GEOSYNTHETIC REINFORCED PAVEMENT ON SOFT SOIL - TRIAXIAL TESTING

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ABSTRACT: The contribution presents an analysis of repeated load triaxial test results for geosynthetic-reinforced base course material, which can be used for pavements. The physical and mechanical characteristics of base course material and subgrade soil were investigated. Triaxial tests were performed simulating traffic loading. Two types of samples were investigated, first without any reinforcement and then with reinforcement. The research was performed with a set of tests, where the conditions varied: density, moisture content, initial effective stress and cyclic loading. An analysis of the strength, resilient deformations and permanent deformations of pavement and sub-grade materials was performed. The reinforcement was found to have no essential effect on the resilient stiffness. A significant reduction in permanent deformations for the reinforced samples compared to the unreinforced samples was determined. Permanent deformations are expressed as a function of the number of loading cycles and the spherical component of the cyclic loading and the distortional component of the cyclic loading. The results can be presented as deformation and failure envelopes for an arbitrary number of cycles. The compaction grade of the material, its moisture content and degree of saturation essentially influence the degree of deformability. An elastic-plastic analysis using the finite element method (FEM) to make a comparison with the test results and to define the parameters for pavement FEM analysis was performed. By including conditions from the AASHTO design method for flexible pavements, the necessary combined structural capacity of the pavement layers overlaying the subgrade was calculated. The AASHTO design method for geosynthetic-reinforced pavements was developed. Pavement design parameters were compared for unreinforced and reinforced pavements and different sub-grade soil conditions. A cost analysis shows a significant savings for a lesser degree of sub-grade strength.

1. Introduction

Soil improvement could be an important solution for foundations on problem soils. Munfakh [1] provides five major functions of soil improvement applied to foundations:
- Increasing bearing capacity
- Controlling deformations and accelerating consolidation
- Providing lateral stability for slopes and excavations
- Seepage cut off and other types of environmental control
- Increasing resistance to liquefaction of loose, saturated granular deposits
Problems foundation soils can be improved by:
- Reducing the load and therefore the stress on the foundation
- Removing any problem soils and replacing them with more competent material
- Increasing the shear strength and reducing the compressibility of the problematic materials
- Reinforcing embankments on soft foundations

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These improvements and their variations provide many soil improvement methods or techniques [2], such as: reduced stress, removal and replacement, dewatering, drainage, vertical drains, preloading, lime cement columns, deep soil mixing, jet grouting, geosynthetic reinforcement, etc. Reinforcement of a pavement structure can be a very effective application for geosynthetics, especially in the case of weak subgrade conditions. The use of geosynthetics applied to improved flexible pavement system has increased markedly over the last three decades. Geosynthetic reinforcement is typically placed in the interfaces between the aggregate base course and the subgrade, although it could be placed in the interface between other layers (between asphalt layers, granular base and sub-base layers).

This paper is focused on an analysis of geosynthetic reinforcement pavements on soft soil. An analysis of the strength, resilient deformations and permanent deformations of pavement and subgrade materials based on a cyclic triaxial test that simulated traffic loading was made. The purpose of the analysis is to predict elastic and permanent deformations of a pavement structure, depending on the number of loading cycles and stress conditions.

The physical and mechanical characteristics of granular base and sub-base material and subgrade material were investigated [3]. Static and cyclic triaxial tests were performed in the Laboratory of Soil Mechanics (LSM), Faculty of Civil Engineering, University of Maribor. A set of tests with a spectrum of different stress states and repeated loading was performed. The granular base material was reinforced with geosynthetic reinforcements and compacted in a mould to a certain degree of density and moisture content.

The variables in the research were: grain size distribution, density, moisture content, interval of reinforcement, part of crushed gravel grains, initial effective stress and cyclic loading conditions (static stress, and spherical and distortional stress components). The design parameters derived from the laboratory testing are analyzed and presented. A cost benefit analysis is also included.

2. Flexible pavement models

The vertical settlement of pavement is treated as a consequence of pavement layer and sub-grade material deformations. Resilient and permanent deformations are determined for pavement layers and sub-grade material and different stress conditions.

Resilient deformation

A resilient strain is taken as the unloading strain from the maximum dynamic stress down to the static contact stress. The resilient modulus is defined as

\[ M_r = \frac{\Delta \sigma_d}{\Delta \varepsilon_a^r} \]  \hspace{1cm} (1)

The \( k-\theta \) model [4] could be used to describe the resilient behavior of unbound granular material in cyclic triaxial tests. Its expression is

\[ E' = k_1 \cdot \left( \frac{\sigma_1 + 2 \cdot \sigma_3}{p_u} \right)^{k_2} = k_1 \cdot \left( \frac{3 \cdot p}{p_u} \right)^{k_2} \]  \hspace{1cm} (2)

where

\( \sigma_d, \sigma_1, \sigma_3 \) cyclic deviatoric stress, principal stresses
\( \varepsilon_a^r \) resilient axial strain
\( E' \) resilient modulus
\( p, p_u \) mean normal stress, reference stress (101 kPa)
\( k_1, k_2 \) model parameters
In this model, the Poisson ratio $\nu$ is constant. With the $k-\theta$ model and a given stress path, we are able to solve elastic problem. The inelastic problem is solved using a constant Poisson ratio $\nu$ and the Young’s modulus $E$. The expression of volume strains $\varepsilon_v$ and shear strains $\varepsilon_q$ and modulus $E$, $K$, and $G$ is done using Boyce [5]. Shear strains $\varepsilon_q$ are

$$\varepsilon_q = \frac{p_a^{1-n} \cdot p_r^n}{3 \cdot G_a} \left( \frac{q_r}{p_r} \right)^2$$

where

$p_a$ constant stress (100 kPa)
$p_r$ mean normal stress
$q_r$ deviator stress
$G_a$ parameter of model

The resilient modulus is dependent on the mean and deviatoric stress. Considering both, Uzan et al. [6] expressed a new model. The modified AASHTO equation for modeling this behavior has been included.

$$M_r = K_1 \cdot \sigma_a \cdot \left( \frac{\sigma_1 + 2 \cdot \sigma_3}{\sigma_a} \right)^{k_2} \cdot \left( \frac{\tau_{oct}}{\sigma_a} + 1 \right)^{k_3}$$

where

$\sigma_a$ reference stress (101 kPa)
$\sigma_1, \sigma_3$ principal stresses
$\tau_{oct}$ octahedral shear stress
$k_1, k_2, k_3$ model parameters

**Permanent deformation**

The determination of permanent deformations of EFG materials is based on cyclic tests from the standards [7] and [8], which differ in some details. The magnitude and development of permanent deformations depend on the static stress state of the material, the magnitude of the cyclic load, the magnitude of the spherical and deviatoric cyclic load components and the relationship between them, the number of loading cycles, and the physical properties of the material (density, water content...).

The limit state plastic strains are calculated in agreement with the Hornych model [9]. The relation between the axial permanent deformations $\varepsilon_1^p(N)$ and the number of loading cycles $N$ is given by

$$\varepsilon_1^p(N) = \varepsilon_1^{p100}(N) + \varepsilon_1^p(100)$$

where

$\varepsilon_1^{p100}(N)$ axial permanent deformation after 100 cycles
$\varepsilon_1^p(N)$ normalized axial permanent deformation at $N > 100$

Eq. 6 describes the normalized axial permanent deformation, where parameters $A$ and $B$ define the deformation growth with the number of loading cycles. Parameter $A$ denotes the limit of the function of the permanent axial deformations, parameter $B$ denotes its deflection. Changes in the axial permanent deformation $\varepsilon_1$ and normalized axial permanent deformation $\varepsilon_1^{p*}$ are given as a function of the number of loading cycles $N$

$$\varepsilon_1^{p*} = A \cdot F(N) = A \cdot \left[ 1 - \left( \frac{N}{100} \right)^{-B} \right]$$
The magnitude of parameters $A$ and $B$ depend on the stress level that is expressed by the spherical and deviatoric stress components

$$ p = \sigma_0 + \frac{\sigma_1 + 2 \cdot \sigma_3}{3} \quad q = \sigma_1 - \sigma_3 \quad (7) $$

The change in the plastic strain with the loading cycles is

$$ \frac{d\varepsilon_p^\rho}{dN} = \frac{d\varepsilon_p^\rho}{dN} + A \cdot B \cdot \left( \frac{N}{100} \right)^{-\beta} \quad (8) $$

The maximum axial permanent deformation or parameter $A$ is proportional to the maximum deviatoric and spherical stresses given in Eq. 6.

$$ A = \frac{q_{\text{max}}}{a - b \cdot \frac{q_{\text{max}}}{(p_{\text{max}} + p^*)}} \quad (9) $$

where the stress parameter $p^*$ is defined with the section of the failure line with the $q$ axis in the $p$-$q$ space. It is determined from the linear equation of the test data, according to the method of least square deviation.

$$ q = k \cdot p - p^* \quad (10) $$

where $k$ is the slope of the failure line. Parameters $a$ and $b$ are determined from the test data, with linear equation of the inverse value of parameter $A$, using the method of least square deviation.

$$ A^{-1} = a \cdot \left[ \frac{q_{\text{max}}}{(p_{\text{max}} + p^*)} \right]^{-1} - b \quad (11) $$

The relation between parameters $a$ and $b$ gives the slope of the failure line, given by parameter $M$

$$ M = a/b \quad (12) $$

The failure line in the $p$-$q$ space is

$$ q = M \cdot (p + p^*) \quad (13) $$

The development of the permanent axial deformations in the $p$-$q$ space can be given by

$$ \varepsilon_p^\rho(N, p, w, p, q) = \varepsilon_0 \cdot f(p) \cdot f(w) \cdot f(p) \cdot f(q) \quad (14) $$

where the characteristic value of the permanent axial deformation $\varepsilon_0$ is given at selected values of the spherical stress component $p_0$ and the deviatoric stress component $q_0$.

$$ \varepsilon_0 = \varepsilon_p^\rho(N_0, p_0, w_0, p_0, q_0) $$

The magnitude of the permanent axial deformation depends on the spherical stress component $p$ and the deviatoric stress component $q$, following the exponential equation

$$ \varepsilon_p^\rho(N, p, q) = \varepsilon_0 \cdot e^{C \left( \frac{p}{p_0} \right)^\rho} \cdot e^{-E \left( \frac{q}{q_0} \right)^q} \quad (15) $$

where $C$ and $D$ are the parameters that define the variations of the normalized permanent axial deformation $\varepsilon_p^\rho$ as a function of the spherical stress component $p$, $E$ and $F$ are parameters that define the variations of the permanent axial deformation as a function of the deviatoric stress component $q$. 
Similarly, the variation of the other influences (density, water content) can also be written. Due to the minimal changes in these parameters for individual type of material, they will be considered as a constant condition. From Eq. 14 the normalized permanent axial deformation $\varepsilon_i^{p^*}$ is expressed regarding to the spherical stress component $p$ and the deviatoric stress component $q$, which can be given with the surface in the $\varepsilon_i^{p^*} - p - q$ space.

At a certain constant value of the spherical stress component $p$, the value of $\varepsilon_i^{p^*}$ increases exponentially with the increased $q$, and at a certain limit, the value $q$ diverges. For each plane in the $\varepsilon_i^{p^*} - p - q$ space with an arbitrary value of $p$, the relation between the deviatoric stress component $q$ and the normalized permanent axial deformation $\varepsilon_i^{p^*}$ can be determined.

$$q(\varepsilon_i^{p^*}) = q_0 \cdot \left[ 1 + \frac{1}{E} \cdot \ln \left( \frac{1}{\varepsilon_1^{p^*}} \frac{\varepsilon_i^{p^*}}{\varepsilon_0} \right)^{\frac{1}{E}} \right]^{\frac{1}{p}}$$ (16)

With the increased value of the spherical stress component $p$ and the certain constant value of the deviatoric stress component $q$, the value of $\varepsilon_i^{p^*}$ converges to a certain value. From Eqs. 15 and 16, the failure line and deformation line in the $p-q$ space can be determined, at a certain number of loading cycles $N$.

3. Repeated load triaxial tests

This paper considers the repeated load triaxial tests of unbound mixtures performed according to the variable confining pressure method (where the cell pressure is cyclically changed in the phase with the vertical axial loading) and to the constant confining pressure method (where the cell pressure is constant, and only the vertical axial loading is cyclically changed).

The investigation of the material comprises a set of individual tests that are performed as a series of stress paths where the stresses are increased. The ratio of the distortional and spherical stresses $q/p$ during an individual test is between 0 and 3. The tests are performed with a low number of cycles ($N=100$), executed after conditioning with 20 000 cycles, to evaluate the elastic parameters and with a high number of cycles (80 000 or more) to determine the permanent deformation characteristics.

In order execute the cyclic triaxial tests, proper test equipment is required which comprises a triaxial cell, a press with appurtenant equipment, measuring equipment, a control system, a data storage system and computer hardware and software. The calculation of the strength characteristics regarding the measured deformation and stress parameters is performed after each individual investigation phase. The following quantities are measured: cell pressure $\sigma_3$, vertical stress $\sigma_1$, vertical strain $\varepsilon_1$ and radial strain $\varepsilon_3$.

The applicability of the presented model was verified on the practical example of the repeated load triaxial test of sub-grade and base course granular material. The Drava river’s sub-grade sandy gravel was investigated using repeated load triaxial tests [10]. Samples with and without reinforcements were tested. The results showed the effect of the preloading, the compaction grade of the material and its moisture content to the deformability.

An analysis of the development of the permanent axial deformations of different base course gravel materials due to repeated loading was performed [12]. The types of stone material were
also determined with regard to the quantity of the crushed grains in the gravel and to the water content.

The investigations of sub-grade and base course stone material were performed in laboratories and in the field. The laboratory investigations comprised:
- Properties of the stone materials (mineralogical and petrographic composition, content of hazardous admixtures, mechanical properties, quality of stone materials, physical properties, geometric and mechanical properties of the stone grain mixture)
- Experimental incorporation of the stone grain mixture (in the laboratory, in the field)
- Procedures for improving the incorporation of the stone grain mixture (crushing, improvement with alloys (electro filter ash) and soil reinforcement)
- Repeated load triaxial tests

**Base course gravel**

Part of the presented analysis uses data from the report [11]. The base course material was tested with repeated loading at a water content of \( w = w_{\text{opt}} - 2\% \). The physical properties were constant for each type of stone material.

**Sub-grade material**

An investigation of the quality of the natural sandy-gravel from the existing deposits in the Drava river region was done in laboratories and in the field. Repeated load triaxial tests were performed. Samples of the stone material were tested. The set of tests on samples with a water content of \( w = w_{\text{opt}} \pm 0.5\% \) was considered. The physical properties of the types of stone material analyzed were varied regarding the subgrade strength selected, which is the given by California bearing ratio (CBR).

**Effect of reinforcement**

Specimens with and without reinforcements were tested. The effect of reinforcements on resilient stiffness and permanent deformations was analyzed.

**4. Deformation analysis**

The deformation analysis was based on a set of repeated load triaxial tests performed on each type of material. Fig. 1 shows the typical result measured (increasing strain) for the repeated load sub-grade sandy-gravel. Fig. 2 shows the result measured for sub-grade sandy-gravel with a relation of deviatoric stress \( \Delta q \) versus a vertical axial strain \( \varepsilon_1 \).

![Fig. 1. Deviatoric stress \( \Delta q \) and vertical axial strain \( \varepsilon_1 \) vs. time \( t \) (number of loading cycles)](image)
Fig. 2. Deviatoric stress $\Delta q$ vs. vertical axial strain $\varepsilon_1$

The parameters $A$ and $B$ were determined from Eqs. (6) and (8), which define a numerical solution for the development of permanent axial deformations $\varepsilon_1'(N)$ and normalised permanent axial deformations $\varepsilon_1^{p*}(N)$.

Fig. 3 shows an increase in the normalised axial permanent deformation $\varepsilon_1^{p*}$ with the number of loading cycles $N$ for the uncrushed type of base course stone material with a water content of $w = w_{opt} - 2\%$.

Fig. 3. Normalised axial permanent deformation $\varepsilon_1^{p*}$ vs. the number of loading cycles $N$ for various loadings of stone materials with a water content of $w = w_{opt} - 2\%$

The course of the normalized axial permanent deformation $\varepsilon_1^{p*}$ could be expressed as a function of the deviatoric stress $q$ and the spherical stress $p$ for each number of cycles. The relation between the normalized axial permanent deformation $\varepsilon_1^{p*}$ and the deviatoric stress $q$ for the constant spherical stress $p$ (viewed in individual planes in the $\varepsilon_1^{p*} - p - q$ space) is obtained. Fig. 4 shows the normalized axial permanent deformation $\varepsilon_1^{p*}(N)$ vs. the spherical stress $p$ and the deviatoric stress $q$ for the sub-grade material.
Fig. 4. Sub-grade normalized axial permanent deformation $\varepsilon_1^{p*}$ in the $\varepsilon_1^{p*} - p - q$ space.

The relation between the normalized axial permanent deformation $\varepsilon_1^{p*}$ and the distortional stress $q$ for the constant spherical stress $p$ (view in individual planes in the $\varepsilon_1^{p*} - p - q$ space) was obtained from Eqs. 15 and 16. Fig. 5 shows these relations for the base course material.

Using Eqs. 10 to 13, we can obtain failure envelopes in the $p - q$ space for the individual types of stone material.

The relation between the spherical stress component $p$ and the deviatoric stress component $q$ at a certain value of the normalized axial permanent strain $\varepsilon_1^{p*}$ provides the so-called deformation envelopes. Deformational envelopes decrease with the number of cycles $N$ and increase with an increase in the axial permanent strain and approach the failure envelope, which represents the limit state of the relation $q/p$ (Fig. 6).

Fig. 5. Normalized axial permanent deformation $\varepsilon_1^{p*}$ vs. distortional stress $q$ for various constant spherical stresses $p$
5. Analysis of pavement structure

An elastic-plastic analysis using the finite element method (FEM) to make a comparison with the test results and to define the parameters for the pavement FEM analysis was performed. From the results (the axial force, horizontal deformation and shear stress) of the elastic-plastic analyses, it is clear that the results differ a lot when a reinforcement is installed.

The elastic-plastic analysis of a flexible pavement structure involves three steps:
- The mesh, material properties, load and boundary conditions of the finite elements are defined.
- An elastic analysis with the maximum (minimum) stress components is performed.
- The plastic displacements and strain components are computed.

The relation of the subgrade CBR versus the number of loading $N$ to reach a defined rut depth (10, 20 mm) for different thicknesses of gravel layers is determined. The gravel layer is reinforced or unreinforced. By including the conditions from the AASHTO design method for flexible pavements, the necessary combined structural capacity of the pavement layers overlaying the subgrade is calculated. Figs. 8 shows the necessary thickness of the base layer for different subgrade strengths and weightless, medium and heavy traffic pavements and...
with asphalt layers 10, 15 or 18 cm thick. The calculations with and without reinforcement, considering the 80 or 90 cm freezing zones, are compared.

**Cost-saving analysis**

Using an average cost of 15 EUR for an engraved aggregate base and a cost of 1.2 EUR for an engraved geogrid reinforcement in place, the savings for utilizing reinforcements at different subgrade strengths is calculated. Figs. 9 shows the cost saving in pavements with reinforcement per m² for different subgrade strengths and traffic loading for 80 and 90 cm freezing zones.

**6. Conclusions**

A set of tests with a spectrum of different stress states and dynamic loadings were performed. The base course material and sub-grade material was reinforced with geosynthetic reinforcements and compacted in a mould to a certain density and moisture content. Two types of samples were investigated: first, without any reinforcement, and second with reinforcement. The research was performed with a set of tests, where the conditions varied by density, moisture content, initial effective stress and cyclic loading.

The varying in the materials stiffness was observed during the cyclic loading. The resilient modulus was defined using the AASHTO 2002 PDG model. The reinforcement was found to have no essential effect on the resilient stiffness.
A significant reduction in permanent deformations for the reinforced samples compared to the unreinforced samples was determined. The permanent deformations are expressed as a function of the number of loading cycles $N$, the spherical component of the cyclic loading $p$ and the distortional component of the cyclic loading $q$. The results can be presented as deformation and failure envelopes for an arbitrary number of cycles $N$.

An elastic-plastic analysis using the finite element method (FEM) was performed to make a comparison between the test results and to define the parameters for a pavement FEM analysis. By including conditions from the AASHTO design method for flexible pavements, the necessary combined structural capacity of the pavement layers overlaying the subgrade was calculated. The AASHTO design method for a geosynthetic reinforced pavement was developed. The pavement design parameters were compared for unreinforced and reinforced pavements and various subgrade soil conditions. A cost-benefit analysis was also included and shows a significant degree of savings for a smaller sub-grade strength.

References