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Chairman's report: Testing methods

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ABSTRACT: The excellent long term performance of reinforced soil structures is highlighted. However, it is observed that current techniques for testing and their use for establishing design parameters may result in overly conservative design. In this context, nine questions relating to testing are discussed. It is concluded that there is still considerable research required to identify the reasons for the overdesign evident in many monitored reinforced soil structures. The use of low quality fills in reinforced soil structures was identified as the area where there is the greatest potential benefit for expanding the use of reinforced soil but also noted that much more needs to be known before reinforced soil structures with low quality (eg. cohesive) fills could be used with confidence in regular applications.

The Summary Discussion Sessions at IS Kyushu '96 were initiated with the intention of evaluating where we are after 30 years of experience with respect to modern reinforced soil structures and after three truly international symposia on the subject (Kyushu '88, '92 and '96). The proceedings of IS Kyushu '96 alone contain 151 papers with 32 of these papers dealing directly with testing and materials and a number of papers being related to testing and materials. This alone highlights the level of continued interest in the subject. The technical literature abounds with examples of reinforced soil structures that have performed well and indeed Kyushu '96 contributes many more papers to this collection. Of particular significance is the excellent performance of reinforced soil structures during recent earthquakes (eg. see White & Holtz, 1996; Tatsuoka et al., 1996; Frankenberger et al., 1996; Kobayashi et al., 1996; Nishimura et al., 1996; Otani et al., 1996). Recognizing that field performance (particularly under extreme conditions) represents the ultimate test, it is useful to begin any discussion on testing and materials by reflecting on how reinforced structures have behaved in the field. From that starting point we can then examine what we know and identify what we need to know and what research needs to be done in the future.

Observation of the performance of reinforced soil walls has demonstrated that under normal design conditions (and sometimes under even extreme conditions) the strains mobilized in the reinforcement are quite small (less than 1%: eg. see Benigni et al., 1996; Tajiri et al., 1996) and that the deformations at the face of the wall are also small (typically less than 1.6% of wall height: eg. see Boyle & Holtz, 1996). Even walls constructed on highly compressible deposits and with cohesive backfill (eg. Kojima et al., 1996) perform well provided they are appropriately designed and constructed with reinforcement that also provides for drainage and provided that large deflections (primarily settlement) can be tolerated.

Observation of the performance of reinforced embankments provides similar reassurance. Under normal design conditions, the strain in the reinforcement is usually smaller than anticipated and it has been observed that in many cases the strain decreases with time as pore pressures dissipate (eg. Barsvary et al., 1982; Rowe et al., 1984; Rowe & Mylleville, 1996). However, the opposite trend can be observed for "unconventional" soils that exhibit a significant visco-plastic response (eg. see Rowe & Gnanendran, 1994; Rowe & Hinchberger, 1995) since for these soils the strain in the reinforcement can increase significantly following construction as the soils shed load to the reinforcement. Design procedures for use of soil reinforcement, in conjunction with staged construction, has also proven to be very effective in the design and construction of embankments on peat (eg. Rowe & Mylleville, 1996; Oikawa et al., 1996) although it must be recognized that there will be significant settlement and care is required to control the rate of pore pressure build-up in peat during construction (see Rowe & Mylleville, 1996).

The one area where, despite successes, there are also potential problems is in the use of cohesive backfill in walls and steep reinforced slopes. Here, the potential problems appear to be developing adequate interface resistance if some geogrids are used and basic soil mechanics issues related to creep of the material and the pressure that can arise when tension cracks develop in the cohesive fill.

The lower than designed strains in the reinforcement of reinforced soil walls, slopes and embankments appears to be the result of several factors: (1) Conservatism in the selection of reinforcement parameters and the fact that the stiffness (and strength) of the reinforcement is often far greater than actually assumed in the design (this will be discussed in more detail shortly). (2) Conservatism in the selection of soil parameters where (a) the designer assumed strength of backfill is usually well below that actually mobilized in the field; (b) there is often matric suction due to the presence of moisture in the fill that gives rise to an apparent cohesion and a substantially higher than assumed strength in the fill; (c) the designer usually adopts a conservative assessment of the undrained strength of the foundation - this can involve adopting a conservative estimate of strength based on the scatter of data as well as an underestimate (or total neglect) of strength increase with depth (which can be masked by the effects of sample disturbance unless appropriate care is taken). (3) Redundancy in the reinforced soil system which is neglected in design (eg. toe resistance for walls and the presence of a root mat or crust for embankments on soft soil); and (4) conservatism inherent in the simplified analysis.

Of these issues, the first (conservatism in the selection of reinforcement) was a major discussion topic in this session (see Voskamp, 1996; Greenwood, 1996) with it being argued that the current approach to limit states design is resulting in unrealistically low strength (and stiffness) being used in design; we will return to this. With respect to conservatism regarding soil parameters - there is some inevitability to engineers being conservative in neglecting apparent cohesion in design (since it is readily lost due to either drying or an increased degree of saturation). More debatable is the choice of the design friction angle - but this debate will be left for the session on Design Methods (Bolton, 1996). Less justifiable is the conservative interpretation of undrained shear strength profiles with depth for embankments on soft clay; here there is sound fundamental argument for considering the real strength increase with depth and the reason for ignoring it is usually either (a) failure to recognize that solutions exist for considering the bearing capacity on soil whose strength increases with depth (eg. see Davis & Booker, 1973; Rowe & Soderman, 1987) or (b) failure to recognize the increase in strength with depth due to sample disturbance which tends to increase with increasing depth and often masks the actual strength increase (this can be dealt with by the use of better sampling techniques). The issues of redundancy and the conservatism inherent in simplified analyses while warranting discussion are beyond the scope of this report on "Materials and Testing". Hopefully, they will be addressed in the report on "Design Methods" (Bolton, 1996).

The Summary Discussion Session on "Testing and Materials" involved panel presentations by Prof. Makiuchi (Geosynthetics Testing in Japan), Mr. P. Segrestin (Monitoring of Reinforcement Durability and Thoughts About Experience and Needs), Mr. W. Voskamp (Proposed Method to Determine the Safety Capacity of Reinforced Soil Structures During Its Lifetime) and Dr. J. Greenwood (Residual Strength: An Alternative to Stress-Rupture for Earth Reinforcement Design). In addition, there were five oral paper presentations by Profs. Palmeira (Palmeira et al., 1996), Chang (Chang et al., 1996), Mitachi (Nakamura et al., 1996), Gourc (Morel & Gourc, 1996) and Adanur (Adanur et al., 1996). The session was organized to address a number of questions; the remainder of this article will itemize these questions and the corresponding presentations/discussions.

1. Are current standard tests (ISO/CEN/ASTM) adequate?

Mr. P. Stevenson indicated that the standard ASTM tensile test (D4595) gave tensile stiffness results that were too low for high strength materials and that the sample length affects the results. He showed that for one geotextile tested, the tensile stiffness for 100 mm gauge length was less than half the value obtained for a 450 mm gauge length. He indicated that the gauge length should exceed 450 mm if meaningful results are to be obtained.

Mr. B. Myles agreed that the problem is real but a much greater problem is the poor definition of the initial part of the extension curve and he was critical of both the ASTM and ISO wide width tensile test methods. He stated they use much too high a preload and this masks quite a lot of the "softness" of the geotextiles. Consequently, people believe that the geosynthetic will take up load much quicker than they do since many geotextiles need 1% extension (or more) before they come onto what is normally called the "tensile curve".

Mr. Stevenson accepted Mr. Myles' comment and said that they looked at the initial part of the curve (with no preload). Dr. Greenwood indicated that he was not aware that ISO 10319 was inadequate for strong materials.

It is clear from the discussion that while current tensile index tests serve a useful purpose in identifying changes in the tensile characteristics of a given product, there is still some question regarding the validity of comparing tensile characteristics of different products on the basis of the existing ISO and ASTM tests both for nonwoven geotextiles (whose in-soil stiffness may be greater than the in-isolation stiffness) and for some high strength geosynthetics where gauge length may result in the tensile stiffness being underestimated.

2. How different are the stress-strain and creep characteristics of geosynthetics in unconfined tests from those in-soil, and what are the problems of doing confined tests?

Three papers in the proceedings address this question (see Adanur et al., 1996; Chang et al., 1996; Palmeira et al., 1996) and numerous questions arose from the floor. Dr. Yogarajah commented that the stress-strain behaviour of geosynthetics in both the unconfined and confined state tests are different from the stress-strain-behaviour of geosynthetics in walls and slopes. Dr. Palmeira agreed and indicated that even the confined tests are "index tests" in that they do not fully duplicate field conditions but that they do give a better indication of likely behaviour than unconfined tests (at least for some types of geosynthetics).

Questions then arose regarding the difficulty of isolating the effects of friction and boundary conditions when performing confined tests. Dr. Palmeira indicated that lubrication of the membrane had a significant effect on the results (as compared to those obtained using an unlubricated membrane). This clearly shows that there are potential problems of boundary effects (especially if an unlubricated membrane is used). What is not yet clear is what effect the lubricated membrane has on the results. Dr. Palmeira stated that much of the improvement in the mechanical behaviour of nonwoven geotextiles was due to soil becoming impregnated into the geotextile and thereby reducing fibre mobility.

Dr. Alexiew suggested that we need to carefully consider how the results of laboratory tests will be used before embarking on extensive testing. All the tensile tests being conducted confined in soil are similar to pullout tests and the tensile force is applied externally to the reinforcement, then from the reinforcement to the soil and then to the test box. The results may be applicable to equivalent situations in the structure - however, this is only in the anchoring zones; in all other zones of the reinforced structure there is the reverse situation where the soil mass is loading the reinforcement. Thus, in reality, in 70-90% of zones in the structure the soil mass is loading the reinforcement (this is why we need the reinforcement) and hence the so called confined tests are not really relevant to these regions of the soil and hence one must be very careful in applying the results from confined tests to real structures. Dr. Chang responded that they were in an early stage of their research and he wanted to understand if frictional resistance reduces creep and the results were not intended to be used in design. He agreed that his test does not simulate in-situ conditions.

In support of Dr. Alexiew's comment, Mr. Voskamp noted that in confined tests, movement between a geosynthetic and the confinement soil will take place during loading, due to the extensibility of the geosynthetic. This will give a lower load in the confined area and gives rise to the following questions: How representative is this test? What is the actual load level and what is the actual strain in the geosynthetic in the confined area? How is the different strain in the unconfined area considered? He stated that, in fact, the confining stress results in increased friction between the fibers which explains some of the short term improvement in stress-strain behaviour relative to an unconfined test. However, there is no long term data to confirm the short term results.

Prof. D. Leshchinsky indicated that except for nonwoven geotextiles, there is a problem with confined tests since people are confusing the intrinsic material properties with the effect of the interfaces (which increases the apparent stiffness). He suggested that the confined test is unnecessary in these cases.

Prof. Wu stated that

"(i) the creep/potential of geosynthetics as evaluated by the in-isolation creep tests presented in this discussion session (i.e., tests performed on a geosynthetic specimen) is misleading;

(ii) subjected to the same load, a geosynthetic can exhibit very different creep behaviour when it is embedded in a granular soil versus a cohesive soil--because of soil-geosynthetic interaction;

(iii) a soil-geosynthetic long-term interactive test method (Wu & Helwany, 1996) has been developed (and used) at the Colorado Transportation Institute to account for the effects of soil."

Dr. Greenwood responded to the effect that: "Experimenters have been struggling for over 20 years to devise a reliable and acceptable method for measuring the mechanical properties of geosynthetics in soil and I am pleased to learn of Professor Wu's success. Other laboratories should start intercomparison tests as soon as possible to confirm the reproducibility of his results, prior to establishing the method as a standard and building it into the design codes." Interest was also expressed in the cost of this test.

Mr. Voskamp did not agree with Prof. Wu's comment that creep tests performed in-isolation are misleading. He stated that "in-isolation tests give insight regarding the polymer characteristic: creep. When tests are executed in-confinement this creep may be restricted or slowed down, but at the end the in-isolation obtained results are the ultimate failure level. Because of soil-geosynthetic interaction the results of the in-confinement tests may give more optimistic results. However, there is no proof in practice that the in-confinement tests give representative values. The actual load level in the test specimen is not the same as the applied load due to the same soil geosynthetic interaction. Further, the load, soil and vertical stress vary in every application. The statements of Jonathan Wu are based on limited results and as such should not be considered valid in general terms."

In response to Prof. Wu's comment about creep in confined tests, Mr. Segrestin commented that the debate about the way in which creep tests and (even more useful) creep rupture tests should be performed in order to take the confining effect of the soil into account is certainly an important one. However, in order to avoid hasty or optimistic conclusions, it seems useful to note that this confining effect is highly dependent on the configuration of the reinforcement. For example, it is presumably notable for a bidirectional isotropic mesh, but negligible for a linear strap.

3. Do pullout and shear box tests really tell us what we need to know and what are the problems with interpretation? (See Adanur et al., 1996; Ghosh & Bhasin, 1996; Hayashi et al., 1996; Konami et al., 1996; Lin et al., 1996; Lopes & Ladeira, 1996; Segrestin & Bastick, 1996; Chang & Milligan, 1996.)

Prof. Makiuchi had presented details of the standard pullout tests in Japan and indicated the pullout box was 30 cm square and 20 cm high. The specimen width was the same as the box. Dr. Chang indicated that in his experience one can get misleading results from this test because even if you do reduce friction the distribution of vertical stress is not even and this may have a significant effect on the results of the test.

Dr. Palmeira raised questions regarding what would be the appropriate length of sample for assessing the equivalent friction angle (interaction factor) and also the boundary effects that can occur in these tests. Mr. Voskamp stated that the shear box should be large (approximately 2 x 1 m with 2 x 0.5 m height is common these days). Prof. C. Jones stated that the boundary conditions at the face of the pullout box are very important and that this is currently being examined in Europe. Prof. Wu pointed out that there is a technical paper by Sohbi and Wu in the next issue of the Geosynthetics International Journal which addresses how to obtain a consistent interpretation of pullout test results.

Dr. Lo stated that a rigid sleeve can be a problem in pullout tests and that one needs a flexible sleeve. However, he stated that a more fundamental problem is the progressive failure that occurs along the reinforcement. He has found that in the pullout box the geosynthetic moves a lot relative to the soil and hence gives a lot of progressive failure however in a field case with which he has been involved the reinforcement is approaching the limit state at much less relative movement between the soil and geosynthetic and hence there is less progressive failure and a higher apparent friction angle in the field. The pullout test gives lower friction angles as the length of the box (and sample) is increased.

Dr. Pokharel observed that the pullout test is usually performed under passive conditions whereas in the field the anchorage length is located behind a zone where active conditions prevail and hence he questioned the validity of results obtained from a pullout test for assessing properties required for determining the anchorage length in walls and slopes.

4. Is it practical to develop a meaningful test to establish the effect of construction damage to reinforcement?

Dr. Greenwood responded to this question by stating that he does not believe that one can assess the resistance of all types of geosynthetic to mechanical damage (or robustness) solely on the basis of the traditional tests of tensile strength, static and dynamic puncture and mass per unit area. There are of course large scale performance tests which measure the reduction in tensile strength under simulated site conditions, but the results are specific to the backfill used and the tests are expensive to carry out. There has been considerable experimental work in Europe directed at the development of a simpler laboratory test for resistance to mechanical damage, in particular the dynamic compression of a sample between two layers of angular fill, followed by visual examination for holes and by tensile testing. This method has recently been put forward as a European prestandard, to be supplemented by a further method related to a geotextile over soft ground. There will be a need for international experimental validation to examine whether these tests are representative and practical, and for guidance to the designer as to how he is to apply the results in practice with reference to his own site conditions and the function of the geosynthetic.

5. How does reinforcement influence the dynamic properties of the reinforced mass?

Although many papers addressed the issue of the performance of reinforced walls during earthquake, only one paper (Chen et al., 1996) investigated dynamic behaviour in the laboratory and made a number of conclusions. Dr. Shadu noted that the tests reported in this paper were performed on an "ideal" Ottawa sand and cautioned that similar tests should be performed on other, more typical, sands before extrapolating the results to field conditions.

6. How to interpret triaxial and biaxial test results?

As noted in the paper on page 111 of the proceedings, triaxial tests have been performed to illustrate the benefits of reinforcement. Dr. Pokharel noted these tests are very difficult to interpret since they begin as an axi-symmetric test but the conditions of the test change due to the presence of imperfections and strain localization which remove the symmetry. The biaxial test proposed by Morel and Gourc (1996) has the potential to provide more consistent (tractable) testing conditions although the effects of sample dimensions and boundary effects still require careful examination when interpreting the test results.

7. Are the techniques used to reduce side wall friction in controlled system wall tests (eg. RMC, Denver, Tokyo) effective at reducing side wall friction to negligible values throughout the test?

Large-scale laboratory tests on prototype walls have been performed by a number of investigators (eg. Jarrett & McGown, 1987; Wu, 1992; Huang & Huang, 1996; Tajiri et al., 1996; Tsukamoto et al., 1996). These tests are often used either to extrapolate to field conditions or to test numerical models that are then used to extrapolate to field conditions. It is noted however that these tests may be difficult to interpret and the question arises as to why. For example, on page 533 of the proceedings, the test reported by Tsukamoto et al. (1996) is reported to give a coefficient of active earth pressure of $K_a \approx 0.1$ without any reinforcing for a wide range of applied surcharge pressures (49-294 kPa). This implies a consistently very high friction angle for a relatively uniform ($C_u = 1.7$) sand at 90% relative density.

In this, as in many other similar tests, lubricant and plastic membranes are put on the walls to reduce friction between the backfill and the walls. Prof. Bathurst asked the question both of this and other tests - are plane strain conditions really achieved? Since plane strain conditions are usually assumed in interpreting the results this is a key question. He noted that even when using lubricant and plastic or latex membranes between the sand and walls of the test box, the actual friction angle is still potentially significant, particularly for low ratios of test box width (L_w) to height (H). He indicated that even though people report friction angles as low as 1° for lubricated sides, his experience is that while the friction angle may start off that low, chemical interactions between the membrane and lubricant as well as penetration of soil particles into the membrane increase side wall friction with time. Dr. Gassler indicated that he fully agreed. He said that qualitatively these tests give good results but quantitatively you need to be careful when backanalyzing the results due to the side friction.

Drs. Wu and Huang agreed that this can be a problem but stated that in their tests the problem is minimal. Dr. Wu indicated that in his tests, the lubricant used was a silicon grease (manufactured by Shin-Etsu Chemical Company in Japan). The particular grease is compound KS 63G, comprising a thin silicone base oil and fine angular silica fillers. The membrane used was a thin latex membrane. The thickness of the membrane varied according to the soil type employed in the test. Fumio Tatsuoka, University of Tokyo, recommended a membrane thickness of 200 micrometer for a clean sand. For long term tests he agreed that there is a problem. Dr. Bathurst indicated that in his experience "long term" may be as little as one day. Dr. Huang agreed with Dr. Bathurst that it is important to have a wide specimen (ie. $L_w/H > 1$) in order to minimize the side wall effects and to approach plane strain conditions.

It was clear from the discussion that researchers reporting results from these types of test should provide data to support the friction angle of the wall interface over test periods similar to that of the actual test being reported (see Bathurst & Benjamin, 1988). Another factor that should be reported is the stress-strain characteristics of the soil under the same conditions of moisture content that were used in the test wall; if the sand is moist this may significantly influence the interpretation of the test results.

8. How does one assess environmental damage to reinforcing materials?

A number of papers in the proceedings (see Jailloux & Anderson, 1996; Smith et al., 1996; Greenwood & Yeo, 1996; Mak & Lo, 1996) as well as the Panel Presentation by Mr. Segrestrin, Mr. Voskamp and Dr. Greenwood address, at least in part, the issue of environmental damage. Mr. Kassner questioned the paper by Jailloux and Anderson (p. 45) and the presentation of Mr. Segrestin which discussed the performance of polyester yarn at a pH of 1 (in HCl), 10 (in Lime) and 12 (in NaOH). He asked how would steel perform under the same conditions and under what real soil conditions would one expect to find a pH of 1 or 12? Mr. Segrestin indicated that this work was for "research purposes" and that it confirmed what was already known: that polyester is degraded at very high pH but low pH is not aggressive. In contrast, steel is attacked at low pH but performs well at high pH. Dr. Alexiew cautioned that one must distinguish between external and internal hydrolysis. External hydrolysis corresponds to chemical attack but internal hydrolysis can be examined in water. He indicated that a great deal of work has been done on this issue in Germany and suggested that attendees read the paper by Wilmers (1996) which concludes that there is no hydrolysis occurring in real, normal soils including typical alkaline soils (but excluding lime stabilized soils). Dr. Alexiew also stated that they have recently exhumed a polyester geotextile that was submerged for 14 years and that tests indicate a rate of degradation of about 1% per decade.

In discussing the paper by Smith et al. (1996, page 151) and the panel presentation by Mr. Segrestin, Prof. Jones commented that in his opinion corrosion was a function of surface area and the best shape for steel reinforcement would be a circular section rather than a flat strip of the same area. He said that if you are concerned about pitting corrosion, a pit of a given size is likely to be more critical for the flat strip than a circular bar. He suggested using a circular reinforcing element with a plate anchor (as used in Japan) or a triangular anchor (as developed in the U.K.) and stated that this would minimize the corrosion problem for steel.

Mr. Segrestin replied that "Professor Jones is right when he notes that a round bar is at first more resistant to corrosion than a flat strip. It is clear that, if we compare two reinforcements having the same cross-section, for example a 40 mm x 5 mm strip and a ϕ16 mm bar both with a 200 mm^2 cross-section, an average loss of thickness of say 1 mm would have a smaller impact on the bar. The residual cross-section would be indeed 160 mm^2 for the strip and 177 mm^2 for the bar.

However, the study we made does not relate to reinforcements with the same cross-sections, but to reinforcements which are actually used by the industry, for example 50 mm x 4 mm strips compared to wire-meshes made of ϕ8 mm or ϕ10 mm bars. Our study also deals with the effects of local pitting, rather than with average uniform loss of thickness. As shown in the paper by Smith et al., such pitting proves to be more detrimental to the long-term resistance of small round bars and would justify larger sacrificial thicknesses for design."

Professor Fukuoka stated that it is important to look at case records. He first used steel bars as reinforcement in 1963 and the structure is still standing without any problems. He constructed a tall reinforced earth wall in Kyushu in 1967; and it is still standing without any problems. He used PVC in 1965 and still no problems. He recommends that we collect more case records to contribute to the discussion on durability. Dr. Greenwood responded by stating he agreed entirely that when samples are exhumed, field records should be compiled with as much information as possible on the soil environment, temperature, rainfall etc. One must then ask the question: "Given the environment to which they were exposed, would we have expected the geosynthetics to have degraded?". Exhumations which end up with the conclusion "nothing was expected to happen, and nothing happened" are of limited value.

Mr. Segrestin responded that he agreed with Prof. Fukuoka's statement that actual field records are indispensable for a complete understanding of the question of durability as emphasized in his presentation. He stated that it is not a surprise that structures using steel perform very well after 30 or 35 years; they will undoubtedly remain in service for many more decades. However, he understood that the structures mentioned by Prof. Fukuoka are made up of relatively big galvanized steel anchors or multi-anchors. Again, the discussion in his presentation and in Smith et al.'s paper is about the sensitivity of small bars, only a few millimeters in diameter, to the effects of superficial pitting and this may not be relevant to the structures described by Prof. Fukuoka.

9. How do we evaluate the design strength of reinforcement based on test data?

The panel discussion by Mr. Voskamp and Dr. Greenwood (as well as Greenwood & Yeo, 1996; Mak & Lo, 1996) addressed the question of how to assess the design strength of geosynthetic reinforcement. There is a clear implication that current limit state design procedures (eg. U.K. BS8006) are overly conservative.

In his presentation, Mr. Segrestin implied that the time-temperature shift commonly used to predict long term performance might be questioned under extreme conditions of high pH (Jailloux & Anderson, 1996). Mr. Segrestin cautioned that these are very preliminary results and they are very aggressive environments and here the Arrhenius coefficient is non-linear but they need more results.

Dr. Greenwood stated that the temperature transposition involves the use of short term tests at higher temperatures to predict longer term effects at lower temperatures. For it to be valid, the physical and chemical mechanisms operating at both higher and lower temperatures must be identical, and there should not be any change in the physical state of the polymer between them. In polyesters there is a change in physical state at about 65 degrees celsius, above which the molecules in the amorphous regions (as opposed to the crystalline regions) of the fibres become more mobile. This will, among other things, expose the molecules more to chemical attack, including hydrolysis. Extrapolation of data on the rate of hydrolysis from tests performed above 65 to temperatures below is therefore not strictly valid. Since the phase transition reduces the sensitivity to hydrolytic attack at lower temperatures, however, the extrapolation is likely to be conservative and will predict too low rather than too high a lifetime. In the absence of other data, therefore, a prediction based on a temperature transposition of this kind can be regarded as a minimum value.

Mr. Voskamp added that there can be a difference between the results obtained above and below the glass transition temperature. Long term creep testing has been executed for PET up to 60°C and time temperature shifting gave a perfect fit within that range (see Greenwood & Yeo, 1996).

Dr. Lo asked how one could experimentally determine the residual strength curves for a given material. Dr. Greenwood replied that the residual strength curve is determined in the same manner as for stress-rupture, but the test is interrupted after a fraction of the anticipated time-to-rupture and the tensile strength of the specimen determined.

A key issue in the discussion of limit state design is the identification of what one is trying to do. As noted by Dr. Greenwood, if the "factor of safety" is intended to guard against miscalculation of the overall load on the product, in other words, if the sustained load in practice could be higher than the design load, then the design process must be based on stress-rupture as before. The residual strength approach is valid where the safety factor is intended to maintain a certain reserve of tensile strength throughout the design life to guard against sudden, instantaneous, increases in load - such as seismic events.

In response to Dr. Greenwood's presentation on residual strength, Mr. Segrestin opined that "the "residual strength" approach, i.e. considering the way in which the resistance of the geosynthetic reinforcement varies before it goes to creep rupture, does not finally modify the design procedure. One must bear in mind that the "overall factor of safety", usually of the order of 1.50, essentially accounts for the uncertainties in the applied loads, potential for local overstresses and imperfections of the computation methods. If the calculations are carried out as "working stress" calculations, this means that it cannot be excluded that the actual tensile load might be up to 1.5 times larger (T_d) than calculated (T_m). If the calculations are carried out as "limit state" calculations, the "load factor" is already included in the design load (T_d). In both cases one must check that, in the extreme case, the reinforcement will not break. In other words, one must make sure that T_d is just smaller than the creep rupture strength corresponding to the required service life (all other effects such as construction damage or environmental ageing being put aside here)."

Mr. Lawson noted that there were many important points made in Mr. Voskamp's panel presentation and that he wanted to emphasize some of these points. In particular, he noted that the issues of testing, partial factors and design need to be considered together and not in isolation. One needs to keep in mind the magnitude of the load factor and the material reinforcement factor when one comes to assess the components of the material reinforcement factor otherwise one can start looking for worst case values for the various components. He expressed concern that some people (and some papers in the conference) were going to an extreme in arguing for certain materials factors; he pleaded for logic and consistency in the selection of the materials component of the partial factors used in design. Mr. Voskamp agreed and stated that one has to remember that design life curves are logarithmic and that if one is not careful in designing for a 100 year design life but using excessive partial factors one may be designing for a 1000 year design life.

Mr. Myles commented that part of the reason for conservative factors in the British Standard (BS8006) is that there was not enough evidence available at the time to adopt a less conservative approach. These factors can be revised when more evidence becomes available. He pleaded for the manufacturers to provide more information.

In response to a question from Mr. Yokota regarding reinforcement characteristics relevant to seismic loading, Mr. Voskamp noted that seismic loading of the structure results in high frequency loading during a short period. When the load remains below the stress-rupture line level for the actual conditions, no problem is to be expected especially as the residual strength of the material will be higher than the stress-rupture value at the time of loading. Tests on polyester geogrids have shown that high frequency loading leads to increased stiffness of the material without influencing the strength.

Recognizing that most of the discussion had focused on retaining walls which have a long design life and whose long term performance depends on the reinforcement performing this role for the entire life of the structure, Dr. Sharma emphasized that we need to distinguish between "short term" and "long term" applications of soil reinforcement. He stated that creep is usually not a problem when reinforcing an embankment on soft clay since it is a "short term" situation. Mr. Voskamp agreed and stated that when the stress-rupture line is used for design, the corresponding rupture load can be determined for every design service life. This means for short term applications that the allowable load is much higher than for long term application.

In response to Dr. Sharma's comment, Mr. Segrestin opined that: "Imagining creep is not an issue for short-term applications could be deceptive. It should be borne in mind that the creep rupture strength practically decreases uniformly as a function of time, in a log-log scale. A product expected to have a long-term (100 years) creep rupture strength equal for example to 30% of its short-term breaking load, might break after about 10^5 hours (say 10 years) under 35% of this load, or after about 10^4 hours (say 1 year) under 45%. Its creep rupture strength could be even already reduced to about 70% or so after only one week!".

Concluding Comments

The papers on testing presented at the conference as well as the discussion in the Summary Discussion Session highlight the fact that although we now have 30 years of experience with modern reinforced soil structures and three international reinforcement symposia in Kyushu, there are still some unanswered questions and work for researchers and manufacturers to do. Much of this work is related to identifying the reasons for "overdesign" evident in many monitored reinforced soil structures and improving the parameters (based on testing) used in design. This relates particularly to issues such as the short term stress-strain properties of reinforcement, the time dependent behaviour of reinforcement, the effects of construction damage, and environmental impact on reinforcement. Areas where there is still considerable controversy include the use of standard index tests (eg. ISO/ASTM wide width tensile tests) for assessing design parameters for reinforcement, in-soil (confined) tests, the interpretation of large scale laboratory tests, and the selection of appropriate partial factors (for use in design) that account for stress-rupture, damage and environmental effects.

While there is still much useful work to be done, it is also evident that reinforced soil structures, whether they be embankments, slopes or walls, are performing remarkably well both under static and earthquake conditions. Where they have been monitored, the strains are generally well below what would be expected. Thus much of the discussion today has focused on us being too conservative however it is very comforting to know that current test methods combined with current design methodologies do generally result in structures which may have been a little more expensive than they needed to be, but have performed their design function well and by using soil reinforcement were usually constructed at a cost well below the cost of alternative (more conventional) designs. We are doing well. Reinforced structures may not be understood to the level we would like for research, we may be able to make them more efficient and reduce cost - but they are working remarkably well and we should be pleased that over the last 30 years of research and application we have achieved a great deal.

What about the future? There are many areas for future research arising out of the discussion. However, in terms of advancing the discipline, the use of low quality fills in reinforced soil structures would appear to require the most attention. This will involve dealing with some difficult (and still not fully resolved) issues of soil mechanics as well as issues of soil-reinforcement interaction. A number of papers at IS Kyushu '96 address this issue and I expect to see even more focus on this issue when next we meet again.

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