close and smooth contact between the gage and the geotextile surface.

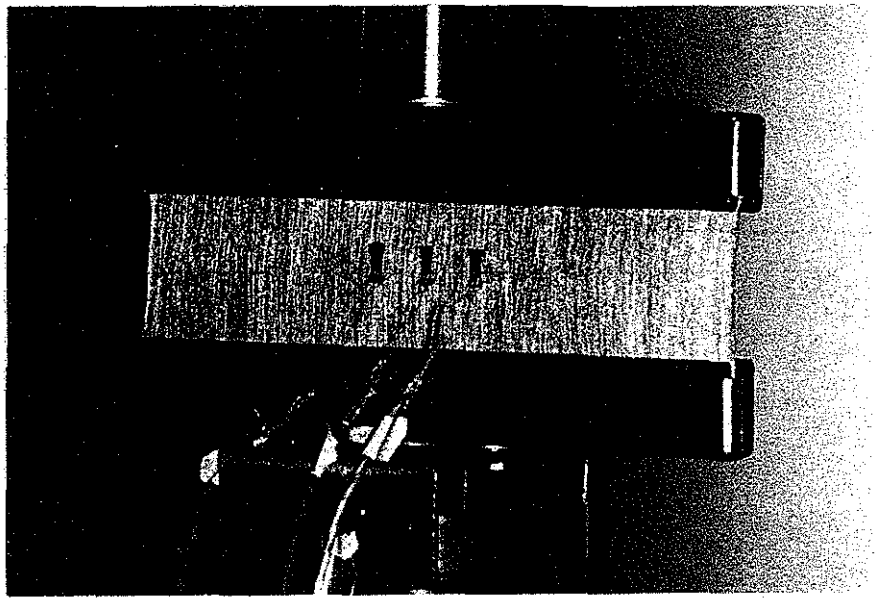

Figure 1 Uniaxial planar test of a geotextile specimen instrumented with strain gages

The attachment method has been recorded on a video tape by Mr. John Billiard, a former graduate student of the University and the developer of the attachment method. The video tape gives detail illustration of the attachment method, including how to solder leads to a strain gage, how to test its balance, how to degrease geotextile surface, how to mount the strain gage to the geotextile, and calibration of the instrumentation.

To examine repoducibility of the method, three civil engineering students learned how to use the method by following the instruction of the video tape. The instrumented geotextile specimens were tested in a uniaxial planar loading condition (see Figure 1). The results are summarized in Figure 2. It is seen that the strains recorded by these strain gages are about 67% of the average strains measured by dial indicators. The relationship is very consistent and the attachment method is highly repeatable.

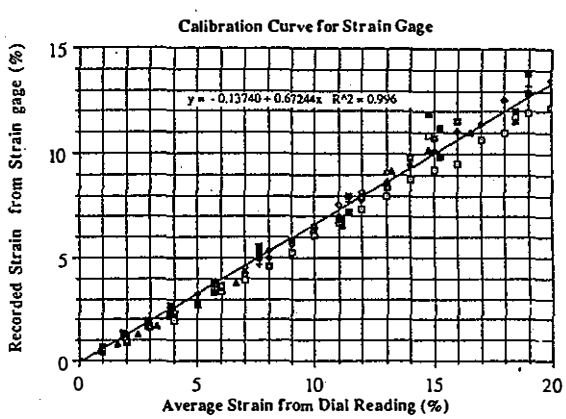

Figure 2 Results of calibration tests from more than 20 strain gages

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1] J. W. Billiard and J. T. H. Wu, 1991. Load Test of a Large-Scale Geotextile-Reinforced Retaining Wall," Geosynthetics '91 Conference, Industrial Fabrics Association International, pp. 537-548.

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3] J. T. H. Wu, 1992. Construction and Instrumentation of the Denver Walls. J. T. H. Wu, Editor. Geosynthetic-Reinforced Soil Retaining Walls, Balkema Publisher.

4] N. N. S. Chou and J. T. H. Wu, 1992. Investigating Performance of Geosynthetic-Reinforced Soil Walls. Geosynthetic Research Report, University of Colorado at Denver, No. GR-92-03, also Final Report to the Colorado Department of Transportation.

• QUESTION TO JONES

Q : G.W. Won
(Road and Traffic Authority, Australia)

I was very interested in the application of high strength geotextiles & geogrids to support embankments over areas of land subjected to mine subsidence.

In Australia as in the UK we have similiar problems where proposed road construction traverses old coal mining areas where bord and pillar methods were used. Secondary extraction of coal pillars in some areas have caused sinkholes to develop.

The question I would like to ask Professor Jones is " Is there a practical upper limit to the void opening size which could be effectively bridged by a high strength geotextile or geogrid under a road embankment say 6 metres high ? "

A : C.J.F.P.Jones
(University of Newcastle upon Tyne, U.K.)

The effectiveness of using a geogrid or geotextile as a tension membrane to span voids depends upon a number of factors. The primary factor is the limit on deflection which is acceptable in the case being considered. If the tension membrane is being used to support the ground over a playing field, the main interest is in avoiding complete collapse, significant deflections or displacements of the surface would be acceptable. In the case of a high speed road, minor deflections greater than 1% could be deemed to be unacceptable.

The amount of deflection which occurs is influenced by a range of factors including the dimensions of the void, the properties and thickness of the overburden or the stiffness of the geogrid or geotextile being used. In answer to Dr. Won's question, the depth of the overburden (height of the embankment) has a significant influence, high embankments are less susceptible to the effect of the creation of a void, thus an embankment 8m high can sustain a larger void than one of 6m in height. In the United Kingdom, the largest voids are likely to be caused by the collapse of a mine shaft. Shaft diameters of 10m are possible but it is likely that the diameter will be in the range 3-5m. The use of tension membranes to provide support for these events is practical.

• QUESTION TO YAGI

Q : C.C. Huang
(National Cheng Kung University, Taiwan)

The authors reported that the calculated value of the bearing capacity of footing placed on the crest of a sandy slope by using a so-called "Generalized Limit Equilibrium method" coincides with the experimental ones. However, this result may be fortuitous, because:
(1) the value of traction force used in the analysis is unclear.
(2) the footing load introduced by the loading system shown in the paper could be eccentric and inclined from vertical direction (Huang, 1990, Tatsuoka and Huang, 1991). However, the analysis aims at predicting the bearing capacity of footing vertically loaded with central load. According to the test result by the discusser, the loading system used by the authors may overestimate the bearing capacity of footing vertically loaded with central load up to 110%.

It is clear, therefore, that the utilization of the peak value of friction angle ϕ (=46°) under plane strain compression condition in the analysis is to compensate the possible overestimation of the test result, and the calibration presented in this paper may have little meaning.

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A: N. Yagi, M. Enoki, R. Yatabe and J. Edirisinghe
(Ehime University, Japan)

The authors wish to thank Dr. Huang for his interest on their paper. The answers for the four questions are given sequentially as follows.

Breaking strengths of reinforcement materials used for both element and model tests were measured

using extension tests using an apparatus as shown in the Fig. 12 and values are as given in the page 716 and the Figs. 9, 10 and 12. After observing the failure pattern of reinforcements it was realized that reinforcements were on tensile failure rather than on pullout failure.

To overcome the sidewall frictional effects two steps were taken as described in the page 717 of the proceeding.

Before answering for the last two questions, the authors would like to direct discusser's attention to the stress distribution obtained by slip line field under a footing on a level ground and on a slope as shown in the Fig. 13.

According to this for the case of footing on a level ground center line of footing is coincident with the total force acting point, whereas in the case of footing on a slope, it is not like that.

The experiments described in the paper were carried out controlling the displacement of footing to be vertically downward and measuring the resulting force. Therefore, even it is obvious to act the resulting force eccentrically from the center line and to be inclined to the base of the footing, if a roughness exists on the base , it is not possible to control. In this kind of experiments what can be controlled is the displacement of the footing but not the resulting force on it.

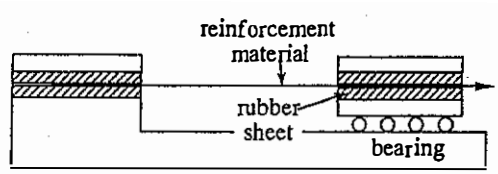

Fig. 12 Apparatus used for measuring breaking strength of reinforcement materials

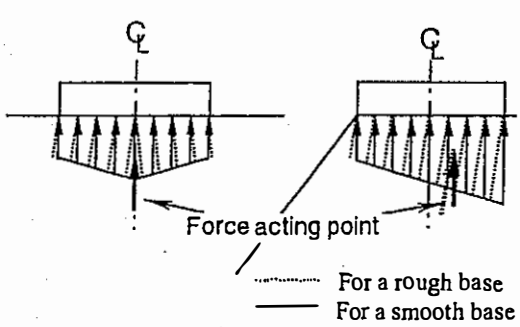

Fig.13 Stress distribution under a footing
a) on a flat ground
b) on a slope

• COMMENT

F. Tatsuoka
(*University of Tokyo, Japan*)

Discussion on the keynote lecture by Leshchinsky,D. (1992): Issues in geosynthetic-reinforced soil," Proc. of Earth Reinforcement Practice, IS-Kyushu '92.

In Session 2.1 **Analysis: Limit Equilibrium**, with respect to the design strength parameters of soils to be used for limit equilibrium-based stability analyses of geosynthetic-reinforced soil retaining walls, it is stated "since all soils strata are not likely to reach their full strength simultaneously, it is recommended to use design values (c_d, ϕ_d) that do not exceed the residual strength within each layer."

This behaviour is known as the progressive failure, as described, for example, in Huang and Tatsuoka (1990) and Tatsuoka et al. (1991) for the bearing capacity of footing on sand. The discussor agrees with this view in general. The discussor has the following two comments, however.

1) For reinforced sand, the degree of progressive failure is a function of the degree of reinforcing. Huang et al. (1991) and Huang and Tatsuoka (1992) found that for the same dimensions of model sand slopes reinforced with metal strips, the degree of progressive failure, which is the degree of non-uniformity in the shear strain in soil along the potential failure surface at the moment of the failure of slope, increased as the slope was reinforced to a larger extent. This means that the use of the same residual soil strength will lead to different actual safety factors depending on each design conditions including the degree of reinforcing.

2) If we use the same design strength value of a given type of soil for poorly- and well-compacted backfills, we cannot evaluate properly the effect of compaction of backfill in design. No doubt, better compaction of backfill is essential for stable and less-deformable walls. It would be necessary, therefore, to use a higher design strength for a better compacted backfill.

Despite the above comments, at present the discussor cannot propose a reliable general guide for design soil strengths which are functions of the degree of reinforcing

and compaction. The study in this direction is now being undertaken by the discussor.

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• COMMENT

C.C. Huang
(*National Cheng Kung University, Taiwan*)

The authors declared that their analytical results are very close to the experimental ones. However, this fortuitous conclusion resulted from the concurrence of the miss-leading theoretical approach and the abusing of published experimental results.

Some fundamental mistakes in the theoretical approach are:

(1) the passive resistance R acting on the vertical face ab could not be increased due to placing the reinforcement horizontally in the passive zone. Because the horizontal reinforcing strips act as compressive members in this zone, and

(2) the active thrust R' acting on the vertical face ab could be reduced rather than increased if reinforcement is placed horizontally beyond the potential failure surface ob.

Additionally, None of the published experimental data should be used for the verification of this erroneous model because the arrangement of the reinforcement in these experiments were completely different from that described in the authors' analytical model. Lastly, the discusser feels that the recommended length of reinforcement which is "effective" to increase the bearing ranging from 3.39B to 7.33B is quite arbitrary.