Technical Paper by A. Sawicki and W. Świdziński

UNCONFINED VERSUS CONFINED TESTING OF GEOSYNTHETICS

ABSTRACT: A theoretical explanation of the confined in-soil behaviour of geosynthetics is presented and compared to standard unconfined load-extension and creep test results. First, it is shown that confined, load-instantaneous geosynthetic extension problems can be successfully described by a pull-out model of a single geosynthetic reinforcement strip in soil. The confined in-soil stiffness of a geosynthetic is derived analytically as a function of the unconfined stiffness and the stiffness of the soil-geosynthetic strip interface. Second, the problem of confined in-soil creep is analysed on the basis of a similar model that takes into account the rheological properties of geosynthetics. Generally, it is shown that the confined in-soil behaviour of geosynthetics can be predicted on the basis of the unconfined properties of geosynthetics and the soil-geosynthetic strip interface stiffness. Practical examples show good agreement between the proposed models and the experimental data.

KEYWORDS: Geosynthetics, Load-Extension, Creep, Unconfined tests, Confined in-soil test.

AUTHORS: A. Sawicki, Professor, W. Świdziński, Assistant Professor, Institute of Hydro-Engineering of the Polish Academy of Sciences, IBW PAN, ul. Koscierska 7, 80-953 Gdañsk-Oliwa, Poland, Telephone: 48/58-552-20-11, Telefax 48/58-552-42-11, E-mail: as@ibwpan.gda.pl.

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1 INTRODUCTION

The load-extension characteristics and creep properties of geosynthetics are usually determined using unconfined tests performed on specimens that are isolated from the soil. The results of such tests are presented in the form of load-instantaneous extension curves and creep curves, which illustrate the strain, ε, as function of time, t, and the applied load or stress, F.

To obtain a better understanding of the effects of soil confinement on geosynthetic creep, confined in-soil creep tests were performed by McGown et al. (1982) and Matichard et al. (1990). According to Ingold (1994) and Leshchinsky et al. (1997), the results reported by McGown et al. (1982) and Matichard et al. (1990) are somewhat inconclusive. For example, McGown et al. (1982) showed the extreme effect of soil confinement on the creep of geotextiles, while Matichard et al. (1990) demonstrated that soil confinement occasionally has an insignificant effect on geotextile creep. In the current paper, it is shown that these very different conclusions can be explained using the single geosynthetic reinforcement strip/layer model proposed by Sawicki (1998).

First, the problem of confined, load-instantaneous geosynthetic extension is analysed. The problem is described using the second order differential equation presented by Sawicki (1998), which is then solved for the respective boundary conditions. The solution is presented in the form of a load-strain relationship that depends on the stiffness coefficient G that describes the soil-geosynthetic interface properties. The value of G is estimated using empirical data provided by McGown et al. (1982), then an empirical formula relating G to the confining stress, σ_{conf}, is proposed. Numerical examples are presented that show an agreement between the theoretical prediction and experimental results presented by McGown et al. (1982).

Second, confined geosynthetic creep tests are analysed. The starting point is the third order differential equation presented by Sawicki (1998), which is solved numerically for the respective initial and boundary conditions, and for the standard rheological model of a geosynthetic. The numerical example deals with experimental data presented by McGown et al. (1982). The parameters of the geosynthetic rheological model are determined using an unconfined creep test. The predicted results of a confined creep test are presented and compared with empirical data; and, again, a good agreement is obtained.

2 THEORETICAL CONFINED IN-SOIL LOAD-EXTENSION RELATIONSHIP

Figure 1 shows the static scheme of a geosynthetic tested in confined conditions. The distribution of tensile forces, F, along the geosynthetic is given by the following differential equation (Sawicki 1998):

$$\frac{\mathrm{d}^2 F}{\mathrm{d}x^2} - \alpha^2 F = 0 \tag{1}$$

with:

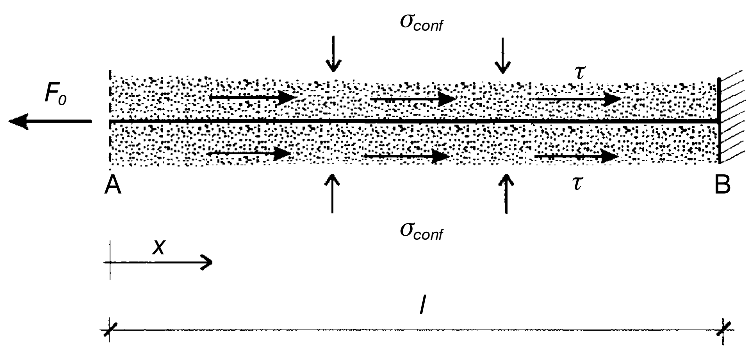

Figure 1. Confined in-soil extension of a geosynthetic strip.

$$\alpha = \sqrt{\frac{2BG}{E}} \tag{2}$$

where: B = width of the geosynthetic strip; E = elastic, instantaneous stiffness of the geosynthetic strip; and G = stiffness coefficient describing the soil-geosynthetic strip interface properties.

The following boundary conditions correspond to the static scheme shown in Figure 1:

$$F(x = 0) = F_0 \qquad \frac{dF}{dx}(x = l) = 0 \tag{3}$$

where: l = length of geosynthetic strip; and F_0 = pull-out force.

The solution of the boundary-value problem defined by Equations 1 and 3 is as follows:

$$F = \frac{F_0}{1 + \exp(-2\alpha l)} [\exp(-\alpha x) + \exp(-2\alpha l)\exp(\alpha x)] \tag{4}$$

The shear stress, τ, at the soil-geosynthetic strip interface can be calculated using the following relationship:

$$\tau = -\frac{1}{2B}\frac{dF}{dx} \tag{5}$$

and the displacement, u :

$$u = -\frac{1}{G}\tau \tag{6}$$

The average geosynthetic specimen strain, measured at $x = 0$, ε_0 , is defined as:

$$\varepsilon_0 = -\frac{u\ (x = 0)}{l} \tag{7}$$

Equations 4 to 7 lead to the following relationship between the geosynthetic pull-out force, F_0 , and the average strain:

$$F_0 = E_{conf}\ \varepsilon_0 \tag{8}$$

where:

$$E_{conf} = \alpha E l\ \frac{[1 + \exp(-\ 2\alpha l\ )]}{[1 - \exp(-\ 2\alpha l\ )]} \tag{9}$$

is the confined stiffness of the geosynthetic specimen. Note that the stiffness depends on the value of α, which in turn is dependent on the value of the stiffness coefficient G (Equation 2).

3 DETERMINATION OF STIFFNESS COEFFICIENT, *G*

A simple method for calculating the value of the stiffness coefficient G was presented by Sawicki (1998); however, this method was for boundary conditions corresponding to a pull-out test, where one end of the geosynthetic specimen was stress free. For the case where one end of the geosynthetic specimen is fixed (see point B in Figure 1), the stiffness coefficient G should be determined numerically using Equation 9 provided that the remaining equation parameters are known.

Example 1. Determine the value of G for Geotextile 2 (Figure 2b) from a pull-out test in Leighton Buzzard sand (McGown et al. 1982, Figure 8b).

The geotextile specimen, with length, $l = 0.1$ m and width, $B = 0.2$ m, was tested in both unconfined and confined (in-soil) conditions. In both cases, the axial strain-load characteristics are approximately linear. The axial load, $F_0 = 1{,}900 \times 0.2 = 380$ N. The geotextile stiffness determined from the unconfined test is $E = 3{,}800$ N. The confined stiffness, $E_{conf} = 10{,}857$ N. According to Equations 2 and 9, the stiffness coefficient $G = 7.65 \times 10^6$ N/m^3.

END OF EXAMPLE 1

Example 2. Determine the value of G for Geotextile 1 (Figure 2a) from a pull-out test in Leighton Buzzard sand (McGown et al. 1982, Figure 8a).

The values of the unconfined and confined stiffnesses of the geotextile specimen are determined using the same method shown in Example 1, by assuming the axial strain-load characteristics are linear. Thus, $E = 7{,}000$ N and $E_{conf} = 11{,}111$ N, and Equation 9 gives $G = 3.4 \times 10^6$ N/m^3. Note that the value of G obtained is smaller than the value of G corresponding to Geotextile 2 (see Example 1) although both geotextiles were tested in

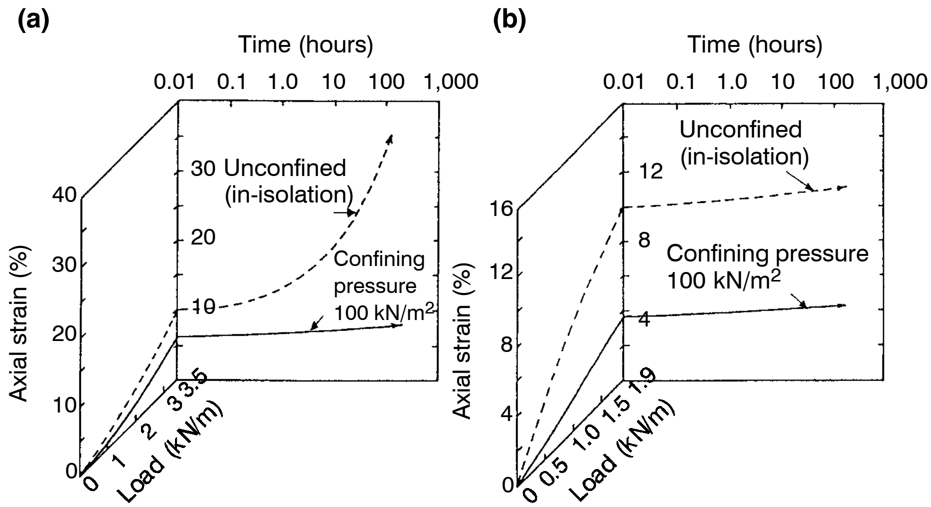

Figure 2. Creep test data (McGown et al. 1982): (a) Geotextile 1; (b) Geotextile 2.

similar conditions. The reason for this difference is likely due to the different structure of both geotextiles and, therefore, the different conditions at the soil-geotextile interface.

END OF EXAMPLE 2

Example 3. Determine the value of G for Geotextile 2 as a function of the confining stress (McGown et al. 1982) (Figure 3).

Figure 3 shows the axial load-strain curves for Geotextile 2 tested at various confining stresses. These curves are approximately linear up to a strain of 10% (see the curves for the 200 mm × 100 mm specimens). The results shown in Figure 3 are not strictly compatible with the results presented in Figure 2b; these differences will not be discussed in the current paper. From the unconfined test data, it follows that E = 3,680 N. From the confined test data, the following confined stiffness values were obtained:

$$E_{conf} = 4,930 \text{ N} \quad \text{for} \quad \sigma_{conf} = 10 \text{ kN/m}^2$$
$$E_{conf} = 7,080 \text{ N} \quad \text{for} \quad \sigma_{conf} = 55 \text{ kN/m}^2$$
$$E_{conf} = 8,620 \text{ N} \quad \text{for} \quad \sigma_{conf} = 100 \text{ kN/m}^2$$

where σ_{conf} is the confining stress (Figure 3), and the respective stiffness values are G = 1 × 10^6, 3.07 × 10^6, and 4.85 × 10^6 N/m^3.

It can be shown, by substituting the appropriate σ_{conf} values above, that the following empirical equation approximates the experimental data of McGown et al. (1982):

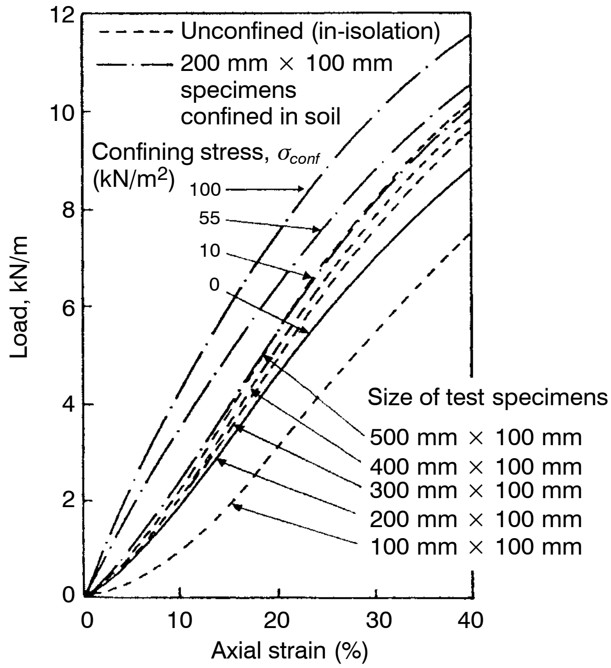

Figure 3. Unconfined (in-isolation) and confined in-soil load-axial strain data for Geotextile 2 (after McGown et al. 1982).

$$G = G_0 \, (\sigma_{conf})^n \tag{10}$$

where: n = 0.712; and coefficient $G_0 = 1.326 \times 10^3$. Note that the units of G_0 (N^{1-n} $m^{-(3-2n)}$) depend on the value of n. These values were determined using the least squares method. The correlation between the approximation, given by Equation 10, with experimental data from McGown et al. (1982) is shown in Figure 4. Equation 10 has a complicated form, therefore, for practical reasons, it can be replaced by a less accurate but much simpler linear relationship with n = 1 and the resulting value of G_0 = 55.82 m^{-1}.

END OF EXAMPLE 3

4 UNCONFINED CREEP OF GEOSYNTHETICS

The problem of unconfined creep of geosynthetics is relatively well developed in the literature when compared to confined in-soil testing. It was shown by Sawicki and Kazimierowicz-Frankowska (1998) that the unconfined creep of some geosynthetics at constant load can be approximated using a standard visco-elastic model , described by the following creep function, $\phi(t)$:

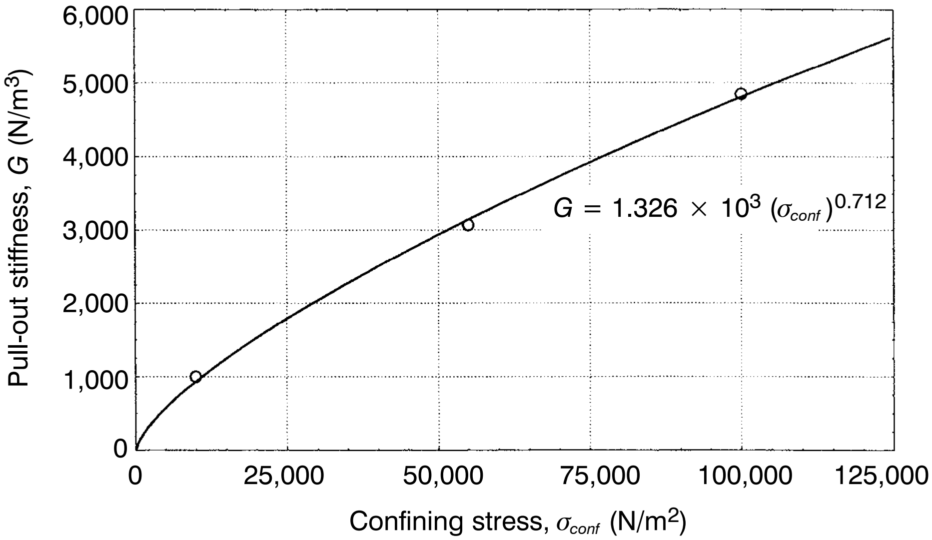

Figure 4. Approximation of stiffness *G*; confining stress relationship using experimental data from McGown et al. (1982).

$$\frac{\varepsilon}{F} = \phi(t) = \frac{1}{E^*} - \frac{1}{E_2} \exp\left(-\frac{E_2}{\eta} t\right) \tag{11}$$

$$E^* = \frac{E_1 E_2}{E_1 + E_2} \tag{12}$$

where: ε = geosynthetic strain; E_1 and E_2 = elastic stiffnesses of the rheological model; E^* = coefficient; t = time; and η = viscosity. The standard rheological model defined by Equation 11 can also be applied to the experimental results presented by McGown et al. (1982).

Example 4. Determine the creep parameters for Geotextile 2 (Figure 2b) (McGown et al. 1982).

The duration of the unconfined creep test was brief (approximately 100 hours), thus, it was assumed that after this time the geotextile creep ceases. This assumption can be used for any other calculated experimental data and respective creep parameters (Sawicki and Kazimierowicz-Frankowska 1998). The following parameters approximate the geotextile creep: $E_1 = 0.19 \times 10^5$ N/m, $E_2 = 1 \times 10^5$ N/m, and $\eta = 1.3 \times 10^6$ Nh/m.

END OF EXAMPLE 4

5 CONFINED IN-SOIL CREEP OF GEOSYNTHETICS

The confined in-soil creep of geosynthetics is described by the third-order differential equation derived by Sawicki (1998):

$$\frac{\partial}{\partial t}\left[\frac{\partial^2 F}{\partial x^2} - b_1 F\right] + a_1 \frac{\partial^2 F}{\partial x^2} - c_1 F = 0 \qquad (13)$$

where:

$$a_1 = \frac{E_2}{\eta}$$

$$b_1 = \frac{2BG}{E_1}$$

$$c_1 = \frac{2BG\,(E_1 + E_2)}{\eta\, E_1}$$

Equation 13 should be solved numerically with the boundary conditions given by Equation 3 and the initial condition from the instantaneous response of the geosynthetic (i.e. the instantaneous, elastic response with stiffness E_1).

Example 5. Determine the confined in-soil creep behaviour of Geotextile 2 (Figure 2b) (McGown et al. 1982).

The unconfined creep parameters were calculated in Example 4. The soil-geosynthetic interface stiffness, determined from Equation 9, is $G = 7.65 \times 10^6$ N/m^3. The remaining required values are: $F_0 = 1.9$ kN/m, $B = 1$ m, and $l = 0.1$ m. The average strains (Equation 7) were determined from Equation 13 with the boundary conditions calculated using Equation 3, and from Equations 5 and 6. The strain values at chosen time intervals are: $\varepsilon(t = 0) = 0.036$, $\varepsilon(t = 1 \text{ hour}) = 0.0362$; $\varepsilon(t = 10 \text{ hours}) = 0.0376$; and $\varepsilon(t = 100 \text{ hours}) = 0.0393$. Note that these values are very similar in magnitude to those experimentally determined by McGown et al. (1982).

END OF EXAMPLE 5

Example 6. Analyse the influence of the soil-geosynthetic interface stiffness, G, on the confined in-soil creep behaviour of a hypothetical geosynthetic. The following values are assumed: $B = 1$ m; $l = 1$ m; $F_0 = 5000$ N/m; $E_1 = 1 \times 10^6$ N/m; $E_2 = 1 \times 10^5$ N/m; $\eta = 1 \times 10^7$ Nh/m; and $G = 1 \times 10^5$, 1×10^6, and 1×10^7 N/m^3.

The tensile force distributions along the hypothetical geosynthetic were determined using Equation 13 with boundary conditions given by Equation 3 and the initial condition given by Equation 4, where the parameter E in Equation 2 was replaced by E_1. Note that, in the case of geosynthetic sheets, $B = 1$ m and, therefore, the value B in Equation 2 can be ignored and the stiffness, E, expressed in N/m. However, for consistency, because Equation 2 should apply for both strip and sheet reinforcement, the notation presented in Equation 2 should be used. Equation 13 was solved using the finite difference method. The soil-geosynthetic interface shear stress, τ, was then determined using Equation 5, and, subsequently, the average strain ε_0 (measured at $x = 0$) was calculated using Equations 6 and 7.

Figure 5 illustrates the influence of the soil-geosynthetic interface stiffness, G, on the average confined in-soil creep strain, ε_0. A weak interface, characterised by a low G value, results in a large amount of creep, in contrast to a strong interface (large G value), which considerably reduces the creep.

Figures 6 and 7 illustrate the strong influence of the soil-geosynthetic interface stiffness, G, on the tensile forces and corresponding soil-geosynthetic interface shear stress distributions, at the beginning of loading (instantaneous response at $t = 0$). Note that, for the unconfined tests, the distribution of F is uniform along the geosynthetic specimen, and there is no soil-geosynthetic interface shear stress. The case of $G = 10^5$ N/m^3 is similar to the unconfined geosynthetic behaviour at the beginning of creep tests (instantaneous response).

Rheological properties of a geosynthetic essentially modify the distributions of F and τ (Figures 8 and 9). There is a regrouping of forces, acting on the geosynthetic undergoing creep, that is characterised by a reduction of the F value along the geosynthetic strip and a corresponding increase of the soil-geosynthetic interface shear forces. This means that an increasing amount of external load is transmitted to the confining soil. The model described is valid until the soil-geosynthetic interface strength is reached. The analysis must then be modified, as shown by Sawicki (1998).

END OF EXAMPLE 6

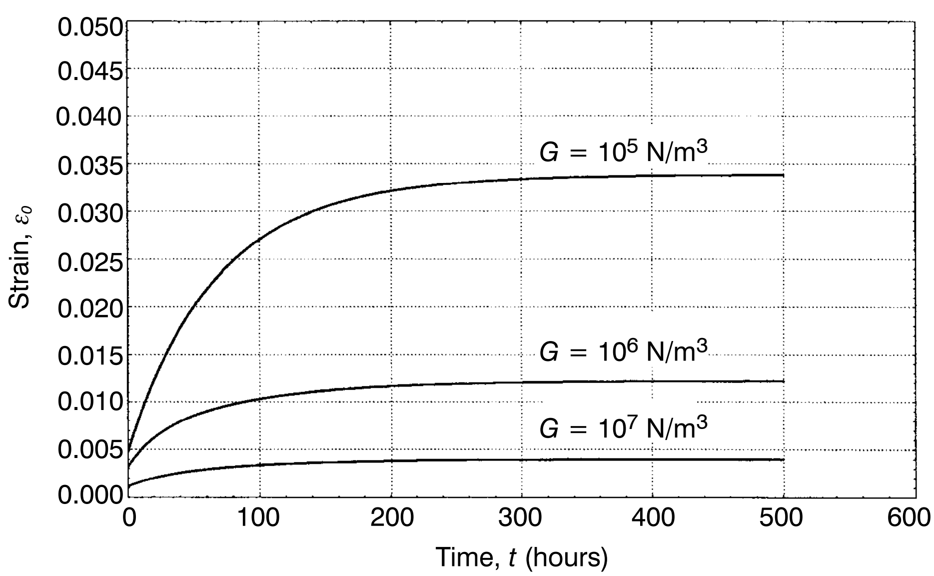

Figure 5. Influence of the soil-geosynthetic interface stiffness, G, on confined geosynthetic creep.

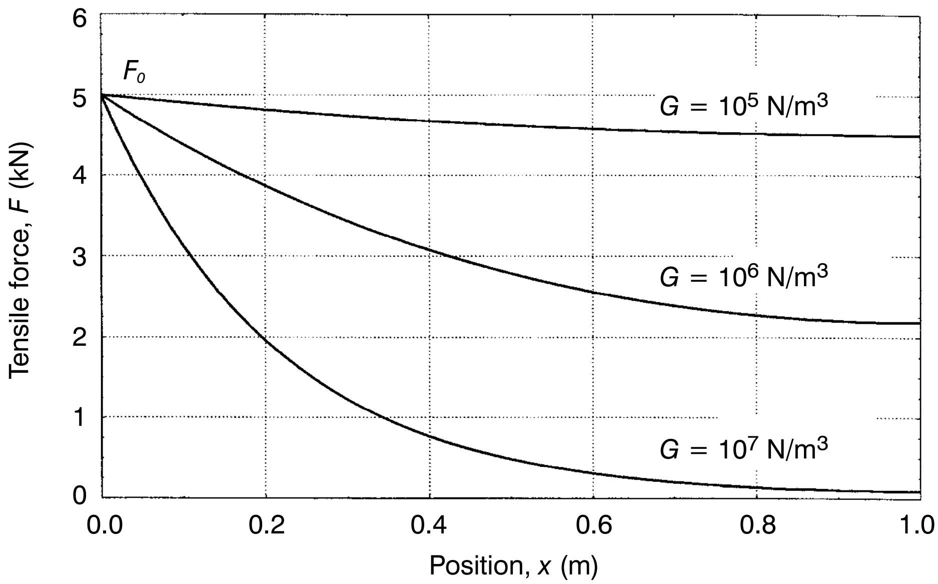

Figure 6. Instantaneous (t = 0 hours) distribution of the tensile force, F, along a confined geosynthetic for different G values.

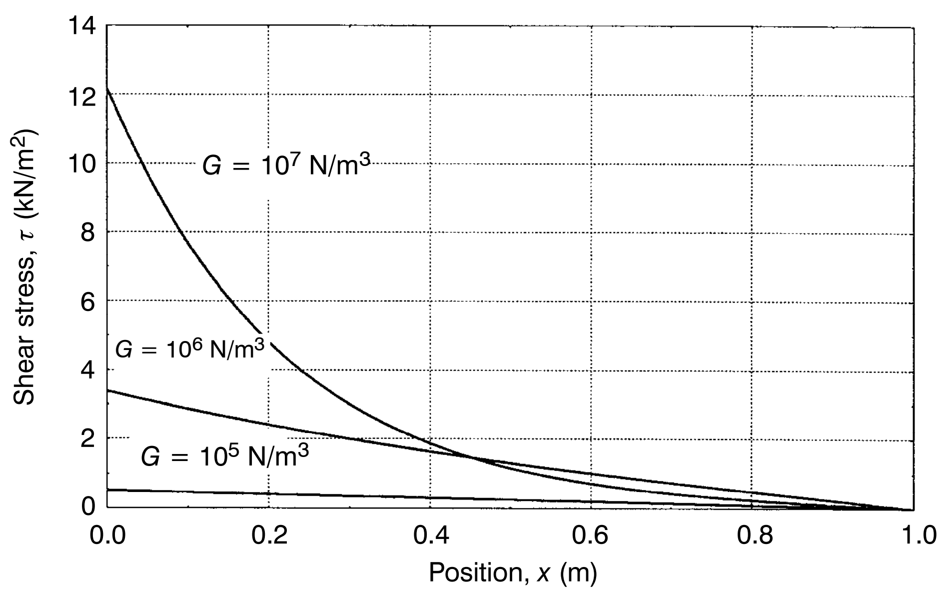

Figure 7. Instantaneous (t = 0 hours) distribution of the soil-geosynthetic interface shear stress, τ, of a confined geosynthetic for different G values.

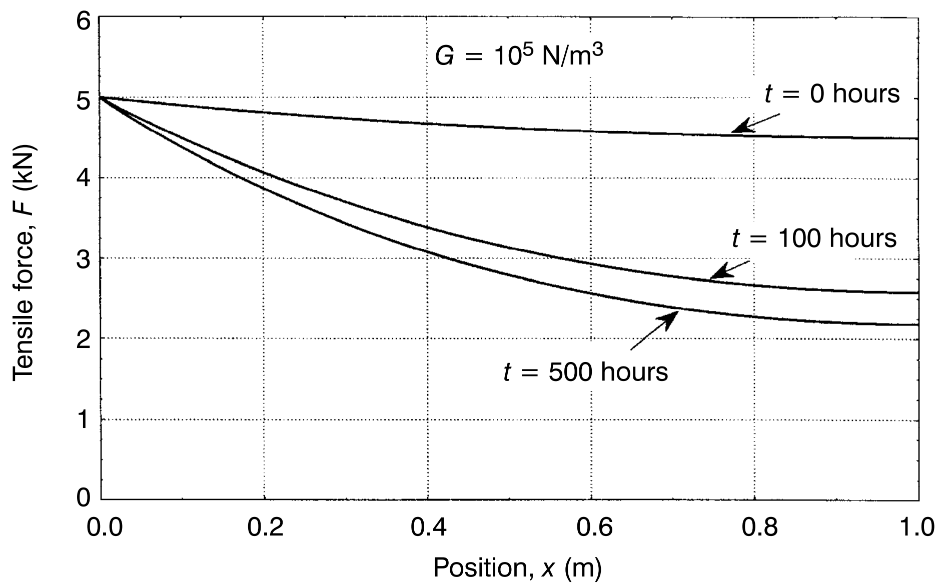

Figure 8. Changes in the distribution of the tensile force in a confined geosynthetic due to creep with $G = 10^5$ N/m^3.

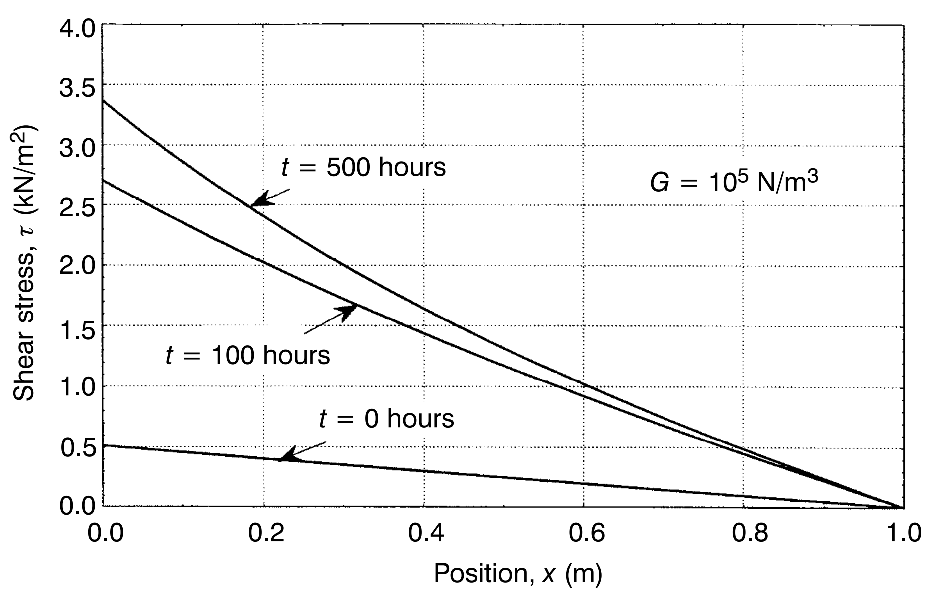

Figure 9. Changes in the distribution of the soil-geosynthetic interface shear stress of a confined geosynthetic due to creep with $G = 10^5$ N/m^3.

6 CONCLUSIONS

The main results and conclusions presented in the current paper can be summarized as follows:

- It was shown that the confined in-soil behaviour of geosynthetics, i.e. instantaneous load-extension and creep, can be predicted on the basis of unconfined test data and the soil-geosynthetic interface stiffness, G.
- The confined load-extension stiffness of a geosynthetic, E_{conf}, was derived analytically (Equation 9), which is a significant result of the current study.
- A method of determining the soil-geosynthetic interface stiffness, G, was proposed (Section 3) and experimentally verified on the basis of a limited amount of data. An empirical equation for G as a function of the confining stress was proposed (Equation 10).
- It was shown that the model of a single reinforcement strip previously proposed by Sawicki (1998) can be used to describe the confined creep of geosynthetics; Example 5 can serve as a limited experimental verification.
- A method of predicting the confined in-soil behaviour of geosynthetics was presented. It clarifies the different theories of confined geosynthetic testing.

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NOTATIONS

Basic SI units are given in parentheses

a_1	=	coefficient defined in Equation 13 (s^{-1})
B	=	width of geosynthetic strip (m)
b_1	=	coefficient defined in Equation 13 (m^{-1})
c_1	=	coefficient defined in Equation 13 (m^{-1} s^{-1})
E	=	elastic, instantaneous stiffness of geosynthetic strip (N)
E_1 , E_2	=	elastic stiffnesses of rheological model (N m^{-1})
E_{conf}	=	confined stiffness of geosynthetic strip (N)
E^*	=	coefficient defined in Equation 12 (N m^{-1})
F	=	tensile force (N)
F_0	=	pull-out force for case of strips (N) and for case of sheets (N m^{-1})
G	=	pull-out stiffness coefficient at soil-geosynthetic strip interface (N m^{-3})
G_0	=	coefficient appearing in Equation 10 (N^{1-n} $m^{-(3-2n)}$)
l	=	length of geosynthetic strip (m)
n	=	coefficient appearing in Equation 10 (dimensionless)
t	=	time (s)
u	=	displacement of geosynthetic strip (m)
α	=	coefficient defined in Equation 2 (m^{-1})
ε	=	geosynthetic strain (dimensionless)
ε_0	=	geosynthetic strain at point of load application (dimensionless)
$\phi(t)$	=	creep function of geosynthetic (N^{-1})
η	=	viscosity of geosynthetic (Ns m^{-1})
σ_{conf}	=	confining stress (N m^{-2})
τ	=	shear stress (N m^{-2})