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Complete List of Authors:	Moraci, Nicola; Mediterranea University of Reggio Calabria, Busana, Stefano; Environmental consultant Cortellazzo, Giampaolo; Università di Padova, DICEA Favaretti, Marco; University of Padova, Civil, Environmental and Architectural Engineering Mandaglio, Maria Clorinda; Mediterranea University of Reggio Calabria, DICEAM Schepis, Micol; ACQUA & SOLE s.r.l
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Design and construction of a compacted clay liner in the cover system of a MSW landfill using not standard procedures

N. Moraci, *DICEAM, University "Mediterranea" of Reggio Calabria, Italy; email: nicola.moraci@unirc.it*

S. Busana, *Environmental consultant, Vicenza, Italy; email: busana@ordine.ingegneri.vi.it*

G. Cortellazzo, *DICEA, University of Padova, Italy; email: giampaolo.cortellazzo@unipd.it*

M. Favaretti, *DICEA, University of Padova, Italy; email: marco.favaretti@unipd.it*

M.C. Mandaglio, *DICEAM, University "Mediterranea" of Reggio Calabria, Italy; email: linda.mandaglio@unirc.it*

M. Schepis, *ACQUA & SOLE s.r.l, Vellezzo Bellini (Pavia), Italy; email: mcl.sch87@gmail.com*

ABSTRACT: *Design and construction of mineral barrier layer involve many experimental and technological aspects. After having chosen a specific soil water content and laboratory compaction energy, required to obtain permeability values according to the national regulation in force must be determined. It is also necessary to control water content, compaction energy, and permeability of liner actually compacted in situ. The paper shows how a compacted mineral barrier mainly composed of silty clay soil (the excavated soil is a natural but potentially re-usable waste product) was put in place to cover a large MSW landfill and compacted using a heavy dumper, capable to achieve an adequate compaction degree. In situ hydraulic properties of liner were compared to those obtained by laboratory tests and to the limits imposed by the Italian regulation. The actual compaction degree was checked by in situ tests. Hydraulic conductivity tests were carried out in situ, using Boutwell and Guelph permeameters, and in laboratory using rigid wall and flexible wall permeameter. In situ testing provides permeability values more realistic than laboratory ones and demonstrated that the actual construction procedure used was effective in order to obtain the design targets.*

KEYWORDS: MSW landfill, cover system, silty clay, compaction, hydraulic conductivity.

INTRODUCTION

The main function of a MSW (Municipal Solid Waste) landfill cover system is to limit the rainwater infiltration, avoiding an excessive production of leachate. In order to have a cover system of a suitable efficiency, hydraulic conductivity of the mineral barrier shall be determined and controlled with a high accuracy.

The construction of a compacted mineral barrier for covering a wide MSW landfill involves many relevant problems. As the wastes underlying the top barrier are often very compressible, a high specific degree of compaction may be difficult to achieve. Furthermore, climate conditions have a great influence on the actual compaction energy, water content of clayey liner and, consequently, on its hydraulic conductivity.

The hydraulic seal of a MSW landfill is guaranteed by the confining mineral barriers, located up, side and down the waste body. Therefore, the key parameter controlling the efficiency of the barrier is hydraulic conductivity, determined by laboratory and/or in-situ permeability tests. In-situ testing provides permeability values, which should be more realistic than those evaluated in laboratory, since greater volumes of soil can be investigated and the important effects of soil macrostructure can be taken into account.

This study shows how a mineral barrier, composed of a excavated soil that is a natural but potentially re-usable waste product, designed according laboratory Proctor and permeability tests, was used to cover a very large landfill site (~140,000 m^2) for non-hazardous/municipal solid wastes (Figure 1) using a non conventional compaction technology.

The field test site constructed in order to verify the design and construction procedure is located at Grumolo delle Abbadesse (NE Italy) (Figure 2).

Detailed laboratory and in situ experimental investigation was carried out on the geotechnical characteristics of compacted soil, in order to assess the influence of several factors (laboratory compaction energy, water content, in situ compaction technology) on density and permeability of the compacted mineral liners.

The current Italian law (Ministerial Decree 161/2012) considers in specific conditions the excavated soil as a by-product, excluded from waste legislation and usable for embankments, fillings and environmental recompositions. However, low plasticity and high hydraulic conductivity, when compacted in situ, may preclude its re-use in typical environmental work, such as mineral barrier for MSW landfills or contaminated areas.

To compact clayey soils, heavy sheep foot rollers are usually used, but this choice depends on soil nature and on the field compaction equipment available by the contractor. In the present case study, a dumper, having its rear compartment loaded with various weights, was used as compacting equipment.

Density and permeability in situ tests were planned for full definition of the properties of the compacted soil, in order to validate data from previous laboratory tests, carried in the design phase, and to verify whether Italian rules for compaction degree and barrier hydraulic conductivity had been respected.

Figure 1. View of landfill

Figure 2. Site location

MINERAL BARRIERS FOR MSW LANDFILL COVER SYSTEMS

The main functions of the cover system of a MSW landfill are to separate waste from the surrounding environment, minimize water infiltration and collect the biogas produced by waste degradation.

Cover system of a MSW landfill, according to current Italian law (D.Lgs. 36/2003), must be composed of five layers, as follows: a) superficial protective cover layer; b) rain water drainage layer; c) low-hydraulic conductivity compacted mineral layer; d) biogas drainage layer; e) regularization layer. A minimum thickness of 0.5 m and hydraulic conductivity not greater than 10^{-8} m/s are required for low-hydraulic conductivity compacted mineral layer.

Further indications on soil to be used are not provided in the Italian regulation. Ductility and self-healing capability against high differential settlements due to the compressibility of the waste body are requirements that the barrier of cover system should have. Clayey soils show a suitable ductility when the plasticity index is between 10 % and 50%, otherwise clay becomes not enough workable. Moreover, highly plastic soils are difficult to compact, especially in landfill covers. They are sensitive to shrinkage, while drying and wetting cycles cause cracks in the compacted soil and an increase of its hydraulic conductivity.

In order to achieve hydraulic conductivity not greater than 10^{-8} m/s, other geotechnical aspects must also be considered, i.e. passing through ASTM 200 $\geq$ 25%; plasticity index PI = 10% ÷ 50%; gravel percentage $\leq$ 40%; maximum grain size = 25 ÷ 50 mm.

Due to these reasons, first of all a wide campaign of laboratory tests was carried out to investigate the effect of compaction energy and of water content on the the hydraulic conductivity of the compacted excavated soil (waste soil) to be used for the cover system of the Grumolo landfill.

LABORATORY TESTS

The excavated soil has been subjected to the following laboratory tests: grain size analysis; Atterberg limits; X-ray diffractometry; scanning electron microscopy (SEM); reduced, and standard Proctor (AASHTO) compaction tests; falling head permeability test in oedometric apparatus; constant head permeability test in triaxial cell. These tests were performed to determine the water contents and laboratory compaction energies required to obtain permeability values less than those required for mineral barrier by national regulation.

Soil was classified according to the USCS (Unified Soil Classification System) as a medium plasticity inorganic clay (CL). Medium plasticity implies good ductility and workability, and self-healing capability against high differential settlement of waste, which can produce cracks in the barrier and increase its hydraulic conductivity.

Table 1 lists the Atterberg limits and physical characteristics of the test soil, and Table 2 compares the values of passing through ASTM 200 sieve, plasticity index PI, gravel percentage and maximum grain size with those suggested by the Italian guidelines. Figure 3 shows soil grain size distribution.

Table 1. Average index properties and physical characteristics of test material.

Table 2. Geotechnical properties of the test soil

Figure 3. Grain size distribution of the test soil

The classification test results show that the soil is potentially suitable as mineral barrier for the Grumolo cover system. The upper limit of permeability according to Italian regulations is $k < 10^{-8}$ m/s (D.L. 152/2006, D.L. 36/2003); compaction and permeability tests were performed to verify if this soil, compacted at different energy, matches the national regulation requirement.

For this purpose compaction tests have been performed at different compaction energies (standard and reduced AASHTO energies). Moreover, oedometer and triaxial permeability tests have been carried out at the diffent energies and water contents.

The compaction test results are shown in Figure 4 and the pairs of values (ρ_{dmax}; w_{opt}) obtained for reduced and standard Proctor compaction energies are listed in Table 3. The reduced Proctor test (Daniel and Benson, 1990; Benson and Trast, 1995) corresponds to an energy level about half than that achieved with the standard Proctor test (ASTM D 698) and with the in situ compaction procedure.

Figure 4. Proctor curves for different compaction energy

Table 3. Maximum dry density and optimum moisture content (O.M.C.) at reduced and standard Proctor compaction energies.

Optimum moisture content (OMC) is that specific water content at which the soil should ideally be compacted in situ to obtain the maximum densification. Figures 5 and 6 show the trends of hydraulic conductivity versus water content for reduced and standard Proctor compaction energies (Figure 4), obtained in laboratory from falling head permeability tests, by oedometric apparatus, and constant head permeability tests, by triaxial equipment.

Experimental values so obtained were always below the limit imposed by Italian regulation (bold blackline); the test soil compacted at the two different energies is therefore suitable to be used as barrier in the cover system of a MSW landfill.

Figure 5. Trends [k–w] of samples compacted with reduced (w_{opt} = 23%) (5a) and standard Proctor energies (w_{opt} = 21%) (5b): results from falling head and constant head permeability tests.

Figure 6. Trends [k–w] of samples compacted with reduced (w_{opt} = 23%) and standard Proctor (w_{opt} = 21%) energies: results from constant head (6a) and falling head permeability tests (6b).

The results are in good agreement with those obtained by other authors (Benson and Trast 1995) for similar soils with similar index properties and grain size distributions. In particular, it is highlithed that hydraulic conductivity decreases to its minimum value, close to the optimum water content, before it increases again. In addition, similar literature data (Boynton and Daniel 1985) show that the values obtained from constant head permeability tests in triaxial cell were slightly lower (up to one order of magnitude) than those obtained from falling head permeability tests with oedometric apparatus. These differences could be due to different factors such as the sidewall effects in rigid wall permeameter, to the lower void ratios in flexible walls permeameter (triaxial cell) due to the higher confining pressures (isotropic stress state with σ_{cell} = 100 kPa in triaxial apparatus; anisotropic stress state with σ_{axial} = 100 kPa in oedometric apparatus) and to the different values of the gradients used in the different equipment (≈40 in triaxial apparatus and ≈30 in oedometric apparatus).

The hydraulic conductivity values determined by means of laboratory tests represent indicative values, nevertheless the actual values of mineral barrier permeability are those obtained using the in situ construction procedures. This is due to the mixing and compaction techniques used in the laboratory that are more accurate than those used in situ and to the different compaction energies. Mixing procedures for dry and powdered soils are very simple in the laboratory but are much more complicated in situ, where soil is wet and could contain clods.

Therefore, in the second phase of research, a field test site was been realized using the proposed compaction method in order to control the liner water content, compaction energy, density, and permeability obtained in real scale.

Considering the value of optimum water content obtained in laboratory using the different energy (21% for standard energy and 23 % for reduced energy) and related laboratory permeability, it was decided to using water content for field test ranging from 22% to 23,5% in order to evaluate that is on the wet side and also if it is possible using the non usual compaction equipment to reach the desired compaction energy (Figure 4).

FIELD TESTS

Field density and field permeability tests were planned for full definition of the dry density and hydraulic properties of the field compacted soil, in order to validate data from previous laboratory tests carried in the design phase and to verify whether Italian rules for compaction degree and barrier hydraulic conductivity had been respected using the proposed construction procedure.

A dumper, that it is not a usual equipment for field compaction, was used to compact the mineral soil that constituted the test field. Indeed, to compact clayey soils used heavy sheep's foot rollers with feet which penetrates the layer properly are generally used, producing a kind of kneading compaction. The choice of roller type depends on the nature of the soil, and on the availability and requirements of each site.

The Grumolo site operators had a powerful dumper (Figure 7) for large-scale transport of materials, with two axes and six wheels (twin at rear), especially suitable for manoeuvres in restricted spaces. Its weight varied between 300 and 600 kN, according to the load in the rear compartment.

Figure 7. Worksite dumper

To check design requirements of the mineral barrier of the cover system for the field compaction energy, built by means of the dumper, a series of field tests was carried out.

The test soil was initially characterised by high natural moisture content and many aggregated clods. The soil therefore was dried out before tests could take place. This is usually done during spring and summer, and it takes about three days. Subsequentely, clods have been broken up and the soil has been spread evenly.

In order to verify the real field degree of compaction the values of in situ maximum dry density were compared with those obtained in the laboratory for the two different energies.

Indeed, the degree of compaction, Dc, defined as the ratio between the in-situ dry density, $\rho_{d(situ)}$, and the maximum dry density obtained in the laboratory, $\rho_{d(max)}$, for a fixed compaction energy, allows to check the design requirements. In the studied case, the in-situ dry density was evaluated by the sand cone method, shown in Figure 8 (ASTM D 1556). Contract specifications normally require compaction degrees up to 90% of the maximum standard Proctor dry density obtained in laboratory.

Once the design requirements in terms of degree of compaction had been verified, in-situ permeability tests were carried out with Guelph (ASTM D 5126) and Boutwell (ASTM D 6391) permeameters.

Figure 8. Sand-Cone method

Boutwell and Guelph Borehole Permeability Tests

The most important geotechnical parameter for compacted clay used in landfill is the hydraulic conductivity because it controls the barrier efficency. Data available in literature have demonstrated that in situ hydraulic conductivities could be substantially larger than those measured on small laboratory specimens. For this purpose, it is very important to perform field permeability tests to check the real value obtained in situ by the compaction. Among the many different in situ permeability tests, the Boutwell (ASTM D 6391) and Guelph (ASTM D 5126) permeability tests were used to evaluate the permeability of the Grumolo cover system mineral barrier.

Boutwell Borehole permeability test

Boutwell permeability test is a Two-Stage Borehole (TSB) test having a widest acceptance and used all over in the world. The TSB method (ASTM 6391) involves three dimensional infiltration, and both vertical and horizontal hydraulic conductivities can be determined (Figure 9). This test method may be used for compacted fills or natural deposits with a mean hydraulic conductivity not greater than 10^{-5} m/s.

Figure 9. Infiltration path scheme

To perform the test, a hole at least twice larger than the diameter of the casing used for the test is drilled. The hole shall be carefully cleaned and the bottom smoothed, so that the casing can be set firmly on the bottom of the hole on undisturbed soil. A bentonite seal must be formed into the volume within hole and casing (Figure 10).

Figure 10. Boutwell permeameter

The rate of flow of water into soil through the bottom of the casing is determined in one or two stages, normally using a falling hydraulic head procedure. If the soil is considered anisotropic, with different hydraulic conductivity along vertical and horizontal directions, a falling-head test may be carried out in two stages (Figure 9). During Stage 1 the permeant fluid can filter only across the bottom section of the borehole; when the borehole is extended below the bottom of the casing the permeant fluid can filter both vertically and horizontally (Stage 2). The borehole is extended for Stage 2 after Stage 1 is over (Figure 9).

A limiting hydraulic conductivity is computed from the falling head data in both stages (K_1 and K_2). Stages 1 and 2 continue until the limiting conductivity for each stage is relatively constant.

Methods to calculate actual vertical and horizontal hydraulic conductivities (k_v and k_h) from K_1 and K_2, determined during the stages 1 and 2 (Figures 11-12), are described as follows.

Figure 11. Stage 1 of Boutwell permeameter test

STAGE 1

$$K_1 = R_T \cdot G1 \cdot \ln\left(\frac{H1}{H2'}\right)(t_2 - t_1) \qquad (1)$$

where:

- R_T = ratio of kinematic viscosity of permeant at temperature of test permeant during time increment t_1 to t_2 to that of reference fluid and temperature. For most tests, this means water at 20°C (68°F);
- $G1 = (\pi d^2/11D_1)\ [1+a(D_1/4b_1)]$ (cm);
- d = Internal Diameter of standpipe (cm);
- D_1 = effective diameter of Stage 1 (cm), equals Internal Diameter of casing under dry hole conditions when no inward seepage was noted when setting casing, otherwise equals outside diameter of casing;
- a = +1 for impermeable base at b_1,
  = 0 for infinite (greater than $20D_1$) depth of tested material;
  = –1 for permeable base at b_1;
- b_1= thickness of tested layer between bottom of casing and top of underlying stratum (cm).
- H1 = effective head at beginning of time increment (cm), equal to distance from top of water in standpipe to top of underlying stratum or groundwater, whichever is shallower. For calculation purposes, H1 shall not exceed the height of the water column above the bottom of the casing plus 20 test diameters;
- H2' = corrected effective head (cm) at end of time increment, calculated in the same manner as H1, = H2– c;
- c = change in TEG (temperature effect gauge) scale reading between times t_1 and t_2 (cm). An increase in the height of water in the TEG standpipe is positive;
- t_1 = time at beginning of increment(s);
- t_2 = time at end of increment(s).

Figure 12. Stage 2 of Boutwell permeameter test

STAGE 2

$$K_2 = R_T \cdot G2 \cdot \ln\left(\frac{H1}{H2'}\right)(t_2 - t_1) \qquad (2)$$

where:

$G2 = (d^2/16FL)G3$;

$G3 = 2\ln(G4) + a\ \ln(G5)$;

$G4 = L/D + [1+(L/D)^2]^{1/2}$

$$G5=\frac{[4b_2/D+L/D]+[1+(4b_2/D+L/D)^2]^{1/2}}{[4b_2/D-L/D]+[1+(4b_2/D-L/D)^2]^{1/2}}$$

F= 1 – 0.5623 Exp (–1.566 L/D)

and

- L = length of Stage 2 extension below bottom of casing (cm),
- D = Internal Diameter of Stage 2 extension (cm). It shall be equal to the casing ID, and
- b2 = distance from center of Stage 2 extension to top of underlying stratum or groundwater (cm).

Anisotropy of the mineral barrier can be considered by a coefficient of anisotropy m defined as follows:

$$m=\sqrt{k_h/k_v} \tag{3}$$

$$k_v\cdot m=\frac{k_h}{m} \tag{4}$$

Knowing K_1, K_2, L and D, coefficient m can be determined with the following expression:

$$\frac{K_2}{K_1}=m\cdot\frac{\ln\left[\left(\frac{L}{D}\right)+\sqrt{1+\left(\frac{L}{D}\right)^2}\right]}{\ln\left[\left(m\cdot\frac{L}{D}\right)+\sqrt{1+\left(m\cdot\frac{L}{D}\right)^2}\right]} \tag{5}$$

And then k_v and k_h can be found by:

$$K_1=k_v\cdot m=\frac{k_h}{m} \tag{6}$$

During the test the soil suction was controlled by means of tensiometers (Figure 13) in order to measure the suction of the tested layer. If the measured suction is equal to zero, the layer is saturated, while if the suction is greater than zero the soil is unsaturated and the permeability values obtained by the previous equations must be corrected in order to consider the suction.

Figure 13. Tensiometer installed on site

Guelph Borehole Permeability test

The Guelph Permeameter (GP) (Figure 14) is a device designed to determine quickly in-situ hydraulic conductivity of soils. The GP can be moved and used by one operator. Each test requires from 30 to 120 minutes, depending on soil nature, and a few litres of water. Tests, performed on clayey mineral barrier of landfills according to the reference standard (ASTM D 5126), involve a soil thickness ranging from 0,15 to 0,75 m below the ground level. After having excavated a shallow cylindrical hole into the investigated soil, installation of the GP can be made inside it. The GP test employs the Mariotte's bottle principle and is carried out measuring the steady-state rate of water recharge into unsaturated soil from the test hole. The field hydraulic conductivity k_{fs} can be estimated by means of a constant-head test procedure. k_{fs} is referred to the field-saturated bulb (Figure 15) of soil surrounding the test hole.

Figure 14. Guelph permeameter

Figure 15. "Bulb" of saturated soil

Firstly a constant hydraulic head in the hole is fixed and hold at the same level of the lower part of the air tube, located at the centre of the GP. When the water level in the hole starts to drop below the air inlet tip, air bubbles come out from the tip and rise into the tank air space. Vacuum is then partially relieved and the tank provides water to the hole. Size of opening and geometry of the air inlet tip are suitable to control the air bubbles size in order to avoid fluctuations of the well water level. When a water level is maintained constant in the hole, a bulb of saturated soil quickly develops all around, taking a shape depending on soil nature, well radius and imposed hydraulic head.

The bulb shape effect is taken into account in the C factors that appear in the expression for calculating k_{fs}. When a steady-state water flow has been reached the field saturated conductivity k_{fs} can be determined.

A certain amount of air is usually entrapped in the soil voids during the infiltration process, influencing the obtained values of k, which could be lower than obtained in a complete saturation condition. During a measurement, a wetting (but unsaturated) front moves outward from the field-saturated bulb. In this zone, it is possible to evaluate the matric flux potential, which represents the capillarity of the soil. Analysis of steady-state discharge from a cylindrical hole in unsaturated soil, as measured by the GP, accounts for all the forces contributing to three-dimensional flow of water into soils: the hydraulic thrust of water into soil, the gravitational pull of liquid out through the bottom of the well, and the capillary pull of water out of the well into the surrounding soil.

Hydraulic conductivity of saturated soil, k_{fs}, was calculated by means of the equations proposed by Elrick et al. (1989) for two experimental procedures: two-head method (5 and 10 cm), using two tanks at the same time and one-head method (5 or 10 cm), using only the internal or external tank.

Shape factors C_1 and C_2 are a function of soil type, water height in borehole (H) and borehole radius (a). Using the two-head method the factors C must be calculated for each head height H. For one-head method, only C_1 needs to be calculated while for two-head method, C_1 and C_2 are calculated (Zang et al. 1998).

$$C_1 = \left(\frac{H/a}{2{,}074 + 0{,}093 \cdot (H/a)} \right)^{0{,}754} \qquad C_2 = \left(\frac{H/a}{1{,}992 + 0{,}091 \cdot (H/a)} \right)^{0{,}683}$$

One Head Method

$Q_1 = R_1 \cdot 35{,}22$ combined tanks $\qquad Q_1 = R_1 \cdot 2{,}16$ inner tank

$$k_{fs} = \frac{C_1 \cdot Q_1}{2\pi \cdot H_1^2 + \pi \cdot a^2 \cdot C_1 + 2\pi \cdot H_1 / \alpha^*} \qquad (7)$$

Two Head Method

$Q_1 = R_1 \cdot 35{,}22 \qquad Q_2 = R_2 \cdot 35{,}22$ combined tanks $\qquad Q_1 = R_1 \cdot 2{,}16 \qquad Q_2 = R_2 \cdot 2{,}16$ inner tank

$$G_1 = \frac{H_2 \cdot C_1}{\pi \cdot \left(2 \cdot H_1 \cdot H_2 \cdot (H_2 - H_1) + a^2 \cdot (H_1 \cdot C_2 - H_2 \cdot C_1)\right)} \qquad G_2 = \frac{H_1 \cdot C_2}{\pi \cdot \left(2 \cdot H_1 \cdot H_2 \cdot (H_2 - H_1) + a^2 \cdot (H_1 \cdot C_2 - H_2 \cdot C_1)\right)}$$

$$k_{fs} = G_2 \cdot Q_2 - G_1 \cdot Q_1 \qquad (8)$$

where:

- R_1: steady rate of fall of water related to water head H_1
- R_2: steady rate of fall of water related to water head H_2
- Q_1: water flow related to water head H_1
- Q_2: water flow related to water head H_2
- 35,22: tank constant corresponding to the transversal area of the combined tanks (cm^2)
- 2,16: tank constant corresponding to the transversal area of the inner tank (cm^2)
- H_1 : first water head height (cm),
- H_2 : second water head height (cm),
- a : borehole radius (cm)
- α^*: microscopic capillary length factor which is decided according to the soil texture-structure category (It is equal to 0,01 cm^{-1} for compacted clayey or silty materials such as landfill caps and liners)
- C_1: shape factor related to water head H_1
- C_2: shape factor related to water head H_2
- k_{fs}: soil saturated hydraulic conductivity (cm/s).

EXPERIMENTAL FIELD TEST RESULTS

Field density tests

First of all many tests were performed on the field trial to evaluate the possibility to use a dumper for compacting final cover mineral barrier.

Compaction of barriers, located at the top of a landfill, is often difficult having a compressible subbase composing of MSW. This is particularly true when soil water content is high and greater than 3% of the Optimum Moisture Content (OMC). In addition the furrows, left by the dumper wheels, were quite thick (up to 0.15 m from the ground level) and required a suitable levelling to allow the successive layers to be placed and compacted. In-situ density tests carried out with a sand cone test apparatus showed that compaction energy due to the dumper depended on its variable static weight and on the number of passes (Table 4).

Table 4. Sand cone tests results

Increasing the load in the rear compartment of the dumper, the degree of compaction corresponding to the reduced and standard Proctor energies could be achieved without varying the number of dumper passes. In particular, for four passes a dumper weight of 400 kN is necessary to achieve a degree of compaction equal to the 98.7% of the optimum compaction obtained in laboratory with reduced Proctor energy, while the dumper weight must be equal to 450 kN to achieve a D_c equal to 95.1 % of the optimum compaction obtained in laboratory with standard Proctor energy. As previously discussed in situ soil compaction was performed at a moisture content equal to 23%, corresponding to the OMC of the reduced Proctor energy and rather close to OMC of the standard Proctor energy (on the wet side).

Therefore, the proposed compaction procedure can be used to obtain the design target in terms of soil degree of compaction .

For optimal cover operations, the dumper itself was therefore used at 450 kN load, although the loads sometimes had to be limited, to avoid damage to components such as geocomposites under the mineral barrier.

In-situ permeability tests

A further important aspect in the mineral liner design has been to assess its in situ hydraulic conductivity and to compare it with those obtained by laboratory permeability tests using reduced and standard Proctor energies. For this purpose different section of field trial test were constructed using initial water contents ranging from 22% to 23.5%.

The results of in situ permeability tests obtained using Guelph and Boutwell permeameters (Figure 16) on field test are reported respectively in (Table 5) and in Figure 17.

The test results refer to an in situ degree of compaction equal to the 95.1 % of the optimum compaction obtained in laboratory with standard Proctor energy.

Figure 16. Test field during the permeability tests

Table 5. Results of permeability tests carried out with Guelph permeameter.

The values of permeability obtained using the Guelph permeameter range from 10^{-9} to 10^{-8} m/s. This may be due to the relatively small size and the uncompleted saturation of the investigated soil of the bulb involved in seepage, and to the heterogeneity of the mineral barrier. The experimental values obtained by means of Boutwell permeability were always less than the value required by the national regulations (10^{-8} m/s). Figure 17 shows the trend of coefficient k obtanied by Boutwell tests versus water content on site; it can be seen that the permeability decreases with the increase of on site water content and then it becomes steady for w values greater than 22.5 %. This trend is very similar to that obtained with laboratory test. The accuracy of the in situ measurements seems to be very similar to that obtained in laboratory.

The k values are lower than limit imposed by the Italian regulation equal to 10^{-8} [m/s]. Indeed, Boutwell permeameter is one of the most suitable instruments to assess the hydraulic conductivity of a compacted soil. With respect to the Guelph permeameter, installation, testing and data processing are much more complex, but the accuracy of the obtained permeability is greater.

Moreover, comparing the values of permability obtained in situ and in laboratory at the same water content (equal to 22.5%) it is possible to observe that the permeability values are quite similar and that the in situ values are 5 time lesser than those obtained in laboratory by means of triaxal permeability test. This circumstance could be due to the in situ suction and in the soil macrostructure.

On the base of laboratory and field test results, it was decided to construct the actual cover system mineral barrier at a water content equal to 23 % using a dumper of 450 kN adopting for each layer at least 4 passes.

Figure 17. Trend of hydraulic conductivity k_l versus water content

CONCLUSIONS

A wide campaign of laboratory and field investigations were carried out in order to design the mineral barrier of Grumolo delle Abbadesse MSW landfill, using an excavated soil, and a non standard field compaction procedure.

In the first part of the research, the suitability of the excavated soil for the construction of the cover mineral barrier was investigated and the water contents and laboratory compaction energies required to obtain permeability according to the national regulation were determined. Therefore, the excavated soil was used to construct the mineral barrier.

Then it was investigated the efficiency of non standard compaction procedure to put in place the mineral barrier. In particular, water content, compaction energy, and permeability of liner obtained using the not usual compaction method were determined and compared with the degree of compaction and the hydraulic permability obtained in laboratory for different energies.

The in situ effective degree of compaction of the barriers was checked by sand cone tests. Hydraulic conductivity tests were carried out in laboratory using rigid and flexible wall permeameters and in situ by means of Boutwell and Guelph apparatus.

In-situ permeability tests provided more realistic values than those evaluated in laboratory, due to the greater volumes of involved soil and to the important effects of soil macrostructure taken into account.

In situ tests demonstrated that the construction and compaction procedures (i.e. initial water content, breakage of the greater clods) were effective in order to obtain the design targets. In particular, it has been found that to achieve the design requirements four passes of a dumper weighing 450 kN are necessary.

Use of a big dumper to compact the mineral barrier seemed to provide an adequate compaction energy, changing the weight of the rear compartment. Finally the test results highlighted the important aspect of driver training. Workers were familiar with the dumper and quickly acquired the expertise to use it for this new finality, as shown by the good spatial homogeneity of the mineral barrier properties.

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Figure Captions

Figure 1. View of landfill

Figure 2. Site location

Figure 3. Grain size distribution of the test soil

Figure 4. Proctor curves for different compaction energy

Figure 5. Trends [k–w] of samples compacted with reduced (w_{opt} = 23%) (5a) and standard Proctor energies (w_{opt} = 21%) (5b): results from falling head and constant head permeability tests.

Figure 6. Trends [k–w] of samples compacted with reduced (w_{opt} = 23%) and standard Proctor (w_{opt} = 21%) energies: results from constant head (6a) and falling head permeability tests (6b).

Figure 7. Worksite dumper

Figure 8. Sand-Cone method

Figure 9. Infiltration path scheme

Figure 10. Boutwell permeameter

Figure 11. Stage 1 of Boutwell permeameter test

Figure 12. Stage 2 of Boutwell permeameter test

Figure 13. Tensiometer installed on site

Figure 14. Guelph permeameter

Figure 15. "Bulb" of saturated soil

Figure 16. Test field during the permeability tests

Figure 17. Trend of hydraulic conductivity k_1 versus water content

Table Headers

Table 1. Average index properties and physical characteristics of test material.

Table 2. Geotechnical properties of the test soil

Table 3. Maximum dry density and optimum moisture content (O.M.C.) at reduced and standard Proctor compaction energies.

Table 4. Sand cone tests results

Table 5. Results of permeability tests carried out with Guelph permeameter.

Table 1. Average index properties and physical characteristics of test material.

LL (%)	LP (%)	PI (%)	G_s
44	24	20	2.76

Table 2. Geotechnical properties of the test soil

	Passing sieve 0.075 mm (%)	Plasticity index (%)	Gravel percentage (%)	Max. grain size (mm)
Test soil	98	18 ÷ 22	0	0.4
Italian guidelines	≥ 25	10 ÷ 50	≤ 40%	≤ 25 ÷ 50

Table 3. Maximum dry density and optimum moisture content (O.M.C.) of test soil at reduced and standard Proctor compaction energies.

	$\rho_{d,max}$ [Mg/m^3]	w_{opt} (O.M.C.) [%]	$Ec/Ec_{STANDARD}$
Reduced Proctor energy	1,57	23	0.6
Standard Proctor energy	1,63	21	1

Table 4. Sand cone tests results

	Number of passes	Dumper weight [t]	$\rho_{d(situ)}$ [Mg/m^3]	$\rho_{d(max)}$ [Mg/m^3]	D_c [%]	Compliance with design specifications
Reduced Proctor energy (356 kJ/m^3)	4	40	1,55	1,57	98,7	Yes
Standard Proctor energy (593 kJ/m^3)	4	45	1,55	1,63	95,1	Yes

Table 5. Results of permeability tests carried out with Guelph permeameter.

k_{fs} [m/s]
$8,8*10^{-9}$
$2,6*10^{-9}$
$3,5*10^{-8}$
$1,4*10^{-8}$

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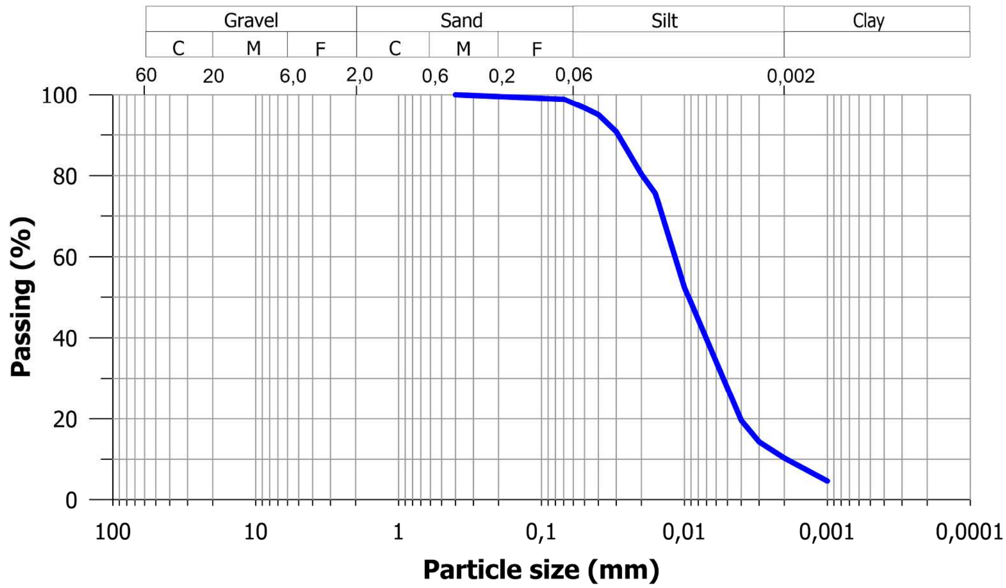

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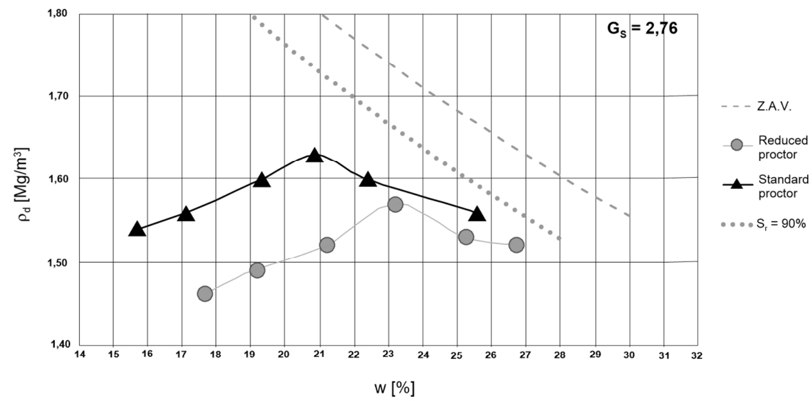

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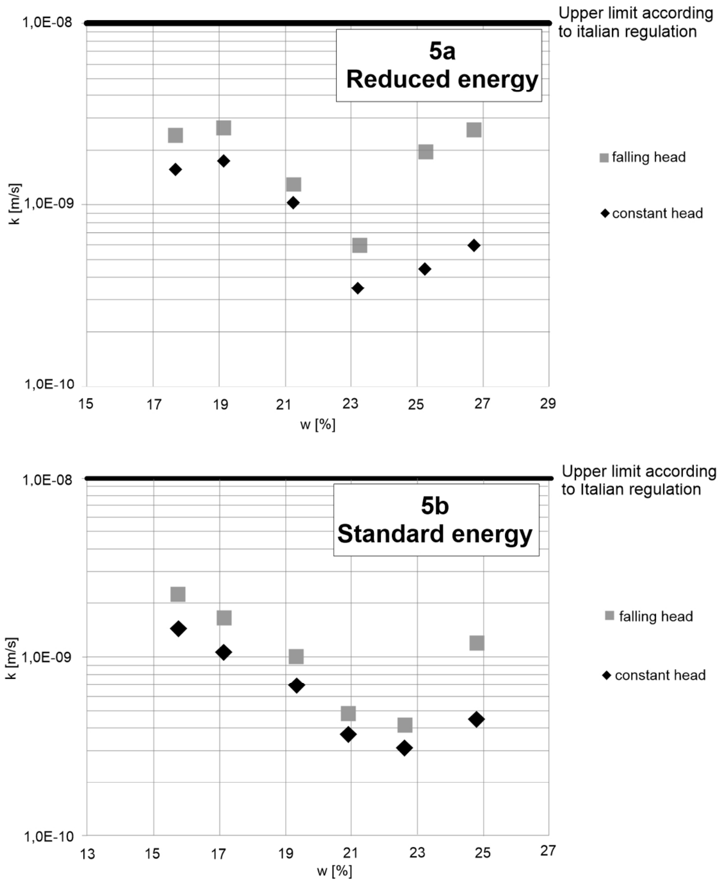

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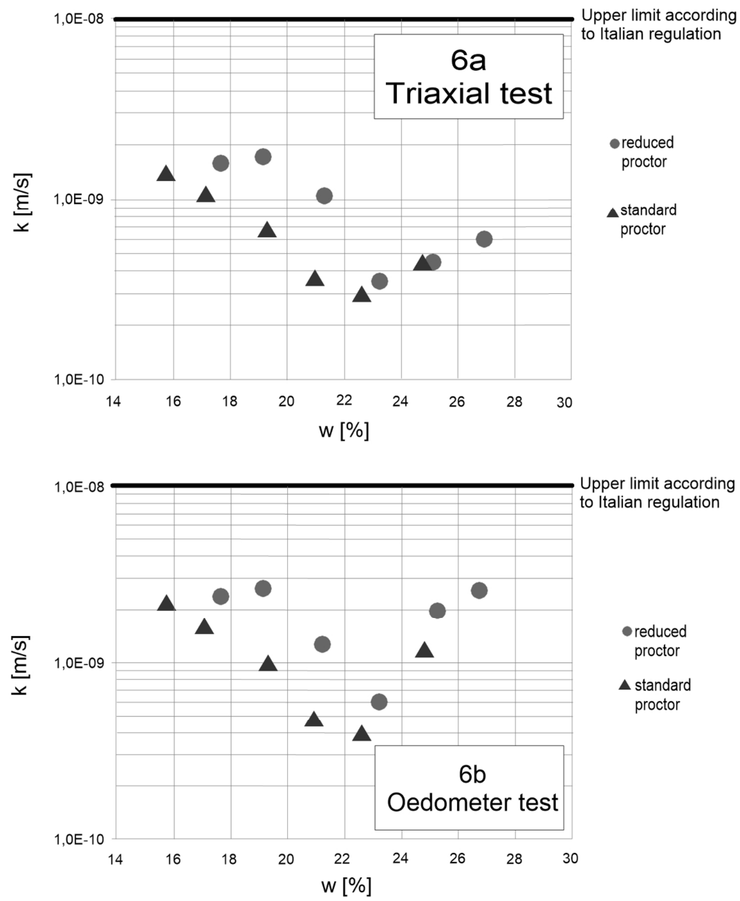

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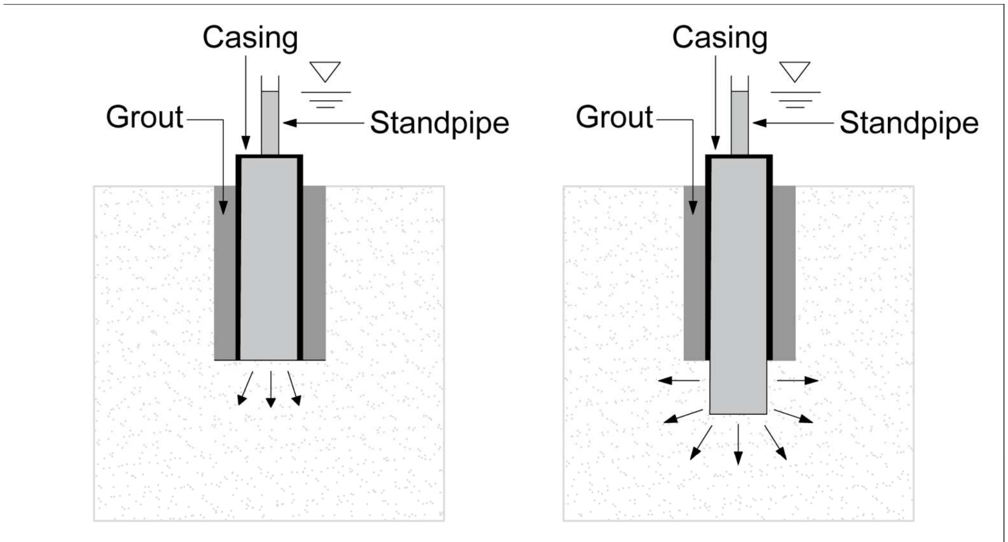

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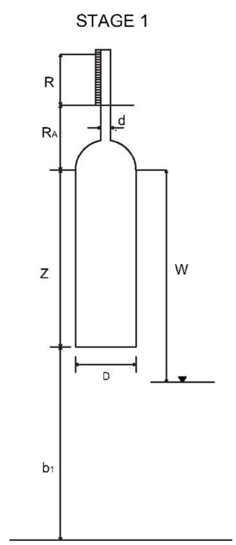

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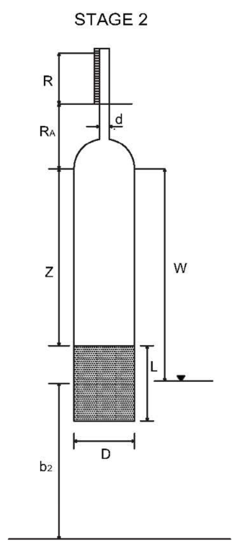

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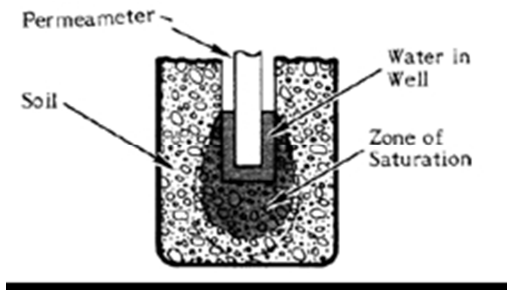

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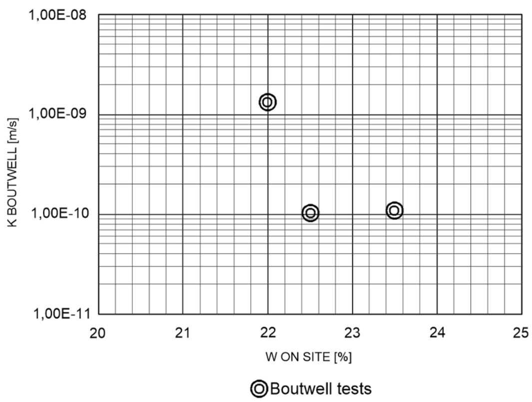

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