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Design and construction of a compacted clay liner in the cover system of a MSW landfill using not standard procedures

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ABSTRACT: *The paper deals with a MSW landfill, located at Grumolo delle Abbadesse (North-Eastern Italy). This landfill was excavated to the depth of -6 m and elevated up to +12 m from the ground level. For pollution migration control a perimetric slurry wall was constructed and keyed into a deep and continuous low permeability layer. The finer part of the excavated soil is used as mineral barrier in the landfill capping system. Design and construction of mineral barrier involve many experimental and technological aspects. After having chosen a specific soil, water content and laboratory compaction energy, required to obtain permeability value according to the national regulation, must be determined. It is also necessary to control water content, compaction energy, and permeability of liner actually compacted in situ. Wastes underlying the capping system of a landfill are very compressible and a specific degree of compaction may be difficult to achieve. Heavy sheep foot rollers are generally used to compact cohesive soils, but this choice may be in contrast with the usual equipment at disposal of contractor. The paper shows how a mineral barrier, mainly composed of silty clay, 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 those obtained in laboratory and demonstrated that the actual construction procedure was effective in order to obtain the design targets.*

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

1 INTRODUCTION

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

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

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

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

17 The field test site, constructed in order to verify the design and construction procedure, is located at Grumolo delle
18 Abbadesse (North-Eastern Italy) (Figure 2).

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

22 A silty clay soil, coming from the area surrounding the Grumolo landfill, was tested in laboratory in order to determine
23 the water contents and laboratory compaction energies required to obtain permeability values less than that required for
24 mineral barrier by national regulation in force. The current Italian law (Ministerial Decree 161/2012) considers in specific
25 conditions the excavated soil as a by-product, excluded from waste legislation and usable for embankments, fillings and
26 environmental recompositions. However, low plasticity and high hydraulic conductivity, when compacted in situ, may
27 preclude its re-use in typical environmental work, such as mineral barrier for MSW landfills or contaminated areas.

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

Field density and field permeability tests were planned for full definition of the dry density and hydraulic 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 using the construction procedure.

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 $PI = 10\% \div 50\%$; gravel percentage $\leq 40\%$; maximum grain size $= 25 \div 50$ 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 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 test were performed in order to determine the water contents and laboratory compaction energies required to obtain permeability values less than that required for mineral barrier by national regulation in force.

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.

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	$\leq 40\%$	$\leq 25 \div 50$

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 energies (standard and reduced energies). Moreover, oedometer and triaxial permeability tests have been carried out for the different energies and the water content.

88 The compaction test results are shown in Figure 4 and the pairs of values ($\rho_{d,max}$; w_{opt}) obtained for reduced and standard
 89 Proctor compaction energies are listed in Table 3. The reduced Proctor test (Daniel and Benson, 1990; Benson and Trast,
 90 1995) corresponds to an energy level about half than that achieved with the standard Proctor test (ASTM D 698) and with
 91 the in situ compaction procedure.

92
 93 *Figure 4. Proctor curves for different compaction energy*

94
 95 *Table 3. Maximum dry density $\rho_{d,max}$ and optimum moisture content (O.M.C.) at reduced and standard Proctor*
 96 *compaction energies.*

	$\rho_{d,max}$	w_{opt} (O.M.C.)	E_c/E_{cSTD}
	[Mg/m³]	[%]	
Reduced Proctor energy	1,57	23	0.6
Standard Proctor energy	1,63	21	1

97
 98 Optimum moisture content (OMC) is that specific water content at which the soil should ideally be compacted in situ to
 99 obtain the maximum densification. Figures 5 and 6 show the trends of hydraulic conductivity versus water content for
 100 reduced and standard Proctor compaction energies (Figure 4), obtained in laboratory from falling head permeability tests,
 101 with oedometric apparatus, and constant head permeability tests, in a triaxial equipment.
 102 Experimental values so obtained were always below the limit imposed by Italian regulation (bold blackline); the test soil
 103 compacted at the two different energies is therefore suitable to be used as barrier in the cover system of a MSW landfill.

105

106

107

108

109 *Figure 5. Trends $[k-w]$ of samples compacted with reduced ($w_{opt} = 23\%$) (5a) and standard Proctor energies ($w_{opt} =$*
110 *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 highlighted 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

122 slightly lower (up to one order of magnitude) than those obtained from falling head permeability tests with oedometric
123 apparatus. These differences could be due to different factors such as the higher sidewall effects in rigid wall
124 permeameter, to the lower void ratios in flexible walls permeameter (triaxial cell) due to the higher confining pressures
125 (isotropic stress state with $\sigma_{\text{cell}} = 100$ kPa in triaxial apparatus; anisotropic stress state with $\sigma_{\text{axial}} = 100$ kPa in oedometric
126 apparatus) and to the different values of the gradients used in the different equipment (≈ 40 in triaxial apparatus and ≈ 30
127 in oedometric apparatus) .

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

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

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

139

140 **FIELD TESTS**

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

145 A dumper, that is not a usual equipment for field compaction, was used to compact the mineral soil that constituted the
146 field test. Indeed, to compact clayey soils heavy sheep's foot rollers with feet which penetrates the layer properly are

147 generally used, producing a kind of kneading compaction. The choice of roller type depends on the nature of the soil, and
148 on the availability and requirements of each site.

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

152

153

154 *Figure 7. Worksite dumper*

155

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

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

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

163 Indeed, the degree of compaction, D_c , defined as the ratio between the in-situ dry density, $\rho_{d(situ)}$, and the maximum dry
164 density obtained in the laboratory, $\rho_{d(max)}$, for a fixed compaction energy, allows to check the design requirements. In the
165 studied case, the in-situ dry density was evaluated by the sand cone method, shown in Figure 8 (ASTM D 1556). Contract

166 specifications normally require compaction degrees up to 90% of the maximum standard Proctor dry density obtained in
167 laboratory.

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

170
171 *Figure 8. Sand-Cone method*
172

173 **Boutwell and Guelph Borehole Permeability Tests**

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

180 ***Boutwell Borehole permeability test***

181 Boutwell permeability test is a Two-Stage Borehole (TSB) test having a widest acceptance and used all over in the world.
182 The TSB method (ASTM 6391) involves three dimensional infiltration, and both vertical and horizontal hydraulic

183 conductivities can be determined (Figure 9). This test method may be used for compacted fills or natural deposits with a
 184 mean hydraulic conductivity not greater than 10^{-5} m/s.

185
 186 *Figure 9. Infiltration path scheme*

187
 188 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
 189 be carefully cleaned and the bottom smoothed, so that the casing can be set firmly on the bottom of the hole on undisturbed
 190 soil. A bentonite seal must be formed into the volume within hole and casing (Figure 10).
 191

192
 193 *Figure 10. Boutwell permeameter*

194
 195 The rate of flow of water into soil through the bottom of the casing is determined in one or two stages, normally using a
 196 falling hydraulic head procedure. If the soil is considered anisotropic, with different hydraulic conductivity along vertical
 197 and horizontal directions, a falling-head test may be carried out in two stages (Figure 9). During Stage 1 the permeant
 198 fluid can filter only across the bottom section of the borehole; when the borehole is extended below the bottom of the

199 casing the permeant fluid can filter both vertically and horizontally (Stage 2). The borehole is extended for Stage 2 after
 200 Stage 1 is over (Figure 9).
 201 A limiting hydraulic conductivity is computed from the falling head data in both stages (K_1 and K_2). Stages 1 and 2
 202 continue until the limiting conductivity for each stage is relatively constant.
 203 Methods to calculate actual vertical and horizontal hydraulic conductivities (k_v and k_h) from K_1 and K_2 , determined during
 204 the stages 1 and 2 (Figures 11-12), are described as follows.

Figure 11. Stage 1 of Boutwell permeameter test

208 **STAGE 1**
$$K_1 = R_T \cdot G1 \cdot \ln\left(\frac{H1}{H2}\right) (t_2 - t_1) \quad (1)$$

209 where:

- 210 • R_T = ratio of kinematic viscosity of permeant at temperature of test permeant during time increment t_1 to t_2 to
 211 that of reference fluid and temperature. For most tests, this means water at 20°C (68°F);
- 212 • $G1 = (\pi d^2 / 11 D_1) [1 + a(D_1 / 4b_1)]$ (cm);
- 213 • 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).
- H_1 = 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, H_1 shall not exceed the height of the water column above the bottom of the casing plus 20 test diameters;
- H_2' = corrected effective head (cm) at end of time increment, calculated in the same manner as H_1 , $= H_2 - c$;
- 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

$$K_2 = R_T \cdot G_2 \cdot \ln\left(\frac{H_1}{H_2}\right) (t_2 - t_1) \quad (2)$$

232

233 where:

$$234 \quad G_2 = (d^2/16FL)G_3;$$

$$235 \quad G_3 = 2\ln(G_4) + a \ln(G_5);$$

$$236 \quad G_4 = L/D + [1 + (L/D)^2]^{1/2}$$

$$237 \quad G_5 = \frac{[4b_2/D + L/D] + [1 + (4b_2/D + L/D)^2]^{1/2}}{[4b_2/D - L/D] + [1 + (4b_2/D - L/D)^2]^{1/2}}$$

$$238 \quad F = 1 - 0.5623 \exp(-1.566 L/D)$$

239 and

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

243

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

$$245 \quad m = \sqrt{k_h/k_v} \quad (3)$$

$$246 \quad k_v \cdot m = \frac{k_h}{m} \quad (4)$$

247 Knowing K₁, K₂, L and D, coefficient m can be determined with the following expression:

$$248 \quad \frac{K_2}{K_1} = m \cdot \frac{\ln\left[\left(\frac{L}{D}\right) + \sqrt{1 + \left(\frac{L}{D}\right)^2}\right]}{\ln\left[\left(m \cdot \frac{L}{D}\right) + \sqrt{1 + \left(m \cdot \frac{L}{D}\right)^2}\right]} \quad (5)$$

249 And then k_v and k_H can be found by:

$$250 \quad K_1 = k_v \cdot m = \frac{k_h}{m} \quad (6)$$

251 During the test the soil suction was controlled by means of tensiometers (Figure 13) in order to measure the suction of
 252 the tested layer. If the measured suction is equal to zero, the layer is saturated, while if the suction is greater than zero

253 the soil is unsaturated and the permeability values obtained by the previous equations must be corrected in order to
254 consider the suction.

255

256

257

Figure 13. Tensiometers installed on site

258

259 **Guelph Borehole Permeability test**

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

266 the test hole. The field hydraulic conductivity k_{fs} can be estimated by means of a constant-head test procedure. k_{fs} is
267 referred to the field-saturated bulb (Figure 15) of soil surrounding the test hole.

268
269

Figure 14. Guelph permeameter

270
271
272

Figure 15. "Bulb" of saturated soil

273 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
274 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
275 the tip and rise into the tank air space. Vacuum is then partially relieved and the tank provides water to the hole. Size of
276 opening and geometry of the air inlet tip are suitable to control the air bubbles size in order to avoid fluctuations of the

277 well water level. When a water level is maintained constant in the hole, a bulb of saturated soil quickly develops all
 278 around, taking a shape depending on soil nature, well radius and imposed hydraulic head.

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

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

288 Hydraulic conductivity of saturated soil, k_{fs} , was calculated by means of the equations proposed by Elrick et al. (1989)
 289 for two experimental procedures: two-head method (5 and 10 cm), using two tanks at the same time and one-head method
 290 (5 or 10 cm), using only the internal or external tank.

291 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-
 292 head method the factors C must be calculated for each head height H. For one-head method, only C_1 needs to be calculated
 293 while for two-head method, C_1 and C_2 are calculated (Zang et al., 1998).

$$294 \quad C_1 = \left(\frac{H/a}{2,074 + 0,093 \cdot (H/a)} \right)^{0,754} \quad C_2 = \left(\frac{H/a}{1,992 + 0,091 \cdot (H/a)} \right)^{0,683}$$

295

296 **One Head Method**

$$297 \quad Q_1 = R_1 \cdot 35,22 \quad \text{combined tanks} \quad Q_1 = R_1 \cdot 2,16 \quad \text{inner tank}$$

$$298 \quad k_{fs} = \frac{C_1 \cdot Q_1}{2\pi \cdot H_1^2 + \pi \cdot a^2 \cdot C_1 + 2\pi \cdot H_1 / \alpha^*} \quad (7)$$

299

300 **Two Head Method**

$$301 \quad Q_1 = R_1 \cdot 35,22 \quad Q_2 = R_2 \cdot 35,22 \quad \text{combined tanks} \quad Q_1 = R_1 \cdot 2,16 \quad Q_2 = R_2 \cdot 2,16 \quad \text{inner tank}$$

$$302 \quad G_1 = \frac{H_2 \cdot C_1}{\pi \cdot (2 \cdot H_1 \cdot H_2 \cdot (H_2 - H_1) + a^2 \cdot (H_1 \cdot C_2 - H_2 \cdot C_1))} \quad G_2 = \frac{H_1 \cdot C_2}{\pi \cdot (2 \cdot H_1 \cdot H_2 \cdot (H_2 - H_1) + a^2 \cdot (H_1 \cdot C_2 - H_2 \cdot C_1))}$$

$$303 \quad k_{fs} = G_2 \cdot Q_2 - G_1 \cdot Q_1 \quad (8)$$

304 where:

- 305 • R_1 : steady rate of fall of water related to water head H_1
- 306 • R_2 : steady rate of fall of water related to water head H_2
- 307 • Q_1 : water flow related to water head H_1
- 308 • Q_2 : water flow related to water head H_2
- 309 • 35,22: tank constant corresponding to the transversal area of the combined tanks (cm^2)
- 310 • 2,16: tank constant corresponding to the transversal area of the inner tank (cm^2)
- 311 • H_1 : first water head height (cm),
- 312 • H_2 : second water head height (cm),
- 313 • a : borehole radius (cm)
- 314 • α^* : microscopic capillary length factor which is decided according to the soil texture-structure category (It is
- 315 equal to $0,01 \text{ cm}^{-1}$ for compacted clayey or silty materials such as landfill caps and liners)
- 316 • C_1 : shape factor related to water head H_1
- 317 • C_2 : shape factor related to water head H_2
- 318 • k_{fs} : soil saturated hydraulic conductivity (cm/s).

319

320 **EXPERIMENTAL FIELD TEST RESULTS**

321 **Field density tests**

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

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

330

331

332

333

Table 4. Sand cone tests results

	Number of passes	Dumper weight [kN]	$\rho_{d(situ)}$ [Mg/m ³]	$\rho_{d(max)}$ [Mg/m ³]	D _c [%]	Compliance with design specifications
Reduced Proctor energy (356 kJ/m ³)	4	400	1,55	1,57	98,7	Yes
Standard Proctor energy (593 kJ/m ³)	4	450	1,55	1,63	95,1	Yes

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 is 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 content ranging from 22% to 23.5%.

The results of in situ permeability test 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.

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

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 bulb involved in seepage, and the heterogeneity of the mineral barrier. The experimental values obtained by means of Boutwell tests were always less than the maximum limit imposed by the national regulations (10^{-8} m/s). Figure 17 shows the trend of the coefficient k , obtained by Boutwell tests, versus the liner water content. 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

374 permeameter, installation, testing and data processing are much more complex, but the accuracy of the obtained
 375 permeability is greater.

376 Moreover, comparing the values of permability obtained in situ and in laboratory at the same water content (equal to
 377 22.5%) it is possible to observe that the permeability values are quite similar and that the in situ values are 5 time lesser
 378 than those obtained in laboratory by means of triaxal permeability test. This circumstance could be due to the in situ
 379 suction and in the soil macrostructure.

380 On the base of laboratory and field test results it was decided to build the real cover system mineral barrier at a water
 381 content eqaul to 23 % using a dumper of 450 kN adopted for each layer at least 4 passes.

382

383 *Figure 17. Trend of hydraulic conductivity k_1 versus water content*

384

385

386 CONCLUSIONS

387 A wide campaign of laboratory and field investigations were carried out in order to design the mineral barrier of Grumolo
 388 delle Abbadesse (Italy) MSW landfill, using non standard field compaction procedure.

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

392 Then it was investigated the efficiency of non standard compaction procedure to put in place the mineral barrier. In
 393 particular water content, compaction energy, and permeability of liner obtained using the not usual compaction method

394 were determined and compared with the degree of compaction and the hydraulic permability obtained in laboratory for
395 different energies.

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

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

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

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

408

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414

415 **REFERENCES**

416 ASTM D 698. Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort.

417 ASTM D 5084 Standard Test Methods for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using
418 a Flexible Wall Permeameter.

419 ASTM D 1556. Standard Test Method for Density and Unit Weight of Soil in Place by the Sand-Cone Method.

420 ASTM D 5126. Standard Guide for Comparison of Field Methods for Determining Hydraulic Conductivity in Vadose
421 Zone.

422 ASTM D 6391. Standard Test Method for Field Measurement of Hydraulic Conductivity Limits of Porous Materials
423 Using Two Stages of Infiltration from a Borehole.

424 Benson C.H., Trast J. M. (1995). Hydraulic conductivity of thirteen compacted clays. *Clays and Clay Minerals*, Vol. 43,
425 No. 6, 669-681.

426 Boutwell G.P., C.N. Tsai. (1992). The Two-Stage field Permeability Test for Clay Liners. *Geotechnical News*. 10:32-34.

427 Boynton, S. S., Daniel D. E., (1985). Hydraulic Conductivity Tests on Compacted Clay. *J. Geotech. Engrg*, Vol. 111,
428 No. 4, 465-478.

429 Daniel D.E. (1989). In-situ Hydraulic Conductivity Tests for Compacted Clay. *J. Geotech. Engrg*.1989.115:1205-1226.

430 Daniel D.E., Benson C.H. (1990). Water content-density criteria for compacted soil liners. *J. Geotech. Engrg*.
431 1990.116:1811-1830.

432 Day S.R., Daniel D.E. (1985). Hydraulic Conductivity of Two Prototype Clay Liners. *J. Geotech. Engrg*. 1985.111: 957-
433 970.

434 Elrick D.E., Reynolds W.D., Tan K.A. (1989). Hydraulic Conductivity Measurements in the Unsaturated Zone Using
435 Improved Well Analyses. *Groundwater Monitoring & Remediation (GWMR)* 1989. 9:184-193.

436 Harrop-Williams K. (1985). Clay liner permeability: evaluation and variation. *J. Geotech. Engrg*. 1985.111:1211-1225.

437 Legislative Decree 36/2003. Attuazione della direttiva 1999/31/CE relativa alle discariche di rifiuti.

438 Ministerial Decree 161/2012. Regolamento recante la disciplina dell'utilizzazione delle terre e rocce da scavo.

439 Osinubi K. 'J., Nwaiwu C.M.O. (2005). Hydraulic Conductivity of Compacted Lateritic Soil. *J. Geotech. Geoenviron*.
440 *Eng*. 2005.131:1034-1041.