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THE EXTENSIVE USE OF GEOSYNTHETICS FOR THE ENVIRONMENTAL PROTECTION IN THE CONSTRUCTION OF A STATE-OF-THE-ART LANDFILL IN ITALY

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1. INTRODUCTION

Landfills have been recently recognised as a "social need" by many committees around the world. In fact one of the main problems of the present society is to find the best way to dispose the huge quantity of urban and industrial waste produced every day by individuals and factories.
The possible solutions vary among incineration, chemical and/or biological and/or mechanical treatments, burial in soil, or other special technologies. Whichever the selected solution, at the end of all treatments there is always a fraction of the original waste that remains in the status of marginal or exhausted material.
The final destination of this material is still the disposal in a technologically managed landfill. The task of environmental engineers is the design of landfill systems able to prevent any pollution to the water, the air, and the surrounding fauna end human life.
In densely populated Countries, the first problem to be solved is the location of the landfill, since the NIMBY syndrome (Not In My Back Yard) is often the reason for harsh discussions between adjacent communities.
This social confrontation often leads to locate the landfill not in most geologically and geotechnically suited site, but in marginal areas which finally satisfy all the fighting communities. Then the environmental engineers are challenged by new problems, which often require an interdisciplinary approach, where geologist, geotechnical engineers, chemists, landscape architects and other technicians are involved. More and more geosynthetics are used to solve the problems associated with landfills located in marginal areas.
A typical case was the Teglio landfill which has to serve population of 300.000 person in one of the most beautiful valleys in Northern Italy. The case history here presented is an example of how geosynthetics engineering can solve hard geological and geotechnical problems in landfills.

2. GEORAFT SYSTEM

The landfill has been constructed on the left side of the Adda river, in Valtellina Valley (Northern Italy).
This valley is characterised by the presence of an important alluvial deposit, with different characteristics when below the river (where it is mainly gravel) or at the side of the river (where the alluvial deposit are made of gravel, sand, silty sand with intercalation of silty peat); close to the side of the valley, there is a transition area with slope debris and alluvial fans made of gravel or sandy gravel alluvial deposits.
Due to lack of better available areas, the landfill had to be constructed over an area where all of the previously described situation were encountered.

Preliminary geotechnical investigations, including seven continuous dynamic penetration tests with 51 mm cone (DPT) and six boreholes, were performed to find out which was exactly the better area for the landfill. It was decided to avoid as much as possible to use the marshy area. Just an edge of the landfill was interested by this subsoil.

Apart from the surface peat layers (which were completely removed), there could be the danger of development of differential settlements. It was thus necessary to design a very stiff platform, able to almost eliminate differential settlements thanks to its internal stiffness, thus preventing the upper waterproofing layers from tear or tensile failures. The stiff platform had to be made up of a Georaft, that is a special system composed of bi-oriented geogrids at the base and monoriented geogrids, placed vertically and arranged in triangular pattern to form 1.00 m high cells; these cells are then filled up with crushed gravel (fig. 1).

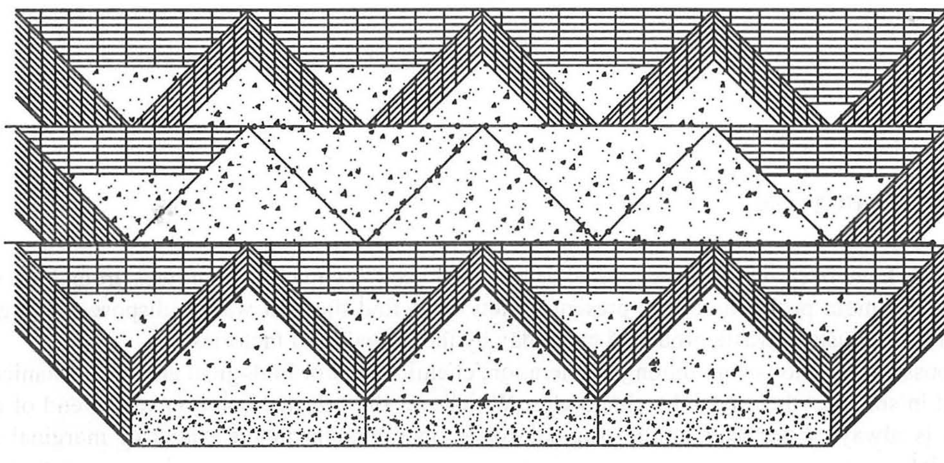

Fig. 1: GEORAFT system

Once the exact area was determined, it was decided to perform a second geotechnical investigation, necessary to find out the mechanical parameters to be used in the calculation of the platform; seven continuous dynamic penetration tests with 51 mm cone (DPT) and two static penetration tests (CPT) were performed; undisturbed sample were brought to laboratory to find out the grain size distribution, unit weight and Atterbergh limits.

The subsoil was found to be mainly sand; the peat was limited to thin intercalation layers, while there were almost no clay layers. The soil, according to U.S.C.S., could be defined mainly as SM (silty sand).

The relative density, determined after the penetration tests (with all the required correction factors), was found to be around 70-80% for the top 9.00 m, and 80-85% deeper. The shear parameter were indirectly obtained from the tests: a value of 34° for the friction angle was found for the 9 upper m; higher values (36°) were obtained for greater depth.

The equivalent undrained cohesion (necessary to perform the stability analysis of the platform), considering the shear force to which the subsoil will be subjected after the landfill is finished, was

$C_{u,eq.} = 65$ kPa

The design method for calculation of a base reinforcement with geogrid mattresses (GEORAFT) is based on Johnson and Mellor (1983) charts on the compression of a block between rigid parallel platens. The starting hypotheses are the following:

- perfectly rough and stiff platform;
- soft soil confined between 2 rigid boundaries;
- cohesive soil with uniform characteristics.

The application of increasing loads causes movement of the plastic yielded zone toward the middle of the foundation; the limit situation is reached when the two symmetrical stress fields meet under the centre of the embankment (fig. 2). The bearing capacity can be found with a slip-line field.

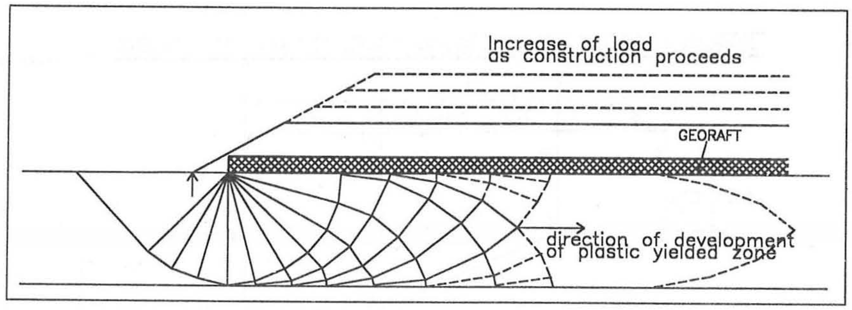

Fig. 2: development of plasticized zone under the Georaft

Tab. 1: design data

whole foundation width	l	70	m
soft soil layer thickness	d	9	m
Undrained cohesion	c_u	65	kPa
Ratio width/thickness	l/d	7.8	

From the chart by Robertson and Gilchrist modified (Bush et al., 1990), choosing the value l/d closer to the real one (see Tab. 1), it is possible to have an idea of the dimension of the "rigid head", that is the area where plasticizing strain will not occur. The value of the bearing capacity beneath the edge is assumed to be equal to $(1+3/2\ \pi)$ Cu. If l/d is greater than 6.00, the value of half the rigid head width is 1.25 d. The ratio between the maximum stress at the border of the rigid head and the undrained cohesion, taken from the chart for l/d = 7.8, is about 9.90 C_u (Fig. 3).

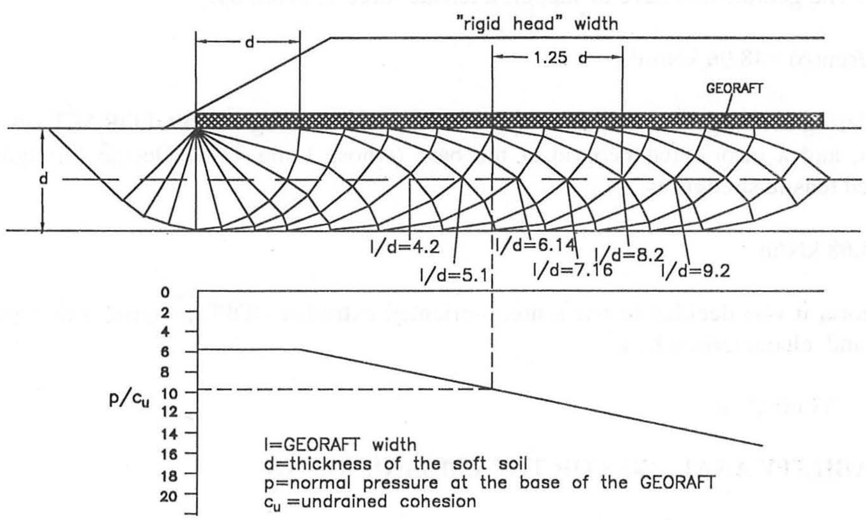

Fig. 3 Design diagram for the Georaft system

In the hypothesis that, inside the rigid head, an increase in the maximum allowable stress equal to the undrained cohesion of the soil can occur, it is possible to draw the envelope of the allowable stress at the base, as shown in fig. 4.

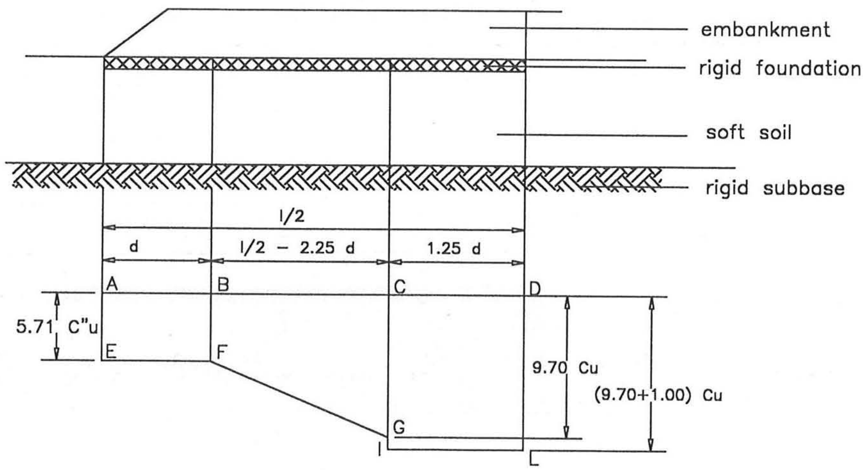

Fig. 4: maximum allowable stresses in the soil under the Georaft system

The maximum vertical stress produced by the embankment weight is: $Q = (q_1 + q_2)\ l/2 = 9100$ kN/m, where:

$\gamma = 11$ kN/m³ (unit weight of wastes)
h = 20 m (maximum expected wastes height)
$q_1 = \gamma \cdot h = 220$ kPa (landfill weight)
$q_2 = 40$ kPa (clay and draining layer)

The base bearing capacity results to be equal to 18794.02 kN/m (area of ABFE + area of BCGF + area of CDLI in fig. 3). The Factor of Safety against foundation failure, calculated as the ratio between the base bearing capacity and Q is equal to 2.065; the required cohesion to have FS = 1.00 is 31.47 kPa. The horizontal stress within the mattress and at the interface between the mattress and the soft layer should be sustained by the geogrids. The friction angle of the coarse gravel within the GEORAFT was assumed to be 40°. The georaft will have to support a tensile force T, given by:

$T = C_u/\mathrm{sen}(\phi) = 48.96$ kN/m².

Considering the long term design strength (LTDS) of the geogrids, a GEORAFT system built in chevron pattern, and a bi-oriented geogrid at the base (whose Long Term Design Strength is neglected), the required tensile strength is:

R = 28.68 kN/m

Therefore, it was decided to use a mono-oriented extruded HDPE geogrid with a peak resistance of 80 kN/m and characterised by a

LTDS = 33.00 kN/m.

3. STABILITY ANALYSIS FOR THE EMBANKMENTS

The front embankments necessary to provide the confinement to the wastes were built using monoriented HDPE geogrids. The mixed construction scheme (Rimoldi, 1997) was used, that is with the main geogrid reinforcement laid straight and a second geogrid wrapped around the face. Movable formworks were used for the construction, in order to avoid any possible damage to the HDPE geomembrane that was used for waterproofing the side slopes. The formworks were made of scaffolding tubes with timber boards: the formworks were placed near the edge of the slope, then a mono-oriented geogrid was placed; a bi-oriented geogrid was placed at the edge of the slope, leaving an edge outside the movable formwork. After laying down and compacting the fill soil, the bi-oriented geogrid was wrapped around the face and then the formwork was extracted (with a back-hoe) and used for the following reinforcement layer (fig. 5). The internal stability analysis was performed according to Jewell's method (Jewell, 1992).

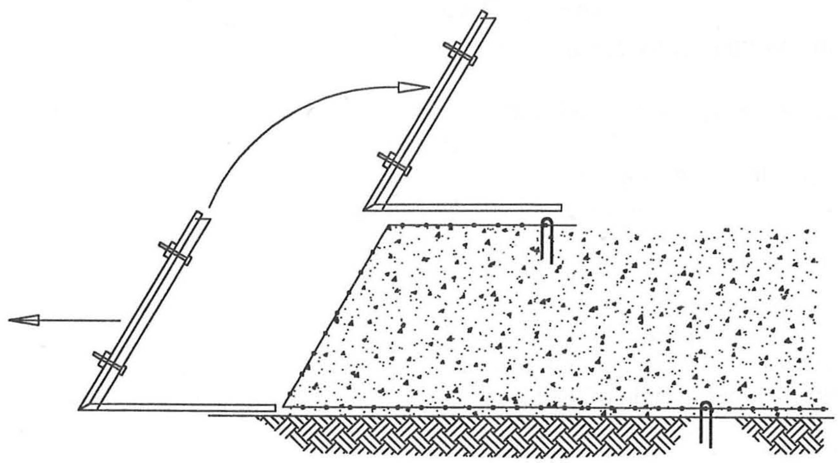

Fig. 5: construction scheme

Apart from the internal stability analysis (that was carried out using the TNXSLOPE, software developed by Tenax), also an external stability analysis was performed, in order to verify whether the reinforced block, although internally stable, was able to withstand the thrust produced by the wastes.
To perform the horizontal sliding analysis it was first of all necessary to simplify the geometry of the landfill, in order to use the Rankine theory for the calculation of the thrust.
The mechanical characteristics of the wastes should then be changed in order to have the possibility to apply the Rankine theory. Obviously the worst situation could occur at the interface between the lower base of the second block (3.00 m high) and the 5.00 m of wastes below it (fig. 6). Using Rankine theory it is not possible to consider the undrained cohesion of wastes. Therefore, in order to consider a real situation, instead of the real values:

$\phi_{wastes} = 27°$
$c_{wastes} = 30$ kPa
$\gamma_{wastes} = 11$ kN/m³

taking into account the fact that the total height of wastes will be $h_{wastes} = 15$ m and that the total vertical stress will be $\sigma_V = 165$ kPa, from Coulomb equation

$\tau = c + \sigma_V \tan\phi$
it is possible to find out an equivalent friction angle

$\phi'_{wastes} = \arctan(\tau / \sigma_V) = 34.66°$.

If it is β = angle between the inner surface of the block and the vertical =22°
i = inclination of the equivalent upper slope = 33°
δ = interface friction between wastes and the reinforced block = 27°

it is possible to find out an active thrust coefficient (according to Rankine theory) equal to

$$K_a = \frac{\cos^2(\phi' - \beta)}{\cos^2\beta \cdot \cos(\beta+\delta) \cdot \left[1 + \sqrt{\frac{\sin(\delta+\phi') \cdot \sin(\phi' - i)}{\cos(\beta+\delta) \cdot \cos(\beta - i)}}\right]} = 0.56$$

The active thrust acting on the block is

$T = 1/2\ \gamma_{wastes}\ K_a\ h^2_{wastes} = 27.96$ kN/m.

This thrust is assumed to be horizontal.
The block geometry is characterised by the following values:

B = 7.2 m = lower base of the block
b = 3.0 m = upper base of the block
h_{block} = 3.0 m = height of each block
γ_{block} = 19.0 kN/m³ = unit weight of the block
W_{block} = 290.7 kN/m = weight of the block

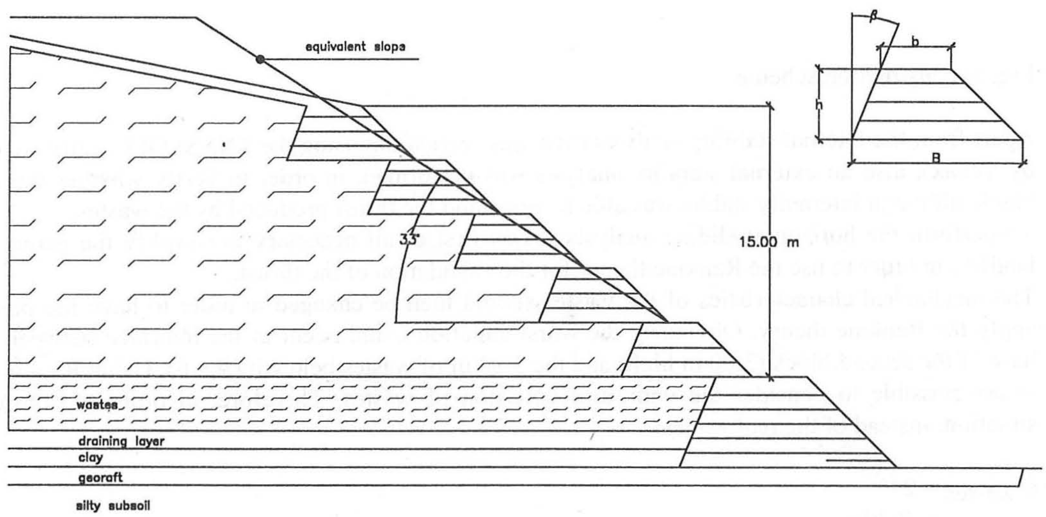

Fig. 6: direct sliding analysis scheme

Evaluating a direct sliding coefficient for the reduction of the friction angle due to the presence of the geogrid f_{ds} = 0.85, it is possible to calculate the resisting force

$R = W\ f_{ds} \tan\phi = 126$ kN/m

The FS against sliding is then FS =4.50

The upper blocks have greater FS, as the resisting action remains the same, while the active thrust reduces.

4. GLOBAL STABILITY ANALYSIS

The global stability analysis have been performed in order to verify the possible presence of circular failure surfaces that could pass beneath the reinforced embankments.

Both Fellenius and Janbu method were used; the check was extended to circular surfaces passing through the toe of the first block and interesting the whole landfill, to circles passing through horizontal surfaces beneath the first embankment, and finally to circular surfaces completely within the reinforced blocks. The tensile resistance of the geogrids have been taken into account imposing an equivalent cohesion of 24 kPa to the reinforced soil; each reinforced block, 3.00 m high, is reinforced with 4 layers of geogrid with a LTDS = 18.3 kN/m; the equivalent cohesion is

$$c_{eq} = 4 \cdot 18.3 \text{ kN/m} / 3.00 \text{ m} = 24.4 \text{ kPa}$$

The tensile resistance of the geomembranes and of the draining geòcomposites were neglected. The minimum overall FS was found equal to 1.80, for circular surfaces passing through the toe of the first block and beneath the reinforced blocks (fig. 7). This value is very satisfactory, most of all considering the fact that the values used for describing the geotechnical behaviour of the wastes are short term values, hence they represent lower limit values of the real properties of wastes.

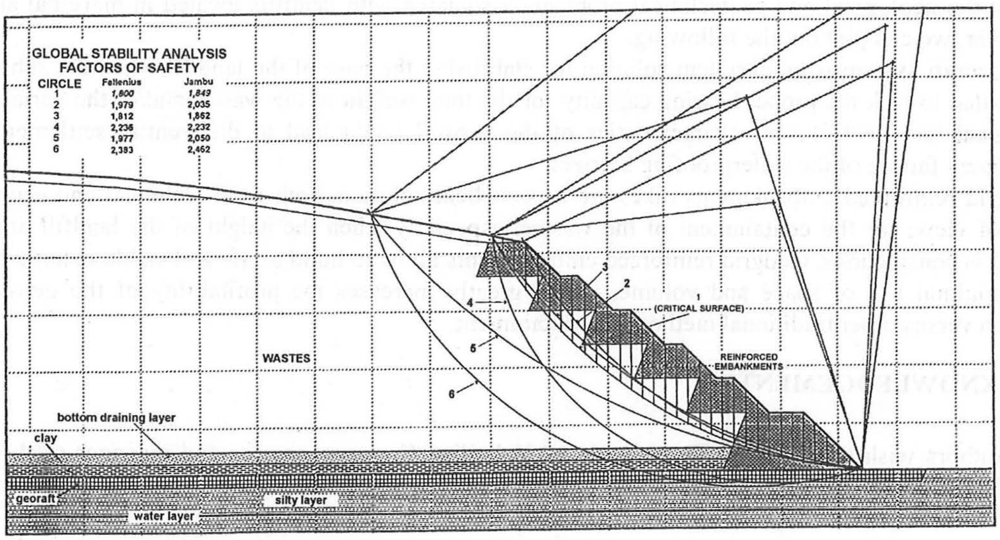

Fig. 7: global stability analysis

5. INSTRUMENTATION

The design of the landfill included a monitoring system for the evaluation of the settlements underneath the primary liner installed at the landfill base. The continuos monitoring and control of the vertical movements at the landfill base allows a safer determination of the actual working conditions and stresses, thus allowing the modification of important parameters such as the landfill height, the filling rate of the landfill, and eventually may allow a future expansion if the settlements are limited. The monitoring system comprises of an hydrostatic profile gauge having a portable probe that can be drawn through an access stiff pipe buried beneath the structure. The probe has an electric pressure transducer inside, filled with gas, and it is connected to the reading device and data logger by a flexible tube filled with a de-aired antifreeze solution. Changes in the level of the probe will result in an hydrostatic pressure change at the transducer which is indicated as an elevation change on the settlement logger. The probe has an internal gas pressure higher than the surrounding environment, thus isolating it from variations in atmospheric pressure. Along the base of each landfill cell, a trench has been excavated to a depth of about 0.5 m along its longitudinal direction from the face of the rim embankment to the toe of the back slope. The length of each trench was ranging between 75 to 90 m. In the trench, two parallel 65 mm diameter 16 atm pressure polyethylene pipes have been installed. At the back, an embedded pulley unit has been connected to both the parallel pipes; inside the pipes and through the pulley unit, a strong PVC coated wire cable is inserted thus allowing the retrieving of the hydrostatic pressure probe from the accessible side of the landfill, outside the rim embankments. A reference point is taken just outside the trench and once the liner and its protective and drainage layers have been installed, a reading is carried out with a horizontal spacing of 1 m for the overall length of the trench. This first reading is consequently compared with the following readings, in order to determine the vertical settlement profile and its rate. Due to the relative low height of the landfill waste at the time of writing this paper, no significant settlements have been recorded yet. Greater settlements are estimated to be recorded when reaching an elevation of about 15 m of waste.

6. CONCLUSIONS

The Teglio Landfill has clearly shown that proper geosynthetic engineering can be fundamental in solving the geological and geotechnical problems associated with landfills located in marginal areas. In particular, we can pint out the following:

- the Georaft system is an excellent solution for stabilising the base of the landfill when the subgrade is anticipated to lack the proper bearing capacity for the total weight of the wastes and/or the variability of geological and geotechnical characteristics of the subsoil could lead to differential settlements with consequent failure of the waterproofing barriers;
- geogrid reinforced embankments dikes are an excellent solution, both from technical and economical point of view, for the containment of the wastes, especially when the height of the landfill above the ground is conspicuous. Geogrid reinforced embankments allow to build a safe and stable external barrier, with minimal use of space and volume, which greatly increases the profitability of the geosynthetic solution versus other traditional methods of containment.

7. ACKNOWLEDGEMENTS

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