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THE USE OF GEOSYNTHETICS TO IMPROVE THE BEARING CAPACITY OF THE FOUNDATION OF A ROAD EMBANKMENT

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The paper describes the difficulties encountered during the design and construction of two different road embankments over marshy areas, both characterised by the presence of highly compressible soils and of a high water table.
The foundation layer of these embankments were reinforced with layers of geosynthetic reinforcement.
The geological situation of the area, the problems encountered and the solutions found to solve these problems using geosynthetics are described, together with the design considerations about the effect of reinforcement and a final evaluation of the advantages provided by geosynthetics to the Contractor and the Client.

1. INTRODUCTION

Sometimes, lack of available sites make it necessary to locate development areas, roads and parking lots over extremely bad soils, like marshy lands or peat soil areas. The use of properly selected geosynthetics can solve the geotechnical problems associated with the reclamation of these marginal lands.
Whenever an embankment has to be built over soils with low bearing capacity, it is necessary to place a base layer between the embankment itself and the poor subgrade, with the main function of spreading the load and ensuring a proper Factor of Safety (FS) to the whole structure.
It is considered usually that the collapse of an embankment occurs when its settlements reach the serviceability limit usually equal to few centimetres.
The collapse of the structure may occur due to:

1. application of a single concentrated load exceeding the bearing capacity of the foundation;
2. small permanent plastic deformations, which summoning up during several load cycles cause a global settlement exceeding the serviceability limit;
3. development of large strains due to shear failure of the structure

Obviously, the first mechanism will never be possible for a road embankment (or, at least, it should never be possible); an estimation of the static bearing capacity of a strip foundation (like an embankment can be considered) doesn't represent a particular difficulty.
On the contrary, it is much more difficult to evaluate permanent strains due to local plasticizations and consequent shear failures.
A good estimation could be done using F.E.M.; anyway, the difficulties in the discretisation of the reinforcement and in the definition of the actions at the interface between reinforcement and soil, together with the costs of this kind of analysis, are well known. In general, all the classical methods for dynamic calculation of embankment foundations yield the required gravel thickness necessary to guarantee a fixed number of passages of a standard axle load.
The insertion of a geosynthetic layer at the interface between the sub-base and the base layer and/or within the base layer itself, allows the design engineer to:

1. reduce the required gravel thickness;
2. increase the design life (that is the number of passages) having the same soil thickness and type;
3. reduce the quality of the fill (hence its cost) for the base layer.

A reinforcing geosynthetics is able to provide its function inside the base layer in four main ways (Giroud et al., 1984).

- First of all, the presence of a geosynthetic able to interact with the surrounding soil limits the horizontal strains, thus limiting the deterioration of the base layer and preserving its thickness. The general behaviour of a geogrid is different from the behaviour of a geotextile (even with similar tensile properties). It is well known, in fact, that a geotextile is able to transmit stresses to the soil through friction only; therefore the effective capacity of a geotextile to transmit stresses is strongly affected by the shape and dimension of soil grains. A geogrid, instead, has a mesh structure; a soil, even coarse, can interlock with the apertures of a geogrid. Stress transmission in this case is immediate.
- The reduced possibility of strains inside the base layer, then, limits the possibility of creation and propagation of fractures in the lower part of the base layer itself. In this sense, the geosynthetic has a sort of anti-contamination effect.
- A geogrid reduce the possibility of local failures (as it reduces the strains inside the reinforced soil).
- Finally, a properly selected geosynthetic reduces the possibility that the soil with good technical properties used for the base layer could sink into the subsoil due to the load-unload cycles suffered since the beginning of the construction.

A geogrid is able to act as a separator layer only when the mesh aperture is less or equal to twice the minimum grain size. Obviously it is nearly impossible to have this condition with typical bioriented geogrids. On the other hand, a geotextile with a proper unit weight (at least 300 g/m²) is perfectly able to act as a separator layer; if the unit weight is lower, then the tensile properties and the strains are greater. The strains that are compatible with a foundation layer are in the order of 2%.

The need to obtain a separation effect together with a good interlocking effect brought to develop a new kind of product: a geocomposite obtained through thermal-bonding of a bioriented geogrid with a nonwoven geotextile. In this way the geotextile acts as a separator layer; the geogrid "stiffen" the geotextile, allowing the use of lower grade ones. At the same time the geogrid acts as a reinforcement of the base layer. In fact this geocomposite, thanks to the high local deformability of the needle-punched nonwoven geotextile (within the geogrid apertures), allow to have a perfect interlocking effect. Nowadays, this innovative solution has been followed by almost all the geosynthetics producer.

The provision through geocomposites of all the above mentioned reinforcement mechanisms can then provide a global improvement of the subsoil characteristics, since its boundary conditions are changed. It is easy to understand that the confinement of the soil particles together with a better load distribution and a sort of "membrane effect" provided by the tensile members of the geogrids can dramatically increase the bearing capacity of the subgrade-base- embankment structure.

If it is easy to have a quality idea of the effect of reinforcement, it is difficult to have a quantification of it; there are some solutions derived from extrapolation of experimental results (in situ - Giroud et al, 1984, or in laboratory - Montanelli et al., 1995). The validation of these methods, anyway, should come only from real applications.

2. BESNATE BRIDGE APPROACH EMBANKMENT

Near the small town of Besnate, in Northern Italy, an embankment had to be built to link the A8 highway Milano-Sesto Calende with a main road (SP 26); the embankment (with a maximum height of 8.00 m) had to pass over an area where the water table is very close to the ground surface.

The impossibility to pass over the marshy area with the testing machinery required the use of a geogrid-geotextile geocomposite, directly laid on the soil and covered with about 0.30 m of sandy gravel.

2.1 Geological and geotechnical situation

The plain area that had to be crossed by the new road embankment is part of an ancient (Late Pleistocene) intramorainic lake, that was more recently filled with Holocene alluvia, ending with lacustrine and swampy deposits. All over the area, the phreatic surface is very close to the ground level (the maximum measured depth is 0.80 m).

Preliminary geotechnical investigations, including six continuous dynamic penetration tests with 51 mm cone (DPT), four static penetration tests (CPT) and one borehole, allowed to recognise the following representative sequence:

1. from the ground level to about 1.00 m: very soft peat with sandy silt (Pt) with cone resistance q_c<0.2 MN/m^2, having a very low unit weight (11.0 kN/m^3); mainly normally consolidated and highly compressible;
2. from about 1.00 m to about 4.00 m: medium dense, medium coarse sand with gravel and rare cobbles (SW);
3. from about 4.00 m to 10.00÷13.00 m: soft to medium consistent, sandy and clayey silt (ML), locally showing a laminated structure, typical of a glacial lake environment; normally consolidated;
4. below to depths greater than 15.00 m: very dense, sandy silt with gravel (SM-ML), to be interpreted as a basal till; practically incompressible.

A cross section with the subsoil characteristics obtained is shown in Fig. 1.

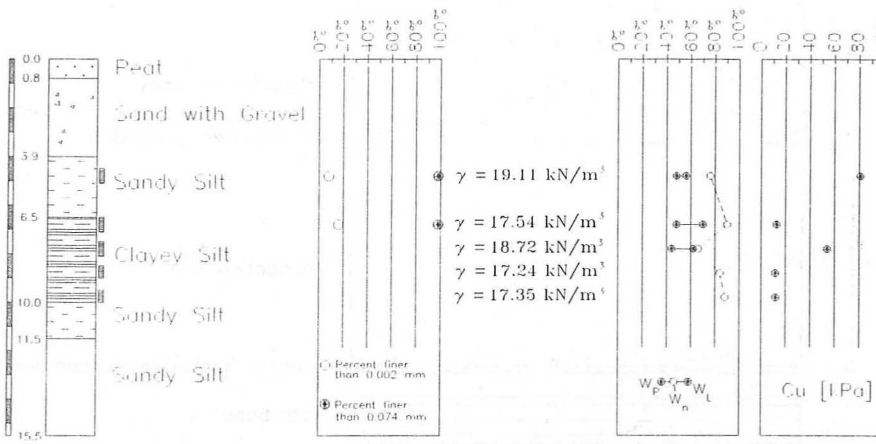

Fig. 1: subsoil characteristics

Nevertheless, during the excavation works for preparing the embankment foundation, the thickness of the peat horizon was found to be randomly varying, even to more than 3.00 m. at some places.

2.2 Construction

The difficulty of the project became evident since the execution of the field tests to determine the sub-soil characteristics. To allow the passage of the machinery for the execution of boreholes and penetration tests, temporary access roads were quickly built by laying down a geogrid-geotextile geocomposite, covered with about 300 mm of granular material.

From the analysis of the results, it appeared that big difficulties could come from the presence of the peat layer and of the high water table. Moreover, the presence of a non homogeneous normally consolidated clay soil was expected to create settlement problems after construction, since the maximum load transmitted by the embankment is equal to 160 kPa. For this reason it has been decided to construct the embankment in steps, in order to allow the primary consolidation of the compressible layer after every load. It was also decided to instrument the embankment to control this settlement. The presence of the water table finally required the creation of a foundation structure, 1.00 m thick, perfectly free draining.

Before going on with the works, the contractor had to install a series of well points all around the excavation area to reduce the water table depth. Since, from the geotechnical investigations, it was found that the peat layer had a non uniform thickness, a further excavation was foreseen in some cases in order to remove completely the peat.

As said before, the foundation layer have been made up of 1.00 m thick perfectly draining gravel. The contractor used a very good material, derived from crushing and milling of concrete debris, classified according to H.R.B. as a A1-a soil, with an index group equal to 0.00.
The big difference in grain size and mechanical characteristics of this soil in respect to the upper layer of the subgrade made it necessary to insert a separation layer (to avoid contamination); the very high transmitted load finally suggested the use of a geogrid-geotextile geocomposite (I) equal to the one used for the access roads. With such geocomposite, in fact, the separation effect can be obtained together with the high tensile resistance and tensile modulus required to reinforce the fill in order to resist to the non uniformity of the subsoil. The foundation structure must have an intrinsic stiffness, able to limit the differential settlements caused by the surcharge over the more compressible subgrade areas. To improve this effect, a second reinforcement layer was designed at the top of the foundation layer; in this case a typical geogrid (II) was used, since no separation effect was necessary. A cross section of the proposed solution is shown in Fig. 2.

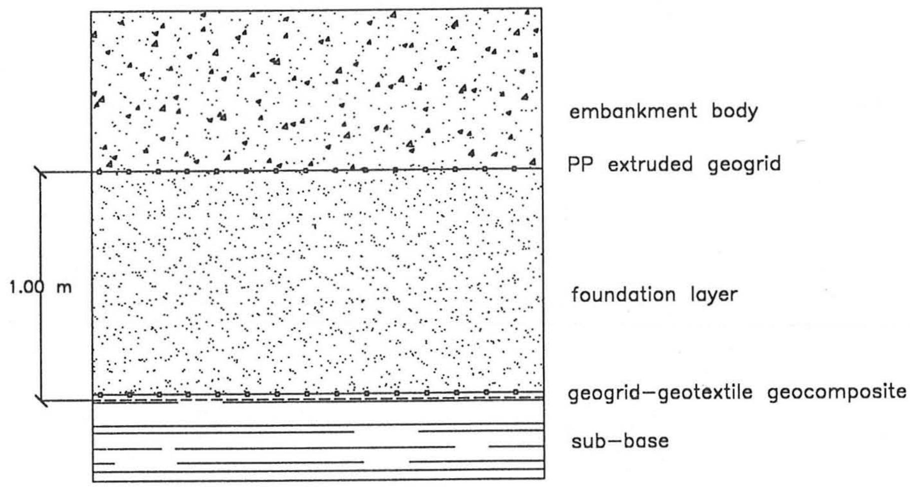

Fig. 2: cross section of the reinforced foundation layer

2.4 Discussion

The geocomposite (I) used was perfectly fulfilling the requirements; in fact:

1. it is produced by thermo-bonding a polypropylene nonwoven geotextile, 140 g/m², to a polypropylene extruded bioriented geogrid. The geotextile acts as a separator, while the geogrid acts as a reinforcement. The particular production process allows very easy installation of the geosynthetic.
2. The geogrid coupled to the geotextile, is characterised by an almost isotropic behaviour; in fact it has a tensile strength of 30.0 kN/m both in longitudinal and transversal direction. This isotropic behaviour ensures the best load distribution and allows the installation of the product in any directions, depending on the need of the contractor.
3. This geocomposite has very high tensile resistance even at low strains; so, it is able to develop its reinforcement action immediately. The geogrid used has a tensile resistance of 10.50 kN/m (both directions) at 2% strain and 21.00 kN/m (both directions) at 5% strain; moreover, the strain at peak is limited to 11.0 % (both directions).
4. The particular production process of the geogrids used, yields a monolithic structure with an uniform distribution of rectangular apertures, with oriented longitudinal and transversal ribs which keep the integrity of the molecular polymer chains for the whole length, even through the junctions. Thanks to this production process, the geogrids used maintain their mechanical properties even after heavy compaction with sharp gravel.
5. A reinforcing geosynthetic could be subjected, during the design life of the work, to a chemically or biologically aggressive environment. Both the geogrid and the geotextile of the geocomposite are

produced from Polypropylene (PP), which is considered as one of the most chemically and biologically inert polymers.

6. A geosynthetic is able to transmit stresses only if it is flat, because it resists to traction only if flat; a deformation of the geosynthetic will cause large settlements at the edges of the geosynthetic before it is tensioned. The geogrids used have a flexural rigidity (measured according to ASTM D 1388) equal to 4.745.000 mg·cm in Machine Direction (MD), 2.969.000 mg·cm in Transversal Direction (TD). This means that the product is able to withstand the weight of the fill material, avoiding its sinking into the subsoil.
7. A bioriented geogrid transmit stresses through friction and interlocking; the apertures (about 40 mm per 27 mm) allows interlocking with soil particles of medium large size. The presence of a geotextile doesn't affect the interlocking, as it is coupled to the geogrid only along the ribs, while it can deform inside the apertures. Obviously the possibility to transmit stresses through interlocking has no meaning if the geogrid junctions have not a good resistance (stresses from longitudinal to transversal ribs transmit through the junction). The geogrids used have a junction resistance equal to 80% of the peak strength (Montanelli e Rimoldi, 1994).

Tab. 1: geosynthetics used

TECHNICAL DATA	TEST METHOD	UNIT	TENAX GT 330 [geocomposite (I)]		TENAX LBO 301 SAMP [geogrid (II)]	
TEST DIRECTION			MD	TD	MD	TD
POLYMER			PP		PP	
PEAK TENSILE STRENGTH	GRI-GG1	kN/m	30.0	30.0	19.5	31.6
YIELD POINT ELONGATION	GRI-GG1	%	11.0	10.0	16.0	11.0
TENSILE STRENGTH AT 2% STRAIN	GRI-GG1	kN/m	10.5	10.5	6.0	10.0
TENSILE STRENGTH AT 5% STRAIN	GRI-GG1	kN/m	21.0	21.0	12.0	20.0
FLEXURAL RIGIDITY	ASTM D 1388	mg · cm	4745000	2969000	1775250	1523500

2.5 Plate bearing tests results

The effect of the insertion of the geocomposite at the base of the foundation layer was demonstrated through a series of plate bearing tests performed in accordance with the Swiss Code SNV 670317a, using a round plate of 300 mm diameter. Tests have been performed with different gravel thickness (300, 600 and 900 mm); the results have been compared with reference sections (without reinforcement) (Fig. 3).

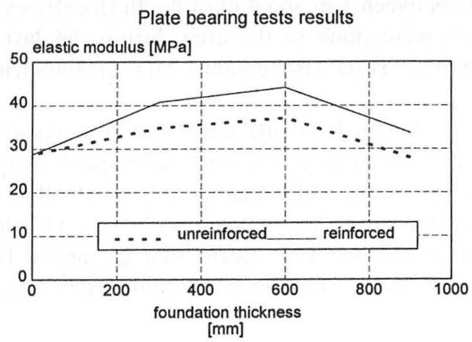

Fig. 3: plate bearing tests results

The first considerations it is possible to do observing the results is the increase in the elastic modulus of the foundation layer with the thickness of gravel, and the lack of significance of the tests performed with a thickness of 900 mm. In fact, the pressures bulb beneath the plate extend to a depth equal to about twice the diameter of the plate. When the plate is placed at a distance of 900 mm above the bottom of the foundation, the result obtained is not the modulus of the foundation, but the modulus of the compacted soil (as if it was not reinforced): in this case, to have a proper evaluation of the modulus, the plate dimension should be larger. The modulus found is anyway greater than the value required by the Italian Code for foundation structures (15 MPa).
From a quantitative point of view, the elastic modulus in the reinforced sections is about 20% higher than the unreinforced modulus, both when the gravel thickness is 300 and 600 mm. This result is surely lower in respect to what has been obtained from laboratory tests on the bearing capacity of reinforced soils; Guido et al (1987) had found a Bearing Capacity Ratio BCR (intended as the ratio between the bearing capacity in a reinforced section divided by the bearing capacity in an unreinforced section) equal to 1.35 with one layer geogrid; Cancelli et al. (1996) have found results even higher. It is anyway important to remember that the elastic modulus of the dry sub-base was 28 MPa.
To do a more accurate analysis of the results it will be necessary to perform a larger number of bearing tests, in order to have a more consistent base for a theoretical approach. In any case, we think that the BCR value of 1.20 obtained so far can be considered in this case a lower boundary, which will be surely increased by the second geogrid layer (placed after execution of the tests above explained).
From a qualitative point of view, the results seem to be much more than satisfying. After the well points have been turned off, and the water table has reached the foundation level, no differential settlement was observed. The material used for the foundation is perfectly draining the water (the phreatic surface crosses perpendicularly the foundation structure), and no contamination seems to have affected the fill soil (demonstrating the fact that the geotextile coupled to the geogrid is perfectly working).

3. BOTTARONE ROAD EMBANKMENT

3.1 Introduction

Under severe geological conditions even a 3 meters high embankment can be subjected to failure or excessive settlements. In Northern Italy, a road embankment had to be built, around the town of Bottarone (Pavia province), to link two main roads, namely S.S.35 and S.P.12.
The embankment had to be no more than 3.0 m high, crossing an area generally covered with an overconsolidated crust, but locally, swampy areas appeared.

3.2 Geological and geotechnical situation

The whole area of Bottarone is part of an ancient meander of river Po. The soils interested by the construction are basically recent fluvial alluvium (gravel, sand, silt and peat). The water table is rather close to the ground level, varying between 1 m and 4 m of depth (locally even less)
Many geotechnical investigations were done in the area during the last 8 years: boreholes, Standard Penetration Tests, Cone Penetration Tests (Begemann) and piezometric measurements. The typical geological sequence was:

1. from the ground level to 1.0 m: from silt to silty clays. Good mechanical properties (cone resistance q_c>1.0 MPa) due to the drying of the surface layers and subsequent overconsolidation.
2. from 0.0÷1.0 m to 6.0÷8.0 m: normally consolidated cohesive soils (silty clays, sometimes organic clays and peat) alternated to sands (fine, loose, often with silt). This level presented highly variable and unpredictable mechanical properties. This means that q_c ranged from quite good values for the sands (1.0÷2.0 MPa) to very low values for the more compressible cohesive soils (down to 0.1÷0.2 MPa).

3. below 6.0 ÷ 8.0 m coarser soils were found (sands with silt, sometimes with gravel) with very good mechanical properties ($q_c \geq 5.0$ MPa).

Common geotechnical correlations led to assess for layer 2 the following strength and deformability characteristics: c_u = 10 kPa or lower, and Oedometer Modulus M even lower than 1.0 MPa.

3.3 Design and construction

The first difficulty in designing the embankment was the unpredictability and the uncertainty on the mechanical properties of the soil located at depth from 0.0 ÷ 1.0 m to 6.0 ÷8.0 m.

Actually, while the subsoil of the first part of the embankment could be reasonably identified with the existing tests (from 1989 to 1986), many uncertainties remained particularly for the last 300 ÷ 400 meters; for this reason three more CPTs were executed in March 1997.

After the interpretation of the results two parts of the embankment were considered as critical, namely:

1. From chainage 125 to 205: height of the embankment from 1.5 to 2.5 m, on clayey silt alternated with fine sands (low strength and medium compressibility).
2. From chainage 880 to 1090: height of the embankment from 1.5 up to more than 3.0 m, on organic soft clay (low strength and high compressibility).

In order to reduce the possibility of punching failure and expected settlements below the embankment, a reinforced foundation structure was designed, according to the following procedure (Fig. 4):

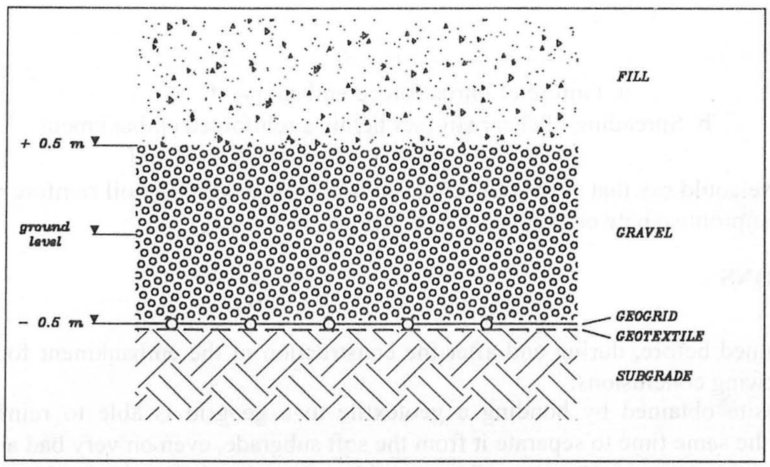

Fig. 4. Layout of foundation structure.

1. Preparation of the sub-base: removing the first 0.50 ÷ 1.00 m of the surface layer and compaction.
2. Placing of a geocomposite (due to practical reasons in this occasion a non-woven geotextile overlaid by a bioriented geogrid (II) were used).
3. Laying of the foundation layer: layered and compacted gravel, about 1 m high.
4. Completing with fill until the required elevation of the embankment.

3.4 Discussion

As already said, the geological conditions caused two kinds of problems to be faced.

Soft layers, between 1.0 to 8.0 meters of depth, could be subjected to large settlements, much more than the embankment could stand (in term of serviceability limit). In the areas covered with crust there could have been concentration of strain followed by undrained shear failure: the abrupt change of deformability parameters of the stiff surface and lower compressible weak layers is often leading to a punching failure (Fig. 5 a).

As in many geotechnical problems, one can improve global stability conditions either by decreasing the applied load, or by improving the mechanical properties of the subgrade, or by redistributing the stresses below the embankment, in order to minimise their concentration.

For this kind of problem the first alternative is not realistic: it means reducing the height of the embankment.
The second approach could have been realised with preconsolidation loading (with or without vertical drains), injections, soil nailing, etc., but it's been judged too expensive in terms of time and money.
The last solution means that if we are able to insert a stiff and tensile-resistant layer between the embankment and the subgrade, then the first settlements below the embankment will mobilise shear stresses along the interface geogrid-subgrade, thus enlarging the reacting surface (Fig. 5 b).

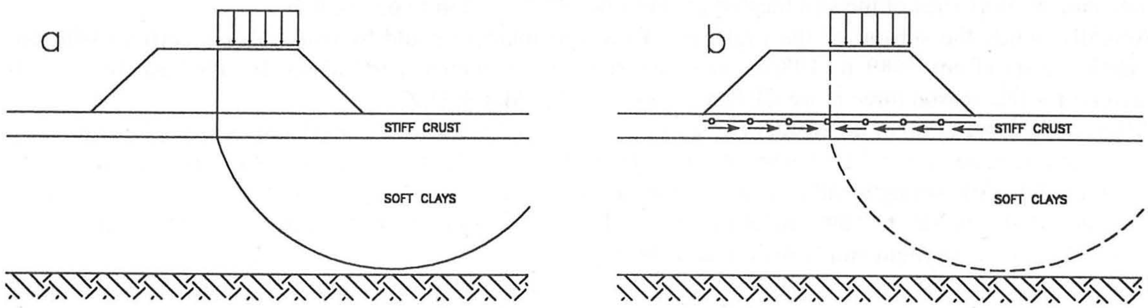

Fig. 5
a. Failure of unreinforced embankment;
b. Spreading of shear stresses below a reinforced embankment.

In other words we could say that the results that can be achieved with the soil reinforcement approach are often the best compromise between costs and efficiency.

4. CONCLUSIONS

The results obtained before, during and after the construction of the embankment foundation structures support the following conclusions:
- A geocomposite obtained by bonding a geotextile to a geogrid is able to reinforce a foundation structure and in the same time to separate it from the soft subgrade, even on very bad and saturated soils.
- The filtration properties of the geotextile were able to prevent the contamination of the fill soil used for the foundation, thus guaranteeing its very good mechanical and hydraulic behaviour for long term.
- The geocomposite allow fast and cheap construction of access roads, even on soils with the water table very close to the ground surface, and even using very thin gravel layers (0.30 m).
- Although fast and easy to interpretate, a plate bearing test is not the best method to quantify the effect of a reinforcement. A proper instrumentation of the structure can surely provide better and more useful results.

5. REFERENCES

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