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Fatigue Limit Load in Triaxial Tests of Geotextile-Reinforced Soil

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ABSTRACT: Triaxial tests with static and cyclic loading were performed on geotextile-reinforced clayey silt. The test results were analyzed using the Fatigue Limit Load method initially proposed for modeling the fatigue behavior of reinforced unpaved roads where tensile membrane action is the main mechanism. In this paper, it is shown that the concept of Fatigue Limit Load is also applicable to the triaxial tests results where the reinforcement mechanism is enhanced confinement.

1 INTRODUCTION

Triaxial tests with monotonic and cyclic loading were performed on geotextile-reinforced partially saturated clayey silt (Ashmawy, 1995, Ashmawy and Bourdeau, 1997, Ashmawy et al., 1999) as part of an extensive investigation, initiated by the Swiss Federal Highway Administration, which also included full-scale tests on geotextile-reinforced retaining walls (Kharchafi and Dysli, 1994). As part of the study, triaxial tests on reinforced sand (Ashmawy and Bourdeau, 1998) and cyclic tension tests on the geotextiles (Ashmawy and Bourdeau, 1996) were also conducted. The triaxial tests indicated that the inclusion of equally-spaced geotextile layers within the soil specimens results in a significant reduction in cumulative permanent strain. The density of reinforcement played a major role in terms of the amount of improvement introduced. Two methods were presented in earlier publications to analyze the response of the geotextile-reinforced soil under axisymmetric cyclic loading (Ashmawy et al., 1999). The first method accounts for the change in stress level caused by additional confinement due to the lateral restraint action of the geotextile. The second method takes into account, in addition to the stress change, the reinforcement-induced changes in strength and deformation modulus. Both analysis methods are based on an equivalent pseudo-confinement concept and were used effectively in interpreting the increase in strength and cyclic modulus of the triaxial reinforced soil specimens. In the present paper, a third way to analyzing these results, the Fatigue Limit Load (or Fatigue Limit Stress) method, is proposed.

2 TESTING PROGRAM

Detailed information on the testing program has been reported in earlier publications (Ashmawy et al., 1996, 1999). Only a short summary is provided herein.

The soil used for these tests was a natural glacial till (Crosby Till), classified as clayey silt (CL-ML), with a plastic limit of 15.5% and plasticity index of 4%. The Standard Proctor optimum water content and maximum dry unit weight were 11.5% and 19 kN/m^3, respectively. This soil is cohesionless and its angle of internal friction is of the order of 37°.

Two types of geotextiles, a woven and a nonwoven, were used as reinforcement. Their tensile strength (ASTM D 4595) was 220 kN/m (woven) and 20 kN/m (nonwoven). The tensile modulus at 10% strain (ASTM D 4595) was 2000 kN/m (woven) and 120 kN/m (nonwoven). The soil-geotextile interface shear strength was tested in direct shear tests. With the woven textile, the interface friction angle was found equal to the soil angle of internal friction (37°). It was slightly lesser with the nonwoven fabric (33°). Cylindrical specimens (71mm in diameter and 170mm high) including equally spaced horizontal geotextile disks were prepared (Figure 1).

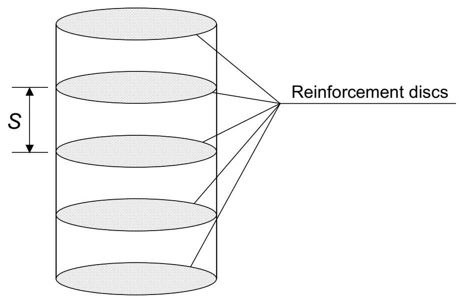

Figure 1. Soil specimen reinforced with geotextile disks

For each type of geotextile, reinforced specimens were prepared with soil compacted "dry of optimum" and "wet of optimum". The number of reinforcement disks per specimen was also variable, from 4 to 6. Unconsolidated undrained monotonic triaxial tests (ASTM D 2850) were conducted on the reinforced soil specimens as well as unreinforced Crosby till. The main part of the testing program consisted of unconsolidated undrained cyclic triaxial tests at a load frequency of 0.5Hz and under stress-controlled conditions. All these cyclic tests were performed at 50kPa confining pressure. Nominal axial deviator stress amplitudes of 45, 60 and 80 kPa were applied, corresponding to 67%, 90% and 120% stress levels. The stress level is defined here as the ratio of the axial deviator stress amplitude to the maximum deviator strength of the unreinforced soil at the same confining pressure. For each test, the cumulative plastic deformations were recorded in function of the number of cycles.

3 FATIGUE LIMIT LOAD METHOD

3.1 *Methodology*

The concept of Fatigue Limit Stress was introduced in the mid 1980's as a methodology for analyzing the fatigue behavior of geotextile-reinforced unpaved roads. The contribution resulted from a broad team investigation, conducted in France in the early 1980s, which included analytical modeling, small-scale experiments and full-scale tests. A summary of the overall project was published by Delmas et al, 1986. Although the original application problem was a two-layer system reinforced at the interface and subjected to concentrated surface load, the present study suggests the method can be extended to the case of soil specimens reinforced by multiple layers and subjected to uniform boundary loads.

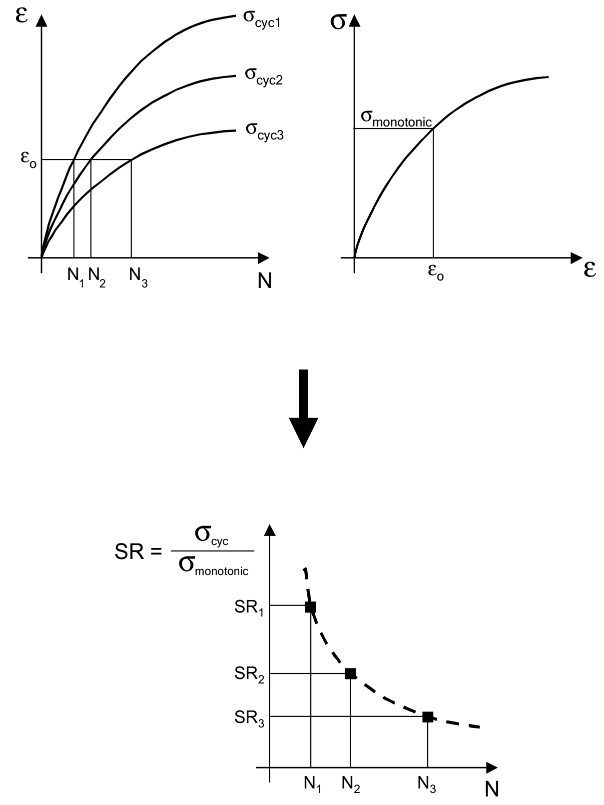

Figure 2. Data analysis using the Fatigue Limit load method

The methodology is based on two important ideas: (a) For a given reinforced soil system, there is a limit to the cyclic load under which the permanent cumulative deformation would remain finite for an infinite number of cycles. This threshold value is the Fatigue Limit load. (b) The ratio of the cyclic load amplitude and of the monotonic load required to cause the same deformation can be related experimentally to the number of cycles. This relationship is a characteristic of a particular reinforced soil system's materials, geometry and loading mode. In the case of triaxial tests considered herein, SR, the ratio of cyclic to monotonic stress corresponding to the same strain level is likely to depend on such factors as: (1) inherent and state parameters of the soil, (2) properties of reinforcement and soil/reinforcement interfaces, (3) strain level, (4) number of load cycles, (5) initial stress state in the reinforced soil mass, (6) stress paths (monotonic and cyclic), (7) spacing between reinforcement layers, (8) amplitude of cyclic stress (or its ratio to the maximum monotonic strength).

For a given soil/reinforcement combination, factors (1) and (2) are fixed, and SR will only be a function of factors (3) through (8). The strain level at which SR is determined and the number of cycles are not independent parameters since higher strains correspond to higher number of cycles. Therefore, factor (3) can be eliminated from the list. For the tests performed in this study, the initial stress state and the stress path of the reinforced soil system are constant, therefore limiting the amount of information available to examine the influence of factors (5) and (6). Consequently, SR can be expressed as:

$$SR = f\,(N, \Delta s, \sigma_{cyc}) \tag{1}$$

where N is the number of cycles, Δs is the reinforcement spacing, and σ_{cyc} is the cyclic stress amplitude.

Examination of the data showed that, for these particular tests, SR is fairly independent of the spacing between reinforcement layers and the cyclic stress amplitude. Therefore, for a given soil/geotextile combination, SR can be simply expressed as a function of only the number of cycles. The data can be reduced in a simple manner, as illustrated in Figure 2, in order to plot SR vs N. In the Figure, three data points are obtained for a specific strain level. Both monotonic and cyclic tests are performed at a given reinforcement spacing, confining pressure (50 kPa), and stress path (triaxial compression). This procedure is repeated for other pairs of monotonic-cyclic tests with different reinforcement spacings, but for the same soil and geotextile type. Ideally, a plot such as that shown on Figure 2 is generated.

3.2 *Model Formulation*

Curve fitting was carried out to find the relationship between SR and N. For this purpose, various forms of rational functions were considered, with two main criteria in mind: first, the parameters have to be dimensionless and to have, as much as possible, a physical significance; and second, the function has to satisfy the limit conditions, such as having a finite value at ½ cycle (monotonic case) and become asymptotic at an infinite number of cycles.

Because the number of cycles may vary in terms of orders of magnitude within a single cyclic test, it is necessary to use a log scale for the abscissa (N) of the SR vs N plot. Among the various candidate fitting functions, the basic form ($1/x^n$), where n is a positive number, was chosen because of its simplicity as well as the high correlation coefficients it provided with respect to the data available. The general form of the function is expressed as:

$$SR = \frac{1}{\left\{ a + b\,[\log_{10}(N + 0.5)]\,\right\}^{d}} + c \tag{2}$$

where a, b, c, and d are regression constants or parameters. The value of (c) represents the asymptote of SR as the number of cycles approaches infinity. In other words, if a cyclic stress amplitude equal to ($c \times \sigma_{monotonic}$) is applied, the cumulative plastic strain will never exceed the corresponding monotonic strain. The value of (a) is such that the stress ratio at ½ cycle (monotonic case) is equal to ($c+a^{-d}$). Ideally, this value should be 1, provided that the rate of loading for both monotonic and cyclic tests is the same. The parameter (b) indicates the relative "lateral spread" of the SR vs N plot, and the parameter (d) is a measure of the curvature.

Preliminary analysis showed that (d) is almost equal to 2 for most cases, and that the coefficient of correlation is fairly insensitive to its value. Furthermore, when (d) is restricted to a value of 2, the regression values of the other parameters (a, b, and c) are more consistent with their physical interpretation (Table 1). Therefore, Equation (2) can be simplified to:

$$SR = \frac{1}{\left\{ a + b\,[\log_{10}(N + 0.5)]\,\right\}^{2}} + c \tag{3}$$

The test results for each combination of Crosby till soil (wet and dry of optimum) and reinforcement (non-woven geotextile, woven geotextile), as well as unreinforced soil, were plotted, and curve fitting was carried out using a commercially available software. The test data together with the fitted function are plotted on Figures 3 through 8.

The coefficients of correlation, for the whole set of data, ranged between 0.82 and 0.97. From the Figures it can be seen that the value of SR corresponding to ½ cycle is not equal to 1 as should be expected, but is in fact higher in all the cases. This indicates that the frequency and rate of loading play important roles in the response of the tested materials.

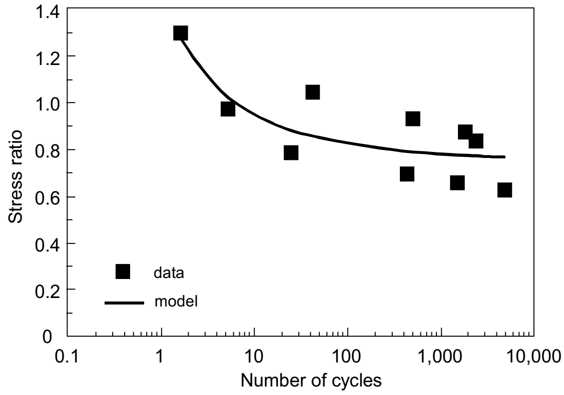

Figure 3. Fatigue limit load data and model for unreinforced Crosby till at 10% water content (dry of optimum)

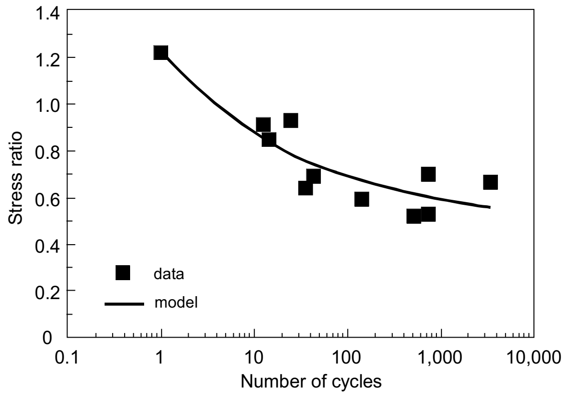

Figure 4. Fatigue limit load data and model for non-woven reinforced Crosby till at 10% water content (dry of optimum)

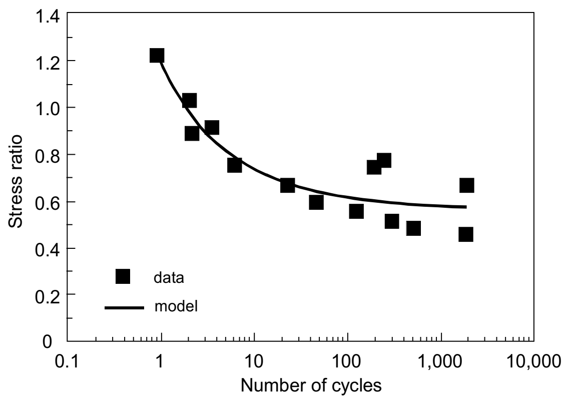

Figure 5. Fatigue limit load data and model for woven reinforced Crosby till at 10% water content (dry of optimum)

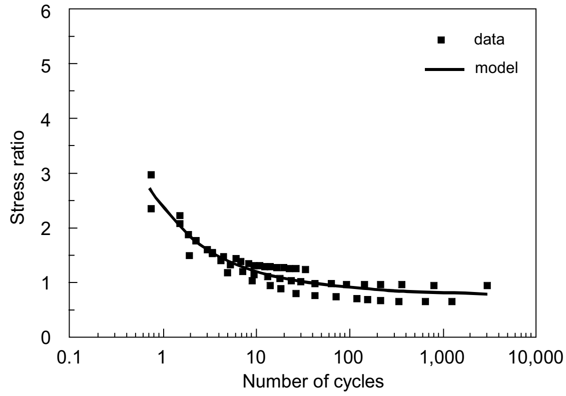

Figure 6. Fatigue limit load data and model for unreinforced Crosby till at 13.5% water content (wet of optimum)

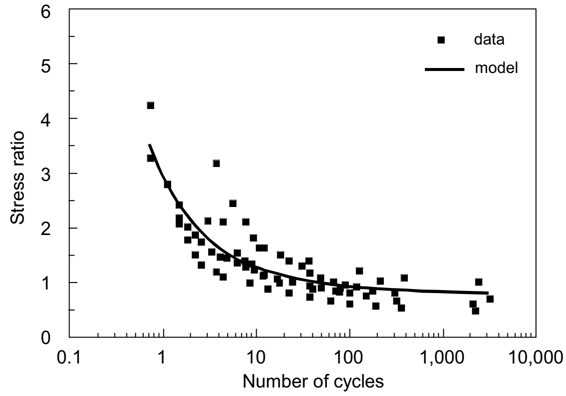

Figure 7. Fatigue limit load data and model for non-woven reinforced Crosby till at 13.5% water content (wet of optimum)

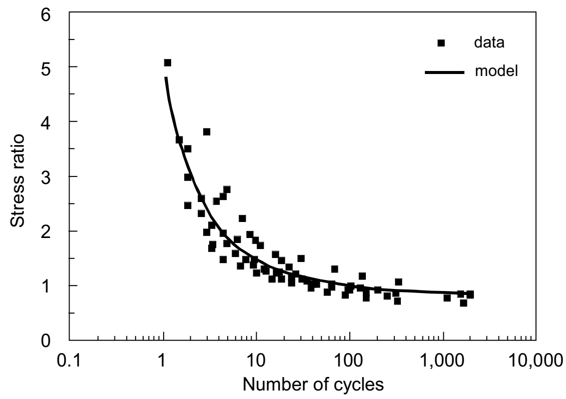

Figure 8. Fatigue limit load data for woven reinforced Crosby till at 13.5% water content (wet of optimum)

The stress ratio of 1 is only reached after roughly 2 to 4 cycles for the dry-of-optimum soil (Figures 3 through 6), and up to 10 cycles for the wet-of-optimum soil (Figures 6 through 8). Had the cyclic tests been performed at the same loading frequency as the monotonic tests, an SR value of 1 would have corresponded to an N value of ½.

The function expressed by Equation (3) may be modified if further experimental evidence shows a dependency of SR on other factors. For instance, if SR varies linearly with the confining pressure, given that all other factors are constant, and that there is no coupling between the effects of confining stress and the number of cycles, Equation (3) can be re-written as:

$$SR = \left[\frac{1}{\left\{ a + b \left[\log_{10}(N + 0.5) \right] \right\}^2} + c \right] . \left[\alpha + \beta \sigma_c \right] \quad (4)$$

where α and β are additional regression parameters and σ_c is the confining pressure. At the present time, given the lack of data, we will assume that SR is independent of the confining pressure.

Water content (%)	Reinforcement type	a	b	c	Correlation coefficient
10	None	0.973	1.10	0.723	0.82
10	Non-woven	1.02	0.429	0.402	0.90
10	Woven	1.03	1.15	0.523	0.91
13.5	None	0.644	0.726	0.668	0.94
13.5	Non-woven	0.527	0.776	0.697	0.87
13.5	Woven	0.337	0.794	0.703	0.92

Table 1. Experimental values of regression coefficients in Eq. 3

4 DISCUSSION AND CONCLUSION

The analysis of triaxial tests results performed on clayey silt reinforced with multiple disks of geotextile shows that the Fatigue Limit Load concept is applicable to this type of reinforced soil configuration. For the materials and load conditions tested, there is experimental evidence that a limit stress ratio exists, for which the SR vs N curve has a finite asymptotic value when the number of cycles becomes very large. The Fatigue Limit load and the relationship between the Stress Ratio and the number of load cycles are characteristics of the soil/geotextile system. The Fatigue Limit Load concept is also applicable to the unreinforced soil tested under the same conditions. These findings suggest the Fatigue Limit Load methodology can be used to analyze reinforced soil systems where the dominant mechanism is the enhanced confinement provided by the geotextile interaction with the soil.

The Fatigue Limit Load method has the advantage of explicitly accounting for the number of cycles. For a given soil-geotextile system, the model parameters seem to be independent of the reinforcement spacing. If more data were available to confirm this tendency, this would allow for the extension of small-scale results to large-scale applications. For instance, the stress state in an embankment reinforced with horizontal geosynthetic layers being similar to that of a triaxial specimen reinforced with horizontal strips, parameters developed based on small-scale triaxial tests would be applicable to the full-scale problem.

For a given applied cyclic load (or stress) and number of cycles, the Fatigue Limit equation allows for estimation of an equivalent monotonic load (or stress) which will produce the same strain. Monotonic tests or numerical simulations can then be performed using the equivalent monotonic load to predict the performance of the system under cyclic loading.

It should be, however, emphasized that prediction behavior for reinforced soil masses under cyclic loading is not an easy task. Even when advanced modeling techniques are used, it must be expected that high scatter in the prediction will always be present. Under cyclic loading conditions, the evolution of strains as a function of number of cycles is highly influenced by small initial variations in material properties. For instance, if high strains develop in the first few cycles of loading, the modulus of the soil degrades even faster during subsequent cycles. If the material withstands the first few cycles without undergoing high deformations, it is more likely that the rate of accumulation of permanent strains will be lower. Relatively high scatter is already observed for geotextile, a material that is presumably subject to a systematic manufacturing process and high quality control. The behavior of the same material under monotonic loading is very consistent in terms of stress-strain behavior, although the strain at failure is somewhat scattered. Properties of reconstituted soil specimens are subjected to even greater scattering, and the behavior of "identically" prepared specimens in the laboratory depends on various factors such as soil fabric, void ratio distribution and initial water content. The variation of such parameters in the field for engineered reinforced fills is expected to be even larger. It should not, therefore, be expected that accurate and reliable estimates of reinforced soil deformability or strength can be easily made without accounting for these sources of uncertainty.

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