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Long term performance requirements for polymeric soil reinforcement in the United Kingdom

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ABSTRACT: Broad guidelines have been set down by the Department of Transport for the long term performance requirements for polymeric soil reinforcement and these are reviewed. A broad approach to design is then presented and this suggests minimum values of partial factors of safety which might be employed to ensure an adequate margin of safety against long term tensile rupture of polymeric soil reinforcement.

1 DEPARTMENT OF TRANSPORT REQUIREMENTS

The principal document dealing with requirements for reinforced soil walls and abutments is Technical Memorandum BE3/78 (DTp 1987). Although this only strictly applies to reinforced soil structures under the jurisdiction of the Department of the Environment, its recommendations are widely accepted for permanent structures in the UK. The Memorandum was originally issued in 1978 and primarily applied to metallic strip reinforcement. Proprietary reinforcing materials falling outside this spectrum were permitted provided they were approved and duly issued with a current Roads and Bridges Certificate. Among other things this certificate quantifies reinforcement design loads consistent with the 120 years design life specified in Technical Memorandum BE3/78.

2 REVISED TECHNICAL MEMORANDUM

Technical Memorandum BE3/78 was revised in 1987. Among other things this revision sets out the principles for the assessment of the tensile strength of non-metallic reinforcement such as polymeric strips and grids. The principles are applied in assessing the permissible tensile strength of materials which exhibit significant long term creep behaviour.

2.1 Principles for assessment of tensile strength

The Memorandum requires the basic permissible axial tensile strength to be derived on the basis of the following two principles:

i) At the end of the design life of the structure strains in the reinforcement shall not exceed a prescribed value. This is 0.5% for abutments and 1% for walls after completion of construction.

ii) During the life of the structure the reinforcement must not fail in tension, for example by brittle failure or though ductile instability.

The permissible axial tensile strength is taken as the lesser of either the permissible average axial tensile load based on long term creep considerations, or the permissible peak tensile stress based on reinforcement failure at a temperature of 10°C. The permissible values incorporate factors of safety but the Memorandum does not define numerical values for these.

2.2 Approval of new materials

Approval is based on the issue of a Roads and Bridges Certificate by the British Board of Agrement. The Memorandum gives no advice on how design strengths are to be determined, however, it does present a check list of factors which should be considered in assessing the mechanical properties and durability of reinforcement. In assessing mechanical properties the Memorandum makes reference to short and long term data relating to load-strain characteristics, creep, ductility and fatigue. Similarly in assessing durability consideration must be given to agencies such as site induced damage, chemicals commonly transported on highways, water, ultra-violet and infra-red light attack, bacteriological attack, fire and vandalism. With the exception of the last two categories these agencies can be divided into the two broad categories of construction induced damage and environmental attack.

3 SERVICEABILITY COMPLIANCE

Compliance with reasonable serviceability requirements does not appear to be problematical for selected polymer reinforcement. For example Carroll and Richardson (1986) report measured short term geogrid strains generally less than 0.6% and state that the tied back wedge analysis, as presented in BE3/78, significantly underestimates reinforcement strains. Since observed reinforcement strains are generally small it is reasonable to analyse these assuming the peak angle of shearing resistance of the fill to be mobilised. Using the techniques prescribed by Andrawes et al (1986) an allowance may be made for the effects of creep on calculated strains. However, any such calculations should make due allowance for the effects of temperature variations as described in later sections.

4 DETERMINATION OF DESIGN STRENGTH

The design strength of the reinforcement governed by tensile rupture must at all times during the design life of the structure be greater than or at least equal to the worst expected design force exerted by the fill and any superimposed loading. It is necessary to assess how the tensile rupture strength will decrease with time. This can be achieved by loading different samples of the reinforcement at different load intensities so that the times to failure fall inside a predetermined range of time at a standard test temperature of, say 20°C. Where the duration of the longest test is less than the required design life extrapolation of results can be facilitated by testing at higher temperatures, subject to certain limitations, to accelerate time.

This is the essence of the technique used by the British Gas Corporation to determine the 50 year design strength

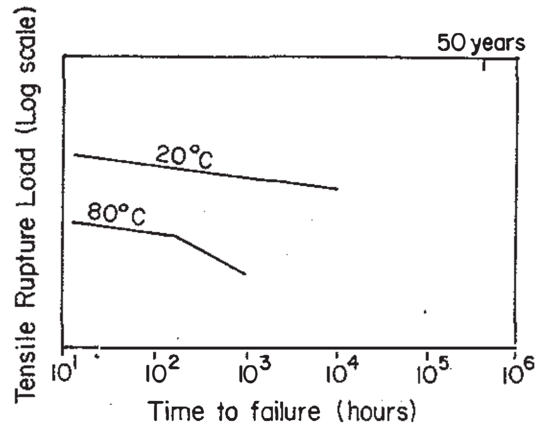

Figure 1. Raw test data for gas pipe

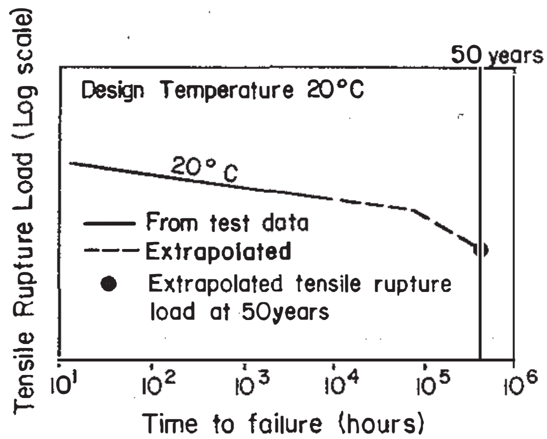

Figure 2. Extrapolation of test data for gas pipe.

of certain polymer gas pipes. Tests are run at temperatures of 20°C and 80°C with times to failure up to 10^4 hours (Greig 1976). Typical results are shown in Figure 1. The tests at the design temperature of 20°C show a log-log linear relationship between rupture load and time. The perils of extrapolating the 20°C test data to 50 years are reflected in the knee in the 80°C results which define a transition between ductile and brittle failure. By combining the test data at 20°C and 80°C it is possible to calculate the time, beyond the maximum test duration of 10^4 hours, at which any knee might develop.

As shown in Figure 2 these combined data are extrapolated to define a tensile rupture load at the end of the required 50 year design life.

This involves extrapolating over 1.64 log-cycles of time from 10^4 hours (1.14 years) to 50 years. Since this is in excess of the maximm extrapolation of one log-cycle of time prescribed in BS.4618:1970, a factor of safety of four is applied to the extrapolated rupture load to give the 50 year design load (Greig 1981).

Similar techniques may be applied to determine the long term tensile rupture strength of soil reinforcement, however, a very clear distinction must be drawn between undamaged control samples tested in a benign medium, such as air at constant temperature, and operational samples which will be damaged during construction and be subject to environmental attack through agencies such as water and chemicals or bacteria in the fill. For a product subject to strict quality control there should be little variation in the extrapolated 120 year characeristic strength from batch to batch. However, the degree of mechanical damage and aggressiveness of the fill will vary from fill to fill as might the operational temperature. It is vital that tensile rupture tests are carried out on operational samples to determine how mechanical damage and environment will reduce the long term rupture strength. This will allow the determination of various partial factors to be applied to the characteristic control strength to reduce it according to the nature of the particular fill to be employed. Ideally laboratory tests on operational samples, which have been pre-damaged, should be carried out in an aggressive environment since the combined effects of environmental and mechanical damage may be synergistic. This means that the combined effects of environmental and mechanical damage may be greater than the sum of the effects of testing damaged samples in a benign environment and undamaged samples in an aggressive environment.

To determine the 120 year design strength the following minimum partial factors are suggested. Those relating to the effects of mechanical damage and environment should be determined by exhaustive testing along the lines described above.

4.1 Material factor: γ_m

This relates to the probability that the control strength of the soil reinforcement may occasionally fall below the specified characteristic strength. The suggested value of γ_m is 1.2.

4.2 Test data extrapolation factor: γ_t

This relates to the decreasing degree of confidence in extrapolated data as extrapolation is made over increasing time intervals. No test data should be extrapolated at the design temperature without the aid of accelerated testing at appropriate higher temperatures. Ideally extrapolation should not exceed one cycle of common log-time, that is logarithmic time to the base 10. Extrapolation should never exceed two log-cycles of time. For n log-cycles of extrapolation where $1<n<2$ the suggested value of γ_t is 1.1n. Laboratory tests should be conducted at the design temperature which should equal the maximum operational temperature in the soil. For temperate climates a standard test temperature of 20°C should be adequate.

4.3 Construction induced damage factor: γ_c

This relates to the long term effect of mechanical damage suffered by the reinforcement during installation. Among other things it will be a function of fill type, layer thickness and type of compaction plant. The effects of mechanical damage should be assessed using long term tensile rupture tests such as those employed to assess the long term tensile rupture strength of intact and undamaged control samples. Short term constant rate of strain tests have indicated reduction factors in the range 1.1 to 1.6 for geogrids (Mitchell and Villet 1987). The minimum suggested value of γ_c is 1.2.

4.4 Environmental attack factor: γ_e

This relates to the long term affect of the fill environment on tensile rupture strength. Both chemical and bacteriological attack must be considered and their effects quantified by carrying out long term tensile rupture tests in an appropriate aggressive environment at the design temperature or higher temperatures as appropriate. The minimum suggested value of γ_e is 1.1.

4.5 Overall factor of safety: γ_r

The partial factors γ_m, γ_t, γ_c and γ_e are applied to the long term characteristic tensile rupture strength to reduce this to the basic design value. Where the ramifications of attaining the ultimate limit state of tensile rupture of the reinforcement are more serious the basic design design strength may be reduced by applying an overall factor of safety γ_r. The suggested minimum values of γ_r are in the range 1.0 to 1.2.

4.6 Design Strength

The design strength of the reinforcement for permanent structures is the 120 year characteristic tensile rupture strength, determined for intact control samples in a benign environment at the design temperature, divided by the partial factors γ_m, γ_t, γ_c, γ_e and γ_r. Minimum values of these factors have been suggested. Actual values of γ_c and γ_e due to construction damage and environmental effects are product and fill specific and must be determined directly by long term testing. As tensile rupture test data are gathered over longer test periods, the uncertainty of extrapolation decreases and therefore γ_t may be decreased as longer term data become available. Depending on the value of γ_r, the compounded minimum values suggested for the above partial factors varies from the range 1.7 to 2.1 for extrapolation through one log-cycle of time to the range 3.5 to 4.2 for extrapolation over two log cycles of time. Although on first sight the latter range of factors may appear severe they do relate to extrapolation of test data of 1.2 years duration by a factor of one hundred to a 120 years design strength. In comparison for extrapolation over 1.64 log-cycles of time the suggested partial factors result in a compounded factor in the range 2.9 to 3.4 which is significantly lower than the factor of 4 used by British Gas. It should be remembered that the above factors are suggested minimum values and these are likely to increase significantly for reinforcement susceptible to environmental attack or mechanical damage induced by the construction process.

5 OTHER FACTORS AFFECTING SAFETY

To obviate tensile rupture during the design life of the strucure the design strength of the reinforcement must never be less than the design force generated under the worst expected loading conditions. The design force will be dramatically affected by the angle of internal shearing resistance mobilised in the fill. The design strength of the reinforcement will be radically affected by operational temperatres in the fill and how these relate to laboratory test temperatures used to predict long term reinforcement design strength. For simple earth retaining structures the majority of the design force will be derived from the active thrust generated by the fill. This is the basis of design in Technical Memorandum BE3/78 where the coefficient of lateral earth pressure is calculated on the assumption that the peak angle of shearing resistance of the fill prevails.

5.1 Fill-reinforcement strain compatibility

The peak angle of shearing resistance is mobilised at small lateral strains which under plane-strain loading would be of the order of 1%. Such small lateral strains may be as much as an order of magnitude smaller than the axial tensile strains required to generate the long term rupture strength of the reinforcement. Consequently there is incompatibility between fill and reinforcement strain. To overcome this the design force for extensible polymeric reinforcement should be based on the constant volume angle of shearing resistance of the fill, ϕ_{cv}. This is smaller than the peak value, however, it is a value which can be relied on even at large strain. For a particular frictional fill with a maximum particle size of 40mm Brady (1987) measured a peak angle of shearing resistance of 61° compared to a constant volume value of 41°. Based on active earth pressure analysis the thrust developed at ϕ_{cv} is in excess of three times greater than that developed at the peak angle of ϕ shearing resistance. This large difference endorses the need to design on the basis of ϕ_{cv} when guarding against tensile failure of the reinforcement.

5.2 Operational and test temperature compatibility

The performance of thermo-plastic polymers will be radically affected by temperature. For this reason the laboratory test temperature should equal or exceed the maximum operational temperature in the reinforced soil structure. To aid extrapolation of labortory tests data, tests may be carried out at elevated temperatures provided these do not change the basic mechanisms controlling the behaviour of the reinforcement. The concept of extrapolation using time-temperature transposition has been reported by Andrawes et al (1986) who suggest an acceleration in time by a factor of 10 by raising the test temperature from 10°C to 20°C. This behaviour would be governed by an Arrhenian relationship of the form given in Equation 1.

$$\ln(\mu) = \alpha(1/\theta_2 - 1/\theta_1) \qquad \ldots 1.$$

where:-

μ = a time multiplier

α = a constant (polymer and product specific)

θ = temperature in degrees Kelvin.

Taking a time multiplier of 10 for a temperature shift between 10°C and 20°C allows evaluation of the constant α. Equation (1) can then be used to assess the relationship between operational temperature θ_1, laboratory test temperature θ_2, and the time multiplier μ. If the operational soil temperature is constant and equal to the laboratory test temperature the time multiplier is unity.

For permanent highway structures the creep properties of polymeric soil reinforcement are required to be determined at 10°C (DTp 1987). Recent work on the measurement of operational temperatures in a reinforced soil wall confirms 10°C as a reasonable mean soil temperature but shows that the seasonal variation, especially near the face of the wall is approximatel ±10°C (Murray and Farrar 1988). The seasonal variation was approximately sinusoidal with a maximum temperature of some 20°C minimum temperature close to 0°C.

Taking this sinusoidal variation Equation (1) can be used to determine the seasonal variation and mean value of the time multiplier μ. The results are shown in Figure 3. Due to the asymmetrical distribution of the seasonal time multiplier about the mean operational temperature the mean value is 2.85. The implication of this is that predictions made from laboratory tests run at the mean operational temperature of 10°C will underestimate performance since "effective" time under operational conditions is running 2.85 times faster than "real" time. This implies that a 120 year service life under the operational temperatures shown in Figure 3 is equivalent to 340 years, i.e., 2.85 x 120 years, at a constant temperature of 10°C. Consequently an extrapolation of test data at 10°C from say 1.2 years to 120 years must be extended to 340 years. This may increase extrapolation from 2 log-cycles of time to 2.5 log-cycles of time in which case γ_t would increase from 2.2 to 2.8. Conversely if the laborator test temperature was 20°C, i.e., equal to the maximum operational temperature in Figure 3, then a degree of conservatism is introduced into the extrapolation.

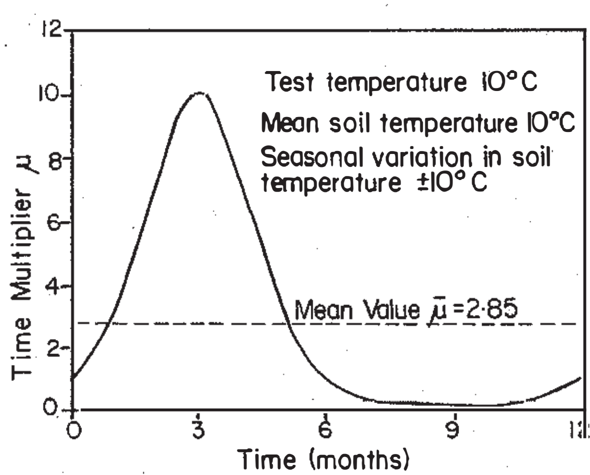

Figure 3. Seasonal variation of time multiplier

Care should be taken to ensure that laboratory test temperatures and operational temperatures are compatible. Consideration should extend to the variations of diurnal temperatures in the close vicinity of preformed facing units and any spontaneous heating in fill containing industrial waste. For example West and O'Reilly (1986) comment on heating in unburnt colliery shale and relate reductions in strengths of plastic reinforcing elements of 30% for temperature increase of 10°C above a 20°C ambient temperature. Riskof fire should also be considered.

6 CONCLUSIONS

The Department of Transport Technical Memorandum BE3/78 sets down broad requirements for the long term performance of polymeric reinforcement. Assessments of proprietary reinforcing materials are made by the British Board of Agrement who issue product specific Roads and Bridges Certificates and so confer compliance with the requirements of the Memorandum. A broad approach to design has been presented and this suggests minimum values of partial factors which might be employed to ensure an adequate margin of safety against tensile rupture of the reinforcement.

REFERENCES

Andrawes, K.Z., McGown, A. and Murray, R.T. 1986. "The load-strain-time-temperature behaviour of geotextiles and geogrids". Proc.III Int.Conf. on Geotextiles, Vienna, Vol.3.

Brady, K.C. 1977. "Performance of a reinforced earth bridge abutment at Carmarthen". Transport and Road Research Laboratory Research Report 111.

BS.4618 : 1970. "The presentation of plastics design data : Subsection 1.1.1 Creep in uniaxial tension or compression". British Standards Institution.

Carroll, R.G. and Richardson, G.N. 1986. "Geosynthetic reinforcd retaining walls". Proc.III Int.Conf. on Geotextiles, Vienna, Vol.2.

Department of Transport 1987. "Reinforced earth retaining walls and bridge abutments for embankments". Technical Memorandum (Bridges) BE3/78, (Revised 1987).

Greig, J.M. 1981. "Specification and testing of polyethylene gas distribution systems for a minimum 50 year operational life". Plastics and Polymer Pocessing and Applications, Vol 1, No.1.

Greig, J.M. 1976. "Fracture and its prevention in plastic gas distribution systems". Gas Engineering and Management, Vol.16, No.2.

Mitchell, J.K. and Villet, W.C.B. 1987. "Reinforcement of earth slopes and embankments". National Co-operative Highway Research Program Report 290. Transportation Research Board, Washington D.C.

Murray, R.T. and Farrar, D.M. 1988. "Temperature distributions in reinforced soil retaining walls". Int.Journal of Geotextiles and Geomembranes, Vol.6.

West, G. and O'Reilly, M.P. 1986. "An evaluation of unburnt colliery shale as fill for reinforced earth structures". Transport and Road Research Laboratory, Research Report 97.