Geosynthetic-reinforced granular materials

Matériaux granulaires renforcés de géosynthétiques

D.GORLÉ, Belgian Road Research Centre, Brussels, Belgium
M.THIJS, Belgian Road Research Centre, Brussels, Belgium

SYNOPSIS : New, high performance geosynthetics were used to improve the behaviour of granular materials in low deformation road structures. On the basis of the results of model tests with prestressed geosynthetics and laboratory tests with granular materials either confined by geotextiles or reinforced with short fibres, it was shown that confinement, and especially fibre reinforcement, lead to a considerable improvement of the mechanical properties of these materials. The tests indicated an increase of their modulus of elasticity and their shear resistance (angle of internal friction, cohesion), capable of transforming the properties of a clean sand into those of a crushed stone layer.

1. INTRODUCTION.

Coarse-grained noncohesive materials are important in road construction as base and subbase materials. Their behaviour depends strongly on their density and the confining stress (Barksdale 1972, Seed 1965, Descornet 1977). When used on soft soils, geotextiles, or more generally geosynthetics, may be very useful to separate them from the soil and thus prevent contamination. For many road bases, subbases, or even roads for construction traffic, the allowed deformations are too small to stress the geotextiles sufficiently in order to make an effective contribution to the stress distribution.

In recent years new geosynthetics have been developed with higher moduli, higher strength and smaller elongation at failure. This creates new possibilities to improve the behaviour of granular materials, the following of which are examined in this paper :

- prestressing geosynthetics,
- confining granular materials by geotextiles,
- reinforcing granular materials by fibre inclusions.

2. MODEL TESTS WITH PRESTRESSED GEOSYNTHETICS.

2.1. Selection of model test parameters.

2.1.1. Test methods.

To study the effect of prestressed geosynthetics in a two-layer model (soil-granular material), the plate-bearing test (BRRC 1978) has been selected. The plate with an area of 750 cm² was used.

During the test, soil strain was measured by a cluster of free floating soil strain gauges (Selig 1975) installed in the soil and in the granular layer of the model. The development of horizontal soil stresses just above the level of the geosynthetic was measured by electric soil pressure gauges. The prestressing and anchoring forces of the geosynthetic were measured in two orthogonal directions by means of digital dynamometers. A possible shift of the moisture contents of both layers was monitored by a nondestructive method based on the time-domain reflectometry principle and developed at the Belgian Road Research Centre (Ledieu 1986). Settlements of the geosynthetic or soil surface were checked by topographical measurements during the construction of the model and after completed testing. The bearing capacities of the soil and the sand layer were estimated during the construction of the model and after completed testing by means of a light dynamic penetrometer, using experimental relations with the Californian Bearing Ratio (CBR value), (BRRC 1978).

2.1.2. Model geometry.

An overall view of the test model is shown in figure 1.

The horizontal dimensions of the model are 2 m by 2 m. The layer thicknesses were 0.15 or 0.30m for the granular material and 0.40 m for the soil.

2.1.3. Geosynthetics.

For the models with sand as a granular material, only a woven polyester geotextile (tensile strength 66 kN/m) was used. The models with crushed limestone were reinforced by either the same geotextile or a geogrid (tensile strength 32 and 18 kN/m).

2.1.4. Soil and granular materials.

The selected soil was a loam, typical of the central part of Belgium. The loam was compacted at moisture contents and to a density to obtain CBR values between 3 and 6 %.

A natural sand meeting the Belgian specifications for draining layers (MPW, 1982) was used for the tests. The sand was compacted at its optimum moisture content, to obtain CBR values around 10 % (0.30 m layer) and 6 %

TABLE I

Results of the Plate Bearing Tests

Model composition			Prestressing Force (kN/m)		CBR Soil	Loading	M1		M2		Mc1		Mc10		Mcm		Δz mm		
Granular material	Thickness m	Geosynthetic	x	y	%	p_{max} MPa	L MPa	U MPa	L MPa	U MPa	L MPa	U MPa	L MPa	U MPa	L MPa	U MPa	total	layer	soil
sand	0.30	gtx	-	15	4.5	0.45	17	104	83	102	73	139	120	148	108	145	10.6	6.3	4.3
sand	0.30	gtx	10	10	3.9	0.45	12	86	51	198	88	131	110	147	96	138	16.2	11.2	5.0
sand	0.30	none	-	-	5.3	0.45	13	91	62	106	62	128	105	143	91	138	14.7	9.4	5.3
sand	0.30	gtx	-	-	5.5	0.45	11	60	39	63	34	73	85	143	57	96	32.4	18.8	13.6
sand	0.15	gtx	8	8	5.7	0.55	13	61	46	81	58	263	143	120	104	137	17.6	3.4	14.2
sand	0.15	gtx	-	8	5.6	0.55	16	-	34	98	60	117	108	105	81	118	12.4	4.6	7.8
sand	0.15	none	-	-	4.3	0.55	11	63	44	86	54	127	83	110	77	111	24.3	3.3	21.0
sand	0.15	gtx	-	-	3.2	0.55	7	53	35	73	43	76	70	90	60	84	29.8	5.3	24.5
stone	0.15	gtx	8	8	4.7	0.55	17	67	48	113	85	115	122	130	104	129	10.0	3.9	6.1
stone	0.15	none	-	-	4.3	0.55	28	141	86	180	90	163	129	162	115	162	14.3	3.2	11.1
stone	0.15	grid	8	8	8.0	0.55	28	88	73	139	90	106	132	146	105	128	5.0	0.9	4.1
stone	0.15	grid	-	-	4.0	0.55	12	75	55	114	63	96	114	137	95	128	21.4	1.8	19.6

Legend: $M = (d \times \Delta p) / \Delta z$ (MPa), coefficient of compressibility: d = diameter of the plate
Mi, Mcj, Mcm coefficient of compressibility of respectively step i, cycle j and the mean of all cycles of the test.
L : loading; U : unloading.

(0.15 m layer). The crushed limestone was a mixture with 0/32 continuous grading according to the type II specifications (MPW 1982). The crushed stone was compacted at its optimum moisture content, to between 90 and 95 % of the modified Proctor USCE density.

2.1.5. Anchoring and prestressing procedures.

Special clamps were made for anchoring or prestressing the geosynthetics, as shown in figure 1. The chosen procedure of prestressing was with internal anchoring of the geosynthetic. Prestressing forces varied between 15 kN/m (0.30 m layer, one direction prestressing) to 10 kN/m (0.30 m, two directions) and 8 kN/m (0.15 m, two directions).

2.2. Test results.

The results of 3 series of plate-bearing tests and some additional information on the bearing capacity and settlements of the subsoil, settlements of the granular layer and maximum plate stress, are summarized in table I.

Releasing of the anchorage of the prestressed geosynthetic resulted in a horizontal confining stress of maximum 20 kPa building up in the granular material. During the test no significant need for anchoring force was measured at the limits of the model. With the 0.30 m sand layer, failure occurred systematically in the sand, while with the 0.15m sand and crushed stone layers, failure generally occurred in the subsoil. Figure 2 represents the development of vertical strains in the sand and the upper part of the subsoil, for a model test with 0.15 m sand and biaxially prestressed geotextile (Fx = Fy = 8 kN/m).

2.3. Conclusions.

For low deformation soil-sand systems (CBR of soil between 3 and 6 %), the prestressing of geosynthetics limits the total settlements of the system and increases the cyclic bearing capacity, especially when the layer thickness is less than or equal to the radius of the loading surface. Supplementary anchoring of geosynthetics is not effective ; current anchoring lengths of 0.5 to 1 m are sufficient. The use of geosynthetics with crushed stone layers does not seem effective in low deformation systems. The prestressing of geosynthetics does not seem to be economically justified on the basis of these results, considering the rather complicated prestressing procedure in full scale application.

3. LABORATORY TESTS ON CONFINED AND FIBRE REINFORCED MATERIALS.

For road applications, the behaviour of noncohesive coarse grained granular materials can be adequately described by a nonlinear resilient modulus depending on the confining stress and the evolution of the total axial strain as a function of the number of load repetitions (Barksdale 1972, Seed 1965, Descornet 1977).

Starting from this behaviour, and from the same point of view, the influences of confining stresses created by geosynthetics and reinforcement by synthetic fibres were examined.

The tensile strengths of the geotextiles used in this study cover a wide range: between 29 and 200 kN/m. The elongation at failure varies from 11 to 25%. Two types of polymers, polyester and polypropylene, with very different mechanical properties, were tested.

Two different granular materials were considered: a coarse uniform sand (0/2 mm) and a well graded crushed stone (limestone 0/20 mm).

The angles of internal friction of the sand and the crushed stone were respectively 35.7° and 45.3° at densities of resp. 1.64 and 2.15 t/m³.

Photo BRRC 2396/35

Figure 1. Overall view of the model

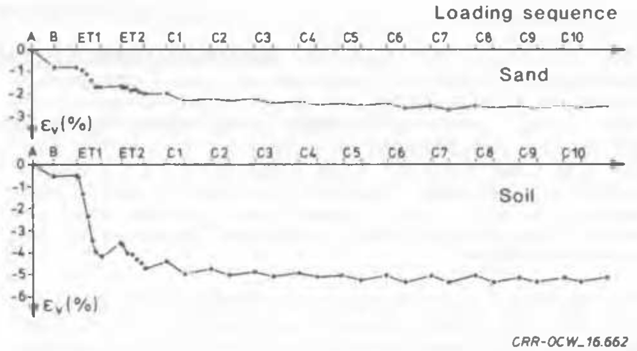

Figure 2. Vertical strains during the test

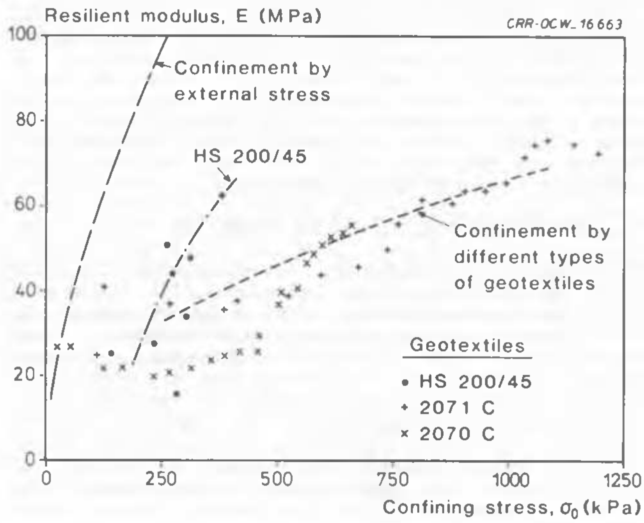

Figure 3. Resilient modulus of cylinders of sand and different geotextiles against the confining stress

3.1. <u>Triaxial tests on granular materials confined by geotextiles.</u>

The test method has been described by Gorlé, 1988. Cylinders of geotextile fabrics with diameter 20 cm and height 25 cm, or diameter 32 cm and height 32 cm for the tubular geotextiles, are filled with the granular material, which is compacted in several layers. The samples are then placed under a press and repeatedly vertically loaded and unloaded. These cycles are repeated with increasing stresses. The vertical stresses and vertical and radial strains are recorded.

The confining stresses σ_0 can be estimated from the measured strain of the geotextiles and their stress-strain characteristics.

A resilient modulus E of the samples can be calculated after a sufficiently high number of cycles, from the reversible strain.

The results of the tests are summarized in figure 3, in which the elasticity moduli of cylinders of sand and different types of geotextiles are plotted against the confining stress.

For the tubular weavings, where no problem of joints exists, the tensile stresses obtained were very close to the tensile strengths of the geotextiles with, however, a larger dispersion for the coarse materials. This dispersion can be explained by the higher local stresses, for example at the sharp corners of the stones. It was also demonstrated that the method of compaction can have an important influence, depending on whether the geotextile is prestressed or not.

For a given granular material, the vertical stresses at failure and the elasticity moduli increase with the tensile strength of the geotextile. Smaller elongation of the geotextile at failure gives a better performance of the system: for the crushed stone mixture as well as for the sand (see figure 3) the HS200/45 geotextile gave, at the same confining stress, a much larger increase of the modulus. This is due to a better compatibility of the stress-deformation characteristics of the textile and the granular material, and shows the necessity of developing fabrics with high strength and small elongation at failure.

3.2. <u>Tests on granular materials reinforced with synthetic fibres.</u>

To demonstrate the influence of synthetic fibre inclusions on granular materials, mixtures of the sand with high and low modulus tapes were tested under triaxial conditions.

The mixtures of sand and fibres are subjected to consolidated and drained triaxial tests, which allow to study the influence of the fibres on the internal friction characteristics.

The results are summarized in figures 4 and 5. They show considerable improvement in the angle of internal friction, the cohesion and the elasticity modulus of the granular materials with, however, larger deformations. For a same number of fibres per unit volume, the high modulus tapes (HM) gives a better performance than the low modulus tape (MM), as illustrated in figure 4.

3.3 Conclusions

The behaviour of granular materials confined by geotextile fabrics depends on many parameters such as the geometry of the confining structure and the characteristics of the granular material, and depends strongly on the properties of the geotextile. Geotextiles with a higher modulus, higher tensile strength and lower susceptibility to creep and relaxation are more compatible with granular materials and perform better.

Geotextiles with tensile strenghts of the order of 30 kN/m can already increase the resilient modulus of a sand to values obtained for crushed stone without geotextiles.

The tests on granular materials reinforced with short synthetic fibres show a considerable influence on the cohesion, the angle of internal friction and the modulus, depending not only on the amount of the fibres but also on their nature and their dimensions in relation to the pore size distribution of the granular material. Even a small amount of fibres (e.g. 0.45 %) is enough to substantially increase the resilient modulus of the granular material.

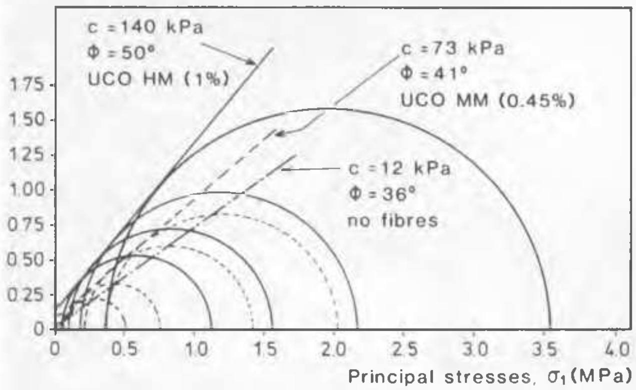

Figure 4. Triaxial tests on sand reinforced with the same number per unit volume of HM and MM fibres

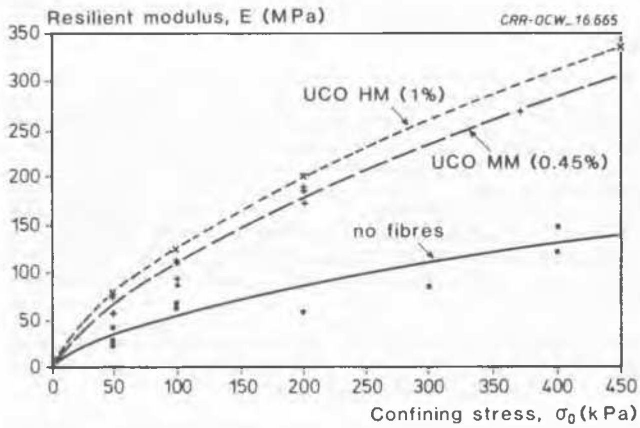

Figure 5. Resilient modulus of sand with and without synthetic fibres as a function of the confining stress

4. PERSPECTIVES FOR FURTHER RESEARCH.

Prestressed geosynthetics do not seem to open real perspectives for economically justified full scale applications with granular materials in road construction. Confinement, and especially short fibre reinforcement of these materials have been very effective in increasing their mechanical performance, while their technological feasibility is much greater. Further research will focus in the first place on fibre reinforcement techniques, which are the best suitable for application in continuous road layers. Research has to deal with fibre geometry optimization, mix design, mixing and compaction techniques, while a more precise knowledge of the constitutive characteristics of the composite material has to be obtained.

5. ACKNOWLEDGEMENTS.

The authors wish to express their gratitude to the Belgian IRSIA for sponsoring the research on prestressed geosynthetics, and the Commission of the European Communities for its financial support to the laboratory study on confined and fibre-reinforced granular materials (BRITE-program). They also thank J. Reichert, Director, J. Verstraeten, Deputy director, and their collaborators of the Belgian Road Research Centre for their contribution to the study.

REFERENCES.

Barksdale,R.D.(1972). Repeated Load Test Evaluation of Base Course Materials. GHD Research Project No. 7002. Final Report, Georgia Institute of Technology.

Seed, M.B., Mitry F.G., Monismith, C.L., Chan, C.K.(1965). Prediction of Pavement Deflections from Laboratory Repeated Load Tests, NCHRP Report No. TE-65-6, University of California, Berkeley, 1965.

Descornet,G.(1977).Essais de compression repétée sur matériaux granulaires. Compte rendu du IXe Congrès Int. Méc. des Sols et Trav. Fond. 2/4/6, 203-211, Tokyo.

Belgian Road Research Centre (BRRC).(1978). Mode opératoire, Essai de chargement à la plaque pour le contrôle du compactage. Méthode de mesure CRR, MF 40/78, Centre de recherches routières, 23 pp., Bruxelles.

Selig,E.T.(1975). Soil Strain Measurement Using Inductance Coil Method. Special Technical Publication 584, ASTM, Philadelphia, U.S.A..

Ledieu, J., Deridder, Ph., De Clerck, P., Dautrebande, S.(1986). A Method of Measuring Soil Moisture by Time-Domain-Reflectometry. Journal of Hydrology, v. 88, 319-328.

Belgian Road Research Centre (BRRC).(1978). Mode opératoire. Estimation rapide de la portance des sols à l'aide d'une sonde de battage légère type C.R.R., Méthode de mesure C.R.R., MF 39/78, Centre de recherches routières, 23 pp., Bruxelles.

Ministry of Public Works. (1982). Cahier des charges-type 150 - Méthode d'essai, Fonds des routes, Bruxelles.

Gorlé, D., Saeys, L. (1988). Reuse of Crushed Concrete as a Road Base Material. Second Rilem Symp. on Demolition and Reuse of Concrete, Tokyo.