

Evaluation of different methods for deriving geotechnical parameters from electric and seismic streamer data

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1 **Evaluation of different methods for deriving geotechnical parameters from electric and**
2 **seismic streamer data.**

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12
13 **Abstract**

14 Geotechnical parameters of linear earth structures, such as embankments and earth dams, are
15 usually obtained from point-wise investigations through drilling or penetration tests, commonly
16 time and cost consuming. Non-invasive geophysical investigations may be considered alternative
17 for a preliminary screening of earth structures physical properties, given their surveying speed and
18 their depth and length of investigation. Seismic and electrical methods can be also used, through
19 specific correlations, for the estimation of geotechnical soil characteristics. Several methodologies
20 have been developed over the years combining two or more geophysical techniques for the
21 estimation of geotechnical parameters.

22 In this paper, three different methods (with theoretical, statistical, and field based approaches
23 respectively) for geotechnical parameters estimation from integrated geophysical surveys were
24 compared, highlighting their strongpoints and limitations also by comparison with available direct
25 geotechnical investigations.

26 Integrated seismic and electrical data from extensive surveying performed over seven retaining
27 structures located in Piedmont Region (NW Italy) were used to forecast their fine content and
28 hydraulic conductivity distributions. Geophysical data were acquired using seismic and electric
29 streamers, useful for the simultaneous execution of the surveys in motion along the earth
30 structures. The results of this study show the effectiveness of the proposed data acquisition
31 approach and elaboration procedures as a first screening tool for earth retaining structure safety
32 assessment. The increased capability of the theoretical method to better predict geotechnical
33 parameters with respect to the other methodologies is also reported.

34
35 **Article Highlights:**

- 36 • different methods for geotechnical parameters estimation from integrated seismic and
37 electrical geophysical surveys were compared;
- 38 • data from extensive surveying performed over seven retaining structures in Piedmont
39 Region (NW Italy) were used to forecast fine content and hydraulic conductivity
40 distributions;
- 41 • strongpoints and limitations of the proposed approaches in the aim of a first screening tool
42 for earth retaining structure safety assessment are discussed.

43 **Keywords:** River embankment, Earth dam, Seismic and electric methods, Geotechnical
44 investigations.

45

46 **1. Introduction**

47 Embankments and earth dams are engineering structures constructed for water supply, energy
48 production or for water flow control in rivers and streams. Their stability and integrity evaluations
49 are an important geotechnical problem for their safety assessment and the prevention from floods
50 and dam-break related risk. Indeed, in the last five decades, these adverse phenomena have
51 generated worldwide significant economic and human losses (Hoyois and Sapir 2003). The
52 reported number of disasters caused by floods has dramatically increased because of climate
53 changes and aging of most of the retaining structures.

54 Stability and integrity of these structures can be compromised by cyclic hydraulic gradients,
55 causing seepage, internal erosion and piping especially when: i) the foundation materials are not
56 sufficiently compacted, ii) heterogeneities are present in the embankment body or iii) the natural
57 aging of the embankment has affected the integrity of some isolated portions. Moreover, localized
58 invasive wildlife activities may negatively affect their hydraulic performances and their structural
59 integrity with burrows excavated in the main embankment body or at the contact with foundation
60 soil. All these phenomena reflect in relevant variations in the geotechnical parameters that need to
61 be properly characterized for assessing the state of health of the structure. Moreover, in
62 correspondence with intense rainfall events, which cause relevant hydraulic gradient variations,
63 the timing of the characterization campaigns can be an important aspect to consider.

64 Consequently, rapid and reliable characterization tools are required for the identification of
65 localized anomalies within the structure bodies. Conventional geotechnical methods for the
66 characterisation involve invasive techniques such as borings (with sample collection for detailed
67 laboratory tests) and penetration tests. These methodologies provide local detailed information of
68 the structure layering but are affected by three main limitations: i) they provide only punctual data
69 and are not sensitive to lateral heterogeneities, ii) they are expensive and iii) time-consuming.

70 On the other hand, non-invasive geophysical techniques allow nearly continuous determination of
71 physical properties that can be helpful in location of anomalies and safety assessment. Given the
72 significant linear extension of protection structures and the localized nature of weakness points,
73 these techniques may be considered a good compromise between the surveying speed, the depth
74 and length of investigation and reliability of the results.

75 Since the soil layering, the variation in water content and the hydraulic conditions have a great
76 influence on the probability of global and local failure, the application of electrical resistivity
77 methods (e.g. Electrical Resistivity Tomography, ERT) and surface wave tests (e.g. Multichannel
78 Analysis of Surface Wave, MASW) are useful tools for linear earth structure characterization.
79 Several applications of these methodologies can be found in literature (e.g. Al-Fares 2014, Arosio
80 et al. 2017, Camarero et al. 2019, Cardarelli et al. 2014, Chen et al. 2006, Comina et al. 2020a,
81 Comina et al. 2020b, Goff et al. 2015, Hayashi et al. 2013, Takahashi et al. 2014, Weller et al.
82 2014, Rittgers et al. 2016). In recent years, the use of mobile geoelectric and seismic systems for
83 a preliminary characterization along river embankment has indeed risen (Brown et al. 2011,
84 Comina et al. 2020a, Comina et al. 2020b, Dabas, 2011, De Domenico et al. 2016, Kuras et al.
85 2007, Sorensen 1996, Vagnon et al. 2021) due to their flexibility and increased surveying speed.

86 In complex geotechnical and hydraulic conditions, and possibly with presence of artefacts (such
87 as metallic diaphragms or drainage pipes), a single geophysical method may lead to
88 misinterpretations. Indeed, ERTs alone cannot distinguish whether low resistivity sectors are due

89 to high water content or clay soil or a buried conduit. Conversely, velocity reductions evidenced
90 by MASW could be associated both to an increase of soil fine fraction content or to an increase of
91 the saturation degree or soil plasticity.

92 Integrated geophysical approaches, combining shear wave velocity (V_s) and resistivity (R), can
93 therefore provide a more accurate description of soil type than the individual methodologies alone
94 (Hayashi et al. 2013). In addition, several researchers have developed theoretical, statistical, or
95 field-based methods for specific geotechnical parameters estimation (soil type, fine fraction
96 content, porosity, hydraulic conditions) from integrated geophysical surveys (Arato et al. 2021,
97 Brovelli and Cassiani 2010, Carcione et al. 2007, Chen et al. 2006, Cosentini and Foti 2014, Glover
98 et al. 2000, Goff et al. 2005, Hashin and Shtrikman 1963, Hayashi et al. 2013, Takahashi et al.
99 2014).

100 In this framework, the present paper report on extensive surveying performed over seven retaining
101 structures located in Piedmont Region (NW Italy) by means of combined ERT and MASW
102 surveys. Both R and V_s data were acquired over the retaining structures by means of appropriate
103 streamers developed for these specific investigations. The geophysical data were used for detecting
104 localized anomalies and estimating the geotechnical parameters with three different methodologies
105 available in literature. Strongpoints and limitation of these methodologies are highlighted and
106 discussed also in comparison with available independent geotechnical data over the same
107 structures.

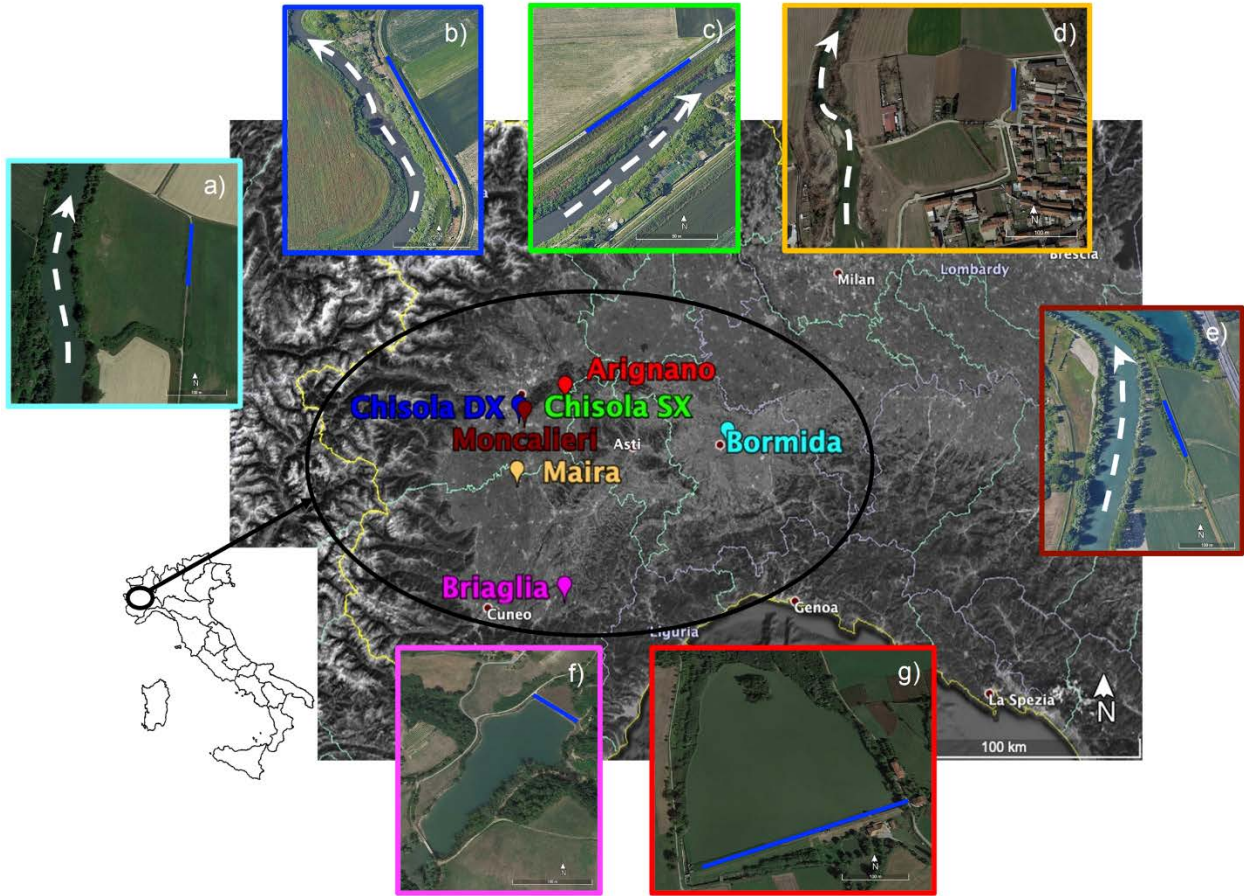
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109 **2. Case studies and data acquisition**

110 Seismic and electric data were collected over seven earth retaining structures located in Piedmont
111 Region (NW Italy): five river embankments (Bormida, Chisola DX and SX, Maira and Moncalieri)
112 and two small earth dams (Arignano and Briaglia). Their geographical location is shown in Fig. 1
113 and their main characteristics are summarized in Table 1 and Fig. 2.

114 These case studies were selected following three main criteria: i) availability of independent
115 geotechnical investigations for comparing and validating geophysical results, ii) coverage of a
116 wide range of construction materials, iii) representativeness of a wide range of structure
117 characteristics. Regarding the last point, the analyzed sites cover different earth retaining structure
118 typologies, characterized by different pathologies. There are two embankments characterized by
119 known anomalies, due to animal burrows (Moncalieri) and rupture restoration works (Chisola SX),
120 one historical embankment subjected to aging phenomena and repeatedly repaired during the time
121 (Bormida), one embankment characterized by a potential seepage phenomenon due to the stress of
122 several flood events (Chisola DX) and one newly built (Maira) but already showing localized
123 instabilities. Finally, two small earth dams were also selected: a historical one with the presence
124 of a brick channel that cross the main body (Arignano) and one built in the 1990s (Briaglia) and
125 affected by aging phenomena.

126 Fig. 2 shows the ternary plot of the average grain size distributions for the embankment bodies and
127 the foundation soils. These data come from point-wise geotechnical investigations performed on
128 each analysed case study: consequently, they refer to an average soil layering and local lateral
129 variations are neglected. As a general comment, embankment bodies are usually made by finer
130 soils (mainly silt and clay with lower percentage of sand) compared to the foundation soils that are
131 generally composed by fluvial deposits with high percentage of gravel and sand and potential
132 presence of rock boulders. The differences between the properties of the main body and foundation
133 soils in earth dams are conversely less marked, especially in the shallow portions (Fig. 2b Arignano
134 and Briaglia markers). A short description of the tested sites is reported in the following.



135
 136 **Figure 1.** Location of the case studies in Piedmont Region (NW Italy): a) Bormida, b) Chisola
 137 DX, c) Chisola SX, d) Maira and e) Mocalieri embankments, f) Arignano and g) Briaglia earth
 138 dams. Blue continuous lines and white dashed arrows respectively represent the geophysical
 139 surveys and the river flow directions.

140
 141 Table 1. Summary of main characteristic of the considered case studies.

Site	Retaining structure type	Average main body height [m]	Survey length [m]	Structural pathologies or potential instability warnings
Bormida	Embankment	5	90	Aging
Chisola DX	Embankment	2.5	114	Stressed by numerous flood events with potential seepage
Chisola SX	Embankment	4	110	Restored after recent flood event
Maira	Embankment	2	76	Newly built with local shallow instabilities
Moncalieri	Embankment	3	126	Presence of localized burrows from wildlife activities
Arignano	Earth dam	8	278	Aging and presence of a brick channel in the main body
Briaglia	Earth dam	11	72	Aging

143 **Bormida River embankment**

144 The right embankment of the Bormida River (44°53'51.16"N, 8°38'46.53"E, Fig. 1a), rises about
145 7 m from the free surface of the river, and about 3 m from the surrounding floodplain. The
146 embankment was repeatedly repaired over years after several flood events that caused local
147 ruptures and instabilities. The soil composition of the embankment consists of silt with fine sand
148 within the first embankment layer and fine to medium-grained sand at the interface with the
149 foundation soil. The latter is mainly made of sand and gravel (Fig. 2).

150

151 **2.1 Chisola DX and SX embankments**

152 The right (DX) and left (SX) embankments of the Chisola River (44°58'43.83"N, 7°40'32.17"E,
153 Fig. 1b and 1c respectively) have a trapezoidal shape with an average height of about 3 m above
154 ground level, a width of about 9 m at the base and of about 4 m at the top. These embankments
155 have been stressed by various flood events during the years, due to intense precipitations and
156 consequent rise of water levels. In the latest event, in November 2016, a localized rupture (about
157 40 m in length) of the left embankment (Chisola SX) occurred, and restoration works were
158 undertaken to seal and repair the embankment. The reconstructed sector of the embankment is
159 mainly constituted of clay and silt, while the surrounding portions and the foundation soils have a
160 high percentage of sand (Fig. 2). The right embankment (Chisola DX) is constituted by natural
161 silty and sandy alluvial deposits taken from the surrounding plain. This embankment was not
162 specifically damaged by previous flood events but, given the damage of the corresponding Chisola
163 SX embankment, the risk of seepage may be hypothesised high.

164

165 **2.2 Maira River embankment**

166 The Maira River embankment (44°46'13.79"N, 7°40'12.48"E, Fig. 1d) is a shallow (about 2 m)
167 newly built embankment to protect the city of Racconigi. This embankment was constructed with
168 selected uniform clayey material directly on the alluvial plain deposits constituted of gravelly sand
169 (Fig. 2). The embankment experienced some landslips along the slopes, caused by the transit of
170 heavy trucks and excavators on the crest road.

171

172 **2.3 Po River (Moncalieri) embankment**

173 The Po River embankment (named here Moncalieri, 44°57'50.48"N, 7°42'7.37"E, Fig. 1e) is 2 m
174 high and was built in the early 20th century to protect the main highway from Torino towards the
175 south. It is built with alluvial sediments (silty sands, Fig. 2) probably exploited from surrounding
176 caves or directly from river deposits. Along this embankment, several badger burrows were
177 detected and considered responsible of several small instabilities.

178

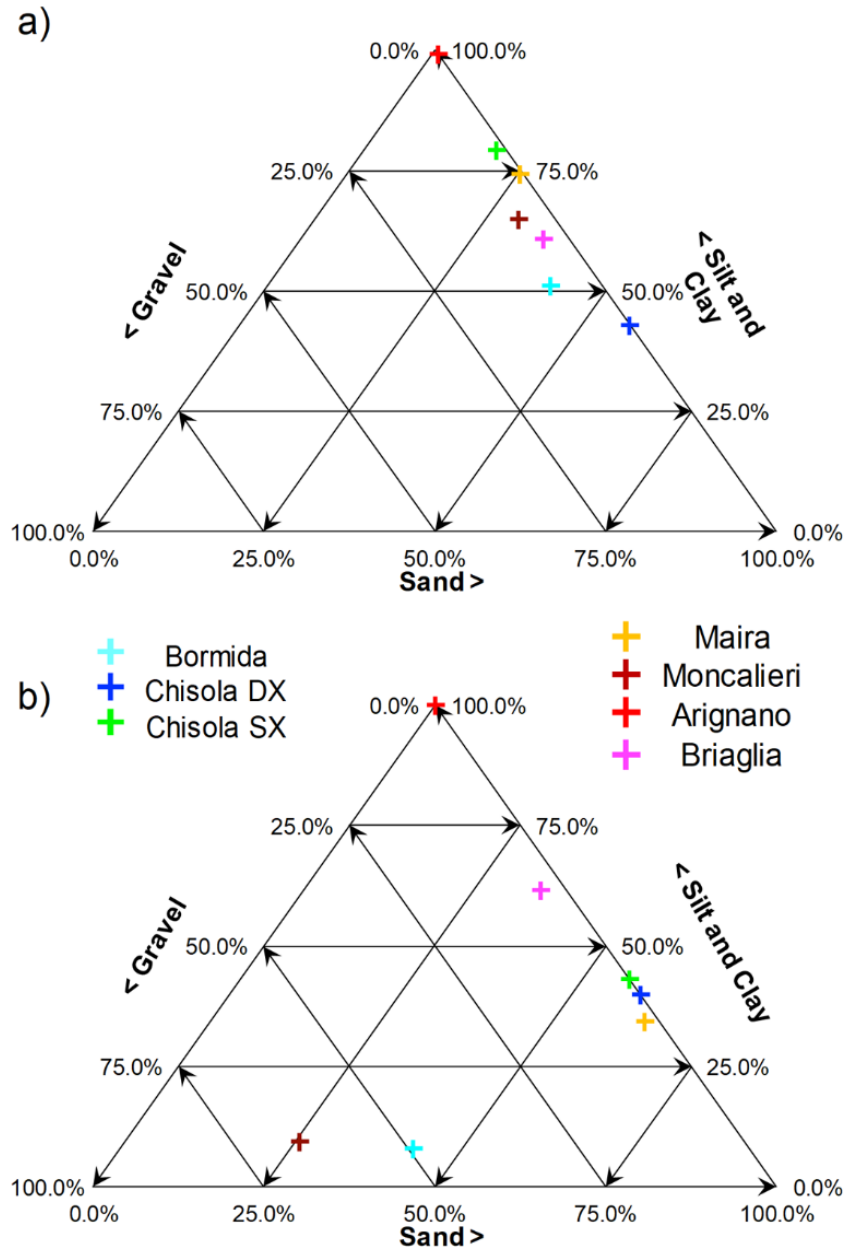
179 **2.4 Arignano dam**

180 The Arignano earth dam (45° 2'40.91"N, 7°53'26.85"E, Fig. 1f) was built at the beginning of 1800s
181 as a water supply reservoir for agricultural purposes. The dam has a trapezoidal shape, with
182 longitudinal extension of about 380 m, maximum height of 8 m and width, at the base, of about 60
183 m, and at the toe of about 4 m.

184 The dam body is mostly made of silt and clay (Fig. 2) and it is founded directly on the natural
185 alluvial soil. The peculiarity of this structure is the presence of a brick channel within the dam
186 body, used in the past for powering the mill located downstream of the dam. This channel, 2 m
187 wide, 1.5 m tall and approximately 20 m long, has warned the authorities on the possibility of
188 inducing preferential seepages and local instabilities.

189 **2.5 Briaglia dam**

190 The Briaglia dam (44°24'10.02"N, 7°53'33.21"E, Fig. 1g) was built at the beginning of 1990s as a
 191 water supply reservoir for agricultural purposes. It has a trapezoidal shape with a spillway and
 192 adequate rockfill on the upstream to protect the dam from the wave flux. The dam has a total length
 193 of about 90 m and a maximum height of about 11 m. The dam body composition varies, from the
 194 embankment crest to the foundation soil interface, between medium-dense sandy silt to silty-
 195 clayey sand. The foundation soil is composed of stiff clay and stiff clayey marl (Fig. 2). The dam
 196 has been monitored in the last years to detect possible aging-related degradation of its geotechnical
 197 performance.
 198



199 **Figure 2.** Ternary plots of the average grain size distributions for a) the embankment bodies and
 200 b) the foundation soils, for each analysed case study.
 201
 202

2.6 Resistivity and shear wave velocity surveys

The surveys over the investigated sites were performed using two different streamers dragged by a vehicle on the top of the retaining structures with data recording at 2 m steps (Fig. 3). For each step, one electric sequence and a single seismic shot were acquired. The data were referred to the respective streamer mid-points and used for integrated interpretation at the same positions. The total survey lengths for each case study are reported in Tab. 1.

The electric streamer consists of 12 electrodes, that can be used both as current and potential electrodes, symmetrically spaced around the streamer mid-point, with a total length of 46 m. The measurement sequence was based on Wenner-Schlumberger and Dipole-Dipole quadrupoles. The electrodes were connected to the acquisition system (Syscal-Pro, Iris Instruments, georesistivimeter), stored on the vehicle, by means of a multipolar cable. For the seismic surveys, an array of 24, 4.5 Hz vertical geophones 1 m spaced was deployed aside to the geoelectrical one and dragged by the same vehicle. A 40 kg accelerated mass was used as a seismic source and located with a 6 m offset from the first geophone. Seismograms were acquired by a DAQ-Link IV seismograph (Seismic Source) with a 0.5 ms sampling interval, -50 ms pretrig and 1.024 s total recording length.

Both electric and seismic acquisitions guaranteed a dense data coverage and a maximum depth of investigation (DOI) of about 10 m (actually the seismic survey DOI is deeper, see Comina et al. 2020b), which is satisfactory for investigating the dam/embankment body and the first meters of foundation soil where the main instability phenomena may occur.

Seismic and electric data were post-processed in office: the electric data were filtered and inverted with the commercial code Res2DInv (Loke and Barker 1996) while the seismic data were analyzed with a specific procedure for the analysis of Rayleigh wave fundamental mode dispersion curves (DC). Further details on the acquisition system and data processing can be found in Comina et al. (2020a, 2020b).

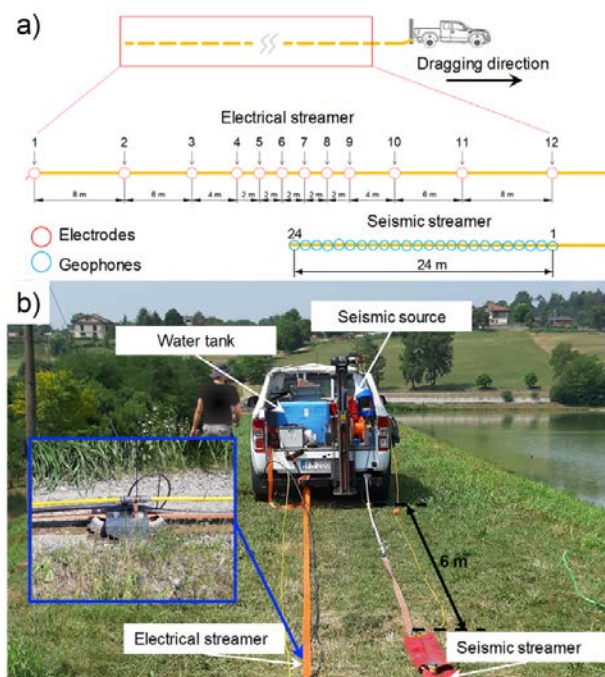


Figure 3. a) Scheme of the electrical and seismic streamers adopted for the characterization. b) Details of the seismic source and acquisition systems.

232 **3. Methodology**

233 In this section, three methods for the estimation of geotechnical parameters from integrated
 234 geophysical data will be analysed. The methods are representative of the main approaches
 235 developed for the characterization of earth linear structures with geophysical data: theoretical,
 236 statistical and field-based approaches. All the three methods have been later applied to the acquired
 237 field data in order to highlight strong points, shortcomings, and possible discrepancies between
 238 predicted results and field evidence.

239 **3.1 Theoretical approach**

241 Takahashi et al. (2014), and later Vagnon et al. (2021), developed an integrated method for
 242 profiling soil permeability of river embankments by coupling seismic and electric data. The clay
 243 content of the soil, C, (assumed as the fine soil fraction i.e. both silt and clay) can be defined from
 244 combined geophysical data by superimposing the experimental electrical resistivity, R, and shear
 245 wave velocity, V_S, values from field measurement to theoretical constant C curves and finding the
 246 nearest C curve to which they can be associated. The theoretical C curves can be derived from the
 247 theoretical V_S-porosity and R-porosity trends, defined from the Glover's model (Glover et al.
 248 2000), the Hashin-Shtrikman upper bound model (Hashin and Shtrikman 1963) and the Voigt-
 249 Reuss-Hill model (Mavko et al. 2009).

250 In detail, the Glover's model expresses the relationship between formation resistivity, R, and
 251 porosity, φ, as follows:

252

$$253 \frac{1}{R} = \frac{1}{R_s} (1 - \phi)^{\frac{\log(1-\phi^m)}{\log(1-\phi)}} + \frac{1}{R_f} \phi^m S_w^q \quad (1)$$

254

255 where R_S and R_f are the soil grains and fluid resistivities respectively , m is the cementation factor,
 256 q is the saturation index and S_w is the saturation degree.

257 The soil grain resistivity, R_S, can be express as a function of the resistivity of the fine soil fraction
 258 (R_{clay}) and its content, C, by using the Hashin-Shtrikman upper bound model:

259

$$260 \frac{1}{R_s} = \frac{1}{R_{clay}} \left[1 - \frac{3(1-C)\Delta R}{R_{clay} - C\Delta R} \right] \quad (2)$$

261

262 with ΔR being the difference between the electrical conductivity of the soil fine fraction, 1/ R_{clay},
 263 and the one of the sand fraction, 1/ R_{sand}, i.e. $\Delta R = \frac{1}{R_{clay}} - \frac{1}{R_{sand}}$.

264

265 The theoretical relationship between V_S and porosity is evaluated by combining the Hashin-
 266 Shtrikman lower bound and the Voigt-Reuss-Hill model as follows:

267

$$268 V_S = \sqrt{\frac{\left(\left(\frac{\phi}{\phi_0} + \frac{1-\phi}{\phi_0} \right)^{-1} - Z \right)}{\rho}} \quad (3)$$

269 with:

270

271

$$Z = \frac{G_{HM}}{6} \cdot \frac{9K_{HM} + 8G_{HM}}{K_{HM} + 2G_{HM}} \quad (4)$$

$$K_{HM} = \left[\frac{n^2(1-\phi)^2 G_g^2}{18\pi^2(1-\nu)^2} P \right]^{\frac{1}{3}} \quad (5)$$

$$G_{HM} = \left[\frac{5-4\nu}{5(2-\nu)} \right] \left[\frac{3n^2(1-\phi)^2 G_g^2}{2\pi^2(1-\nu)^2} P \right]^{\frac{1}{3}} \quad (6)$$

$$G_g = \frac{(1-C)G_{sand} + CG_{clay} + \left(\frac{1-C}{G_{sand}} + \frac{C}{G_{clay}} \right)^{-1}}{2} \quad (7)$$

and where ρ is the bulk density of the soil, G_{HM} and K_{HM} are respectively the shear and bulk moduli of the soil at the critical porosity, ϕ_0 , in the Hertz-Mindlin model (Mavko et al. 2009), n is the coordination number, P is the confining pressure, ν is the Poisson's ratio of the soil, G_{sand} and G_{clay} are respectively the shear moduli of sand and clay components, and G_g is the shear modulus of the soil grains.

These parameters can be assumed based on the wide scientific literature on this topic.

Once the clay content has been obtained, the porosity can be obtained by inverting Equation 1 and R-porosity and Vs-porosity relations can be used for estimating R-Vs relation. The latter can be used to estimate the average grain size, d . The hydraulic conductivity can then be calculated by using Kozeny-Carman relation (Carman 1956):

$$K = 9.8 \cdot 10^6 \cdot \frac{1}{72} \cdot \frac{\phi^3}{(1-\phi)^2 \cdot (1-\ln(\phi^2))} \cdot d^2 \quad (8)$$

Many assumptions are required for the application of this formulation, particularly the value of the clay fraction resistivity, R_{clay} , which has to be calibrated as a function of the specific mineralogy and cation exchange capacity of the clay present at the embankment site. Conversely, the fluid resistivity, R_f , is usually available or can be easily measured independently from samples of the surrounding water. If specifically calibrated with borehole data, this methodology has proven its effectiveness and reliability in profiling earth retaining structures (Takahashi et al. 2014, Vagnon et al. 2021).

3.2 Statistical approach

Hayashi et al. (2013) proposed a polynomial approximation for the estimation of soil parameters, such as fine fraction content (Fc), 20% average grain size (D20), blow counts from standard penetration tests (N_{SPT}) and soil types, by using the cross-plots of shear wave velocity and resistivity.

They collected the results of geophysical surveys performed over 37 Japanese embankments, for a total length of 600 km and correlated them with 400 km of borings. Retaining structures soil was classified into clay, sand and gravel: further distinction was made between foundation soil and embankment body.

The following equation was proposed for the estimation of soil parameters:

$$S_i = aV_S^2 + bV_S + c(\log_{10} R)^2 + d \log_{10} R + eV_S^2 \log_{10} R + fV_S(\log_{10} R)^2 + gV_S \log_{10} R + h \quad (9)$$

312 where S_i is the considered soil parameter (F_c , D_{20} , N_{SPT} and soil type) and a to h are the polynomial
313 coefficients available in Hayashi et al. (2013). These latter were obtained by minimizing the
314 differences between each S_i and the soil parameters obtained from independent geotechnical
315 surveys through a least squares optimization. This formulation is therefore purely empirical, and
316 it is not certain how it can be applied to a broad type of soils.

317

318 **3.3 Field-based approach**

319 Chen et al. (2006) developed a seepage index (F) for assessing potential seepage in the Laocheng
320 embankment (Songzi County, Hubei Province, China) by combining results from surface-wave
321 tests and electric resistivity measurements. F is a dimensionless index defined as:

322

$$323 \quad F = \frac{k_S}{V_S} + \frac{k_R}{R} \quad (10)$$

324

325 where k_S and k_R are empirical coefficients in m/s and Ωm respectively. The index F has both a
326 theoretical and field-based origin. Usually, lower resistivity and shear wave velocity values are
327 correlated with higher moisture content. Moreover, lower shear wave velocity indicates soft soils.
328 Consequently, higher F -values can indicate excessive seepage or piping phenomena.

329 The values of k_S and k_R were calibrated from seismic and electric measurements and on-site
330 characteristics. Indeed, by superimposing V_S and R data on locations where seepage and piping
331 occurred, Chen et al. (2006) observed that F assumed values greater than 2. Consequently, k_S and
332 k_R coefficients were back calculated and set respectively equal to 80 m/s and 5 Ωm . Since their
333 selection is not unique, the authors suggested to determine them by background values (or average
334 values) of shear wave velocity and resistivity through the entire dataset if no drilling data were
335 available. Alternatively, selection of coefficients may be done by comparing with measured V_S
336 and R around seepage areas if such data exist.

337 In this paper, F and k values were compared and the highlighted differences were analysed and
338 discussed with coefficients and soil parameters calibrated on each case study.

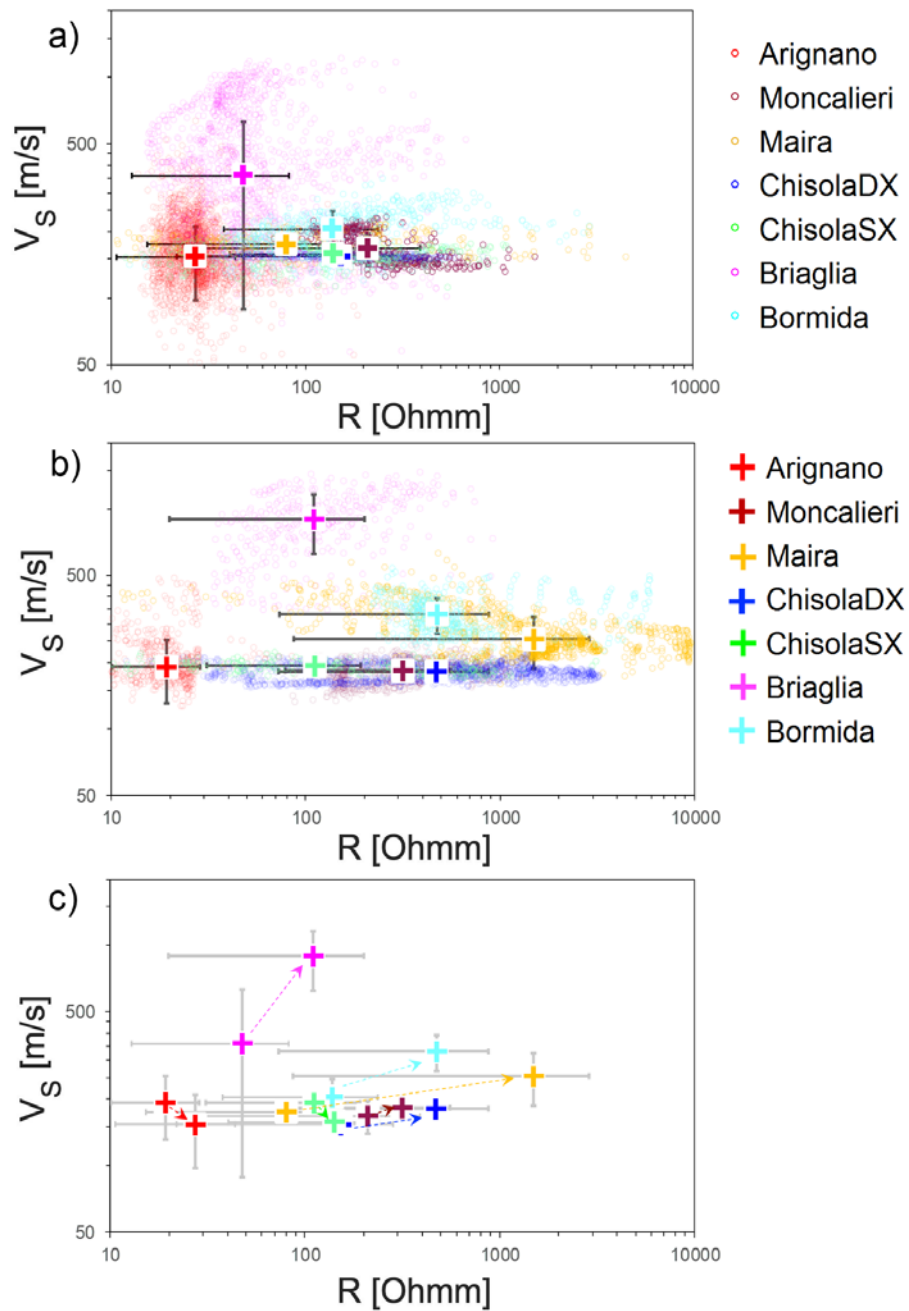
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340 **4. Results**

341 Results of geophysical surveys are shown in Fig. 4. For each case study, V_S - R values along the
342 retaining structures (circle markers) and median values (cross markers) are reported both for the
343 embankment body (Fig. 4a) and for the foundation soil (Fig. 4b). The shift directions between
344 median V_S - R values of embankment body and foundation soils for each analysed structure are also
345 reported (Fig. 4c). For all the investigated structures the constituting soil of the embankment bodies
346 show lower resistivity values than foundation soil (Fig. 4c). These differences are however reduced
347 in some cases (i.e. Arignano, Chisola SX and Moncalieri) due to the reduced contrast among
348 embankment body and foundation soil. In the Arignano and Chisola SX case studies this reduced
349 contrast reflect in a moderate decrease in V_S from embankment body to foundation soil. In all the
350 other structures an increase in V_S from embankment body to foundation soil is observed. This
351 increase is more marked in the Briaglia dam due to the higher stiffness of the constituting
352 foundation soil (stiff clay).

353 At a first sight by analysing Figs 2 and 4, a good correspondence between average grain size
354 distributions and median V_S - R values can be deduced. Generally, by increasing the sand and gravel
355 content of both embankment body and foundation soil, both resistivity and seismic velocity values
356 increase. Indeed the evidenced shifts to higher R values from embankment body to foundation
357 soils (Fig. 4c) is reflected in an increase in sand and gravel content (Fig. 2 a to b). Moreover, the

358 magnitude of the resistivity shift appears proportional to the contrast between the embankment
 359 body and foundation soils.
 360

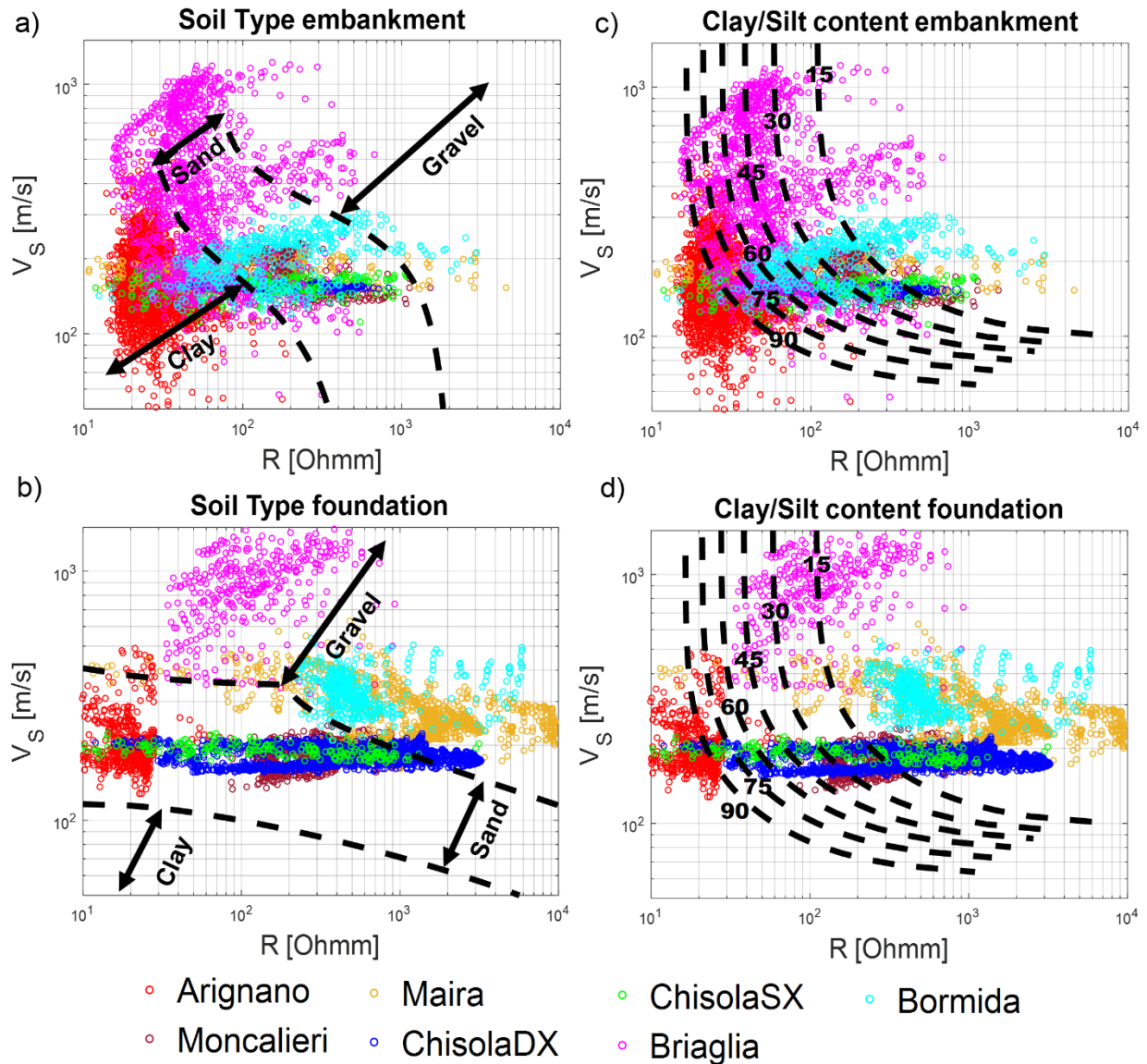


361 **Figure 4.** Distribution of the measured electrical resistivity (R) and shear wave velocity (V_S)
 362 values (coloured circles) in a) embankment bodies and b) foundation soils, for each analysed case
 363 study. Cross markers represent the median values of the distributions, solid lines the corresponding
 364 standard deviation error bars. In a) blue cross marker (Chisola SX embankment) is partially hidden
 365 behind green cross marker (Chisola DX embankment) due to their similar properties. c) Shift
 366 directions (indicated with arrows) between median V_S -R values of embankment body and
 367 foundation soils for each analysed case study.
 368
 369

4.1 Soil Type identification

Theoretical and statistical approaches allow the determination of the soil type. Soil type determination from geophysical data was therefore attempted in the investigated sites with these two methodologies (Fig. 5). With the statistical approach the soil is discretised in three classes: clay, sand and gravel with S_i values (Equation 9) ranging from 1 (clay) to 3 (gravel). In Figure 5a and 5b, the bounds between clay, sand and gravel, defined by the two black dashed lines, are reported. They were drawn by assuming Equation 9 respectively equal to 1.5 (boundary between clay and sand) and 2.5 (boundary between sand and gravel). Analogously, theoretical fine content fraction (C) curves (Figures 5c and 5d) were drawn following the methodology described in Section 3.1, assuming the clay resistivity, R_{clay} , as the minimum measured resistivity value for the given dataset and the fluid resistivity, R_f , on the basis of a priori information. The fine content fraction (C) doesn't provide by itself a clear identification of the soil type: however, many classifications available in scientific literature, are based (among other geotechnical parameters) on this parameter. As an example, the standard UNI EN ISO 14688-1:2018 (CEN 2018) identifies the fine content equals to 35% as the boundary between clayey sand and silt. From 35% up to 100%, the soil is classified into soft silt, soft clay, stiff clay and organic clay. The recommended soil for embankment construction falls into this group. By decreasing the fine content, clayey and silty sand, fine sand and gravel can be identified.

Cross-plots of R and V_s superimposed on the above defined limiting curves show that for both the analysed approaches, R - V_s values for embankment body (Figures 5a and 5c) mainly fall into the sand-clay domain. Conversely, foundation soils (Figures 5b and 5d) are classified as sand and gravel. The statistical approach tends to partially overestimate the soil type granulometry especially in foundation soils (Figures 5b and 5d) compared to the theoretical one. As an example, the foundation soil of Arignano earth dam, that is totally constituted of clay (Figure 2), was predicted to be sand. Similarly, constituting soil of Briaglia earth dam foundation was predicted to be gravel instead of clayey sand.

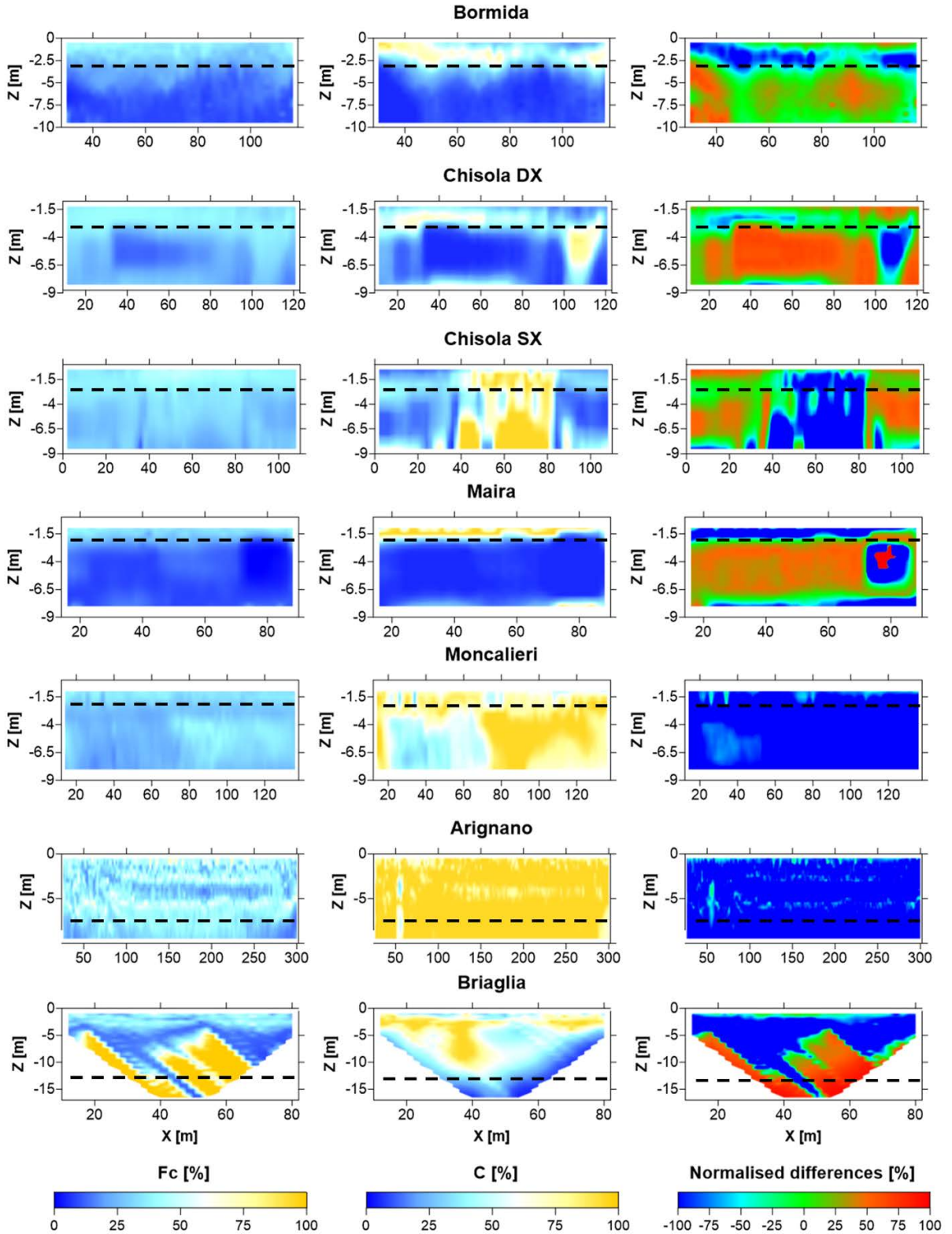


397
 398 **Figure 5.** Soil classification as a function of shear wave velocity (V_s) and electrical resistivity (R)
 399 values based on a-b) Hayashi et al. (2013) approach and c-d) theoretical approach (Takahashi et
 400 al. 2014; Vagnon et al. 2021) for embankment bodies and foundation soils. In all the plots both the
 401 limits among different soil types from the proposed formulations (black dashed lines) and the
 402 experimental data measured in each test sites (coloured circles) are reported.

403
 404 In order to quantitatively evaluate the differences between the two methods and evaluate the
 405 reliability in forecasting soil characteristics, the distributions of the fine fraction contents F_c and
 406 C derived by the statistical and theoretical methods, respectively, were evaluated along the
 407 longitudinal sections of each case study (Fig. 6). Normalised differences, defined as the ratio of
 408 the F_c - C difference to F_c , were also evaluated.

409 The two methods provide analogous results when the constituting soil is coarser and the percentage
 410 of sand and gravel is significant (Chisola SX, Chisola DX and Bormida embankment bodies and
 411 Maira and Bormida foundations, see also Fig. 2). Conversely, in embankments mainly constituted
 412 by clays and silts, the statistical approach generally underestimates the fine content. For instance,

413 analysing the data from the Arignano earth dam, F_c reaches values up to 60-70%, significantly
414 smaller than those obtained by average grain size distributions (Fig. 2). The same considerations
415 can be made for Moncalieri and Maira embankments where fine fraction reaches 75%: barring the
416 first meter depth where the presence of road surfacing, with coarser soil, is well identified, the
417 clayey and silty bodies are not satisfactorily recognized by this methodology. Moreover, the
418 method is not sensitive to sharp soil variations. By focusing on Chisola SX embankment, the
419 statistical approach forecasts a uniform F_c distribution, which is not representative of the real
420 setting of the embankment since the soil in correspondence of the rebuilt sector (between 40 to 80
421 m) is more clayey than the surrounding original embankment body.
422 Conversely, the theoretical approach is more versatile and faithfully forecasts the observed soil
423 distributions. Sharp variations, both vertically, between embankment body and foundations and
424 longitudinally, within the main bodies, are satisfactorily reproduced. Moreover, there is a general
425 better correspondence among the observed C values and the ones expected on the basis of the
426 geotechnical surveys.
427 The predicting capability of the two previous approaches was quantitatively evaluated by
428 comparing the predicted F_c and C results with available grain size distributions performed on
429 borehole logs. Results are listed in Table 2. Local investigations confirm that the forecasting
430 capability of the statistical approach is effective when the constitutive soil is coarser (such as
431 within the main body of Bormida embankment). For clayey and silty soils, the statistical approach
432 generally underestimates the fine fraction content up to 70%, less than what observed in borehole
433 logs. Conversely, the theoretical approach has a higher predicting capability, independently by the
434 overall soil characteristics of the retaining structure with average differences of 15% with respect
435 to borehole logs.



436
 437 **Figure 6.** Distributions of fine fraction contents F_c and C derived by the statistical and theoretical
 438 methods and their respective normalised differences for each analysed case study. In each plot
 439 black dashed lines identify the transition from the embankment body to foundation soil.
 440

441 **Table 2.** Comparison between fine fraction contents Fc and C derived by the statistical and
 442 theoretical methods and available grain size distribution from samples obtained in borehole logs
 443 at each test site.

	X [m]	Z [m]	Fc (<0.075m m) from boreholes [%]	Fc from statistical method (Hayashi et al. 2013) [%]	Difference [%]	C from theoretical method (Takahashi et al. 2014, Vagnon et al. 2021) [%]	Difference [%]
Bormida	48	4.8 - 5	87.6	20.95	76.08	25.00	71.46
		7 - 7.2	11.72	10.75	8.27	10.50	10.41
		8 - 8.2	9.72	9.72	0.02	10.00	-2.88
		9 - 9.3	2.21	9.40	-325.20	10.00	-352.49
Chisola SX	60	1	85.9	45.44	47.11	87.00	-1.28
	70	1	86.3	42.90	50.29	95.00	-10.08
	84	1	54.3	40.34	25.71	57.00	-4.97
Maira	14	1	77.41	24.86	67.89	7.33	90.53
	45	1	73.19	35.23	51.87	76.50	-4.52
	90	1	72.61	42.08	42.05	71.67	1.30
Briaglia	50	3 - 3.5	68	17.50	74.27	56.50	16.91
		15.5 - 16	65	10.65	83.61	43.00	33.85
Arignano	85	3.5 - 4	91.64	39.13	57.30	93.25	-1.76
		6.5 - 7	86.51	35.49	58.97	95.00	-9.81
	283	3.5 - 4	88.07	35.03	60.23	93.00	-5.60
		6.5 - 7	90.52	44.90	50.40	95.00	-4.95

444

445 **4.2 Seepage index and hydraulic conductivity estimation**

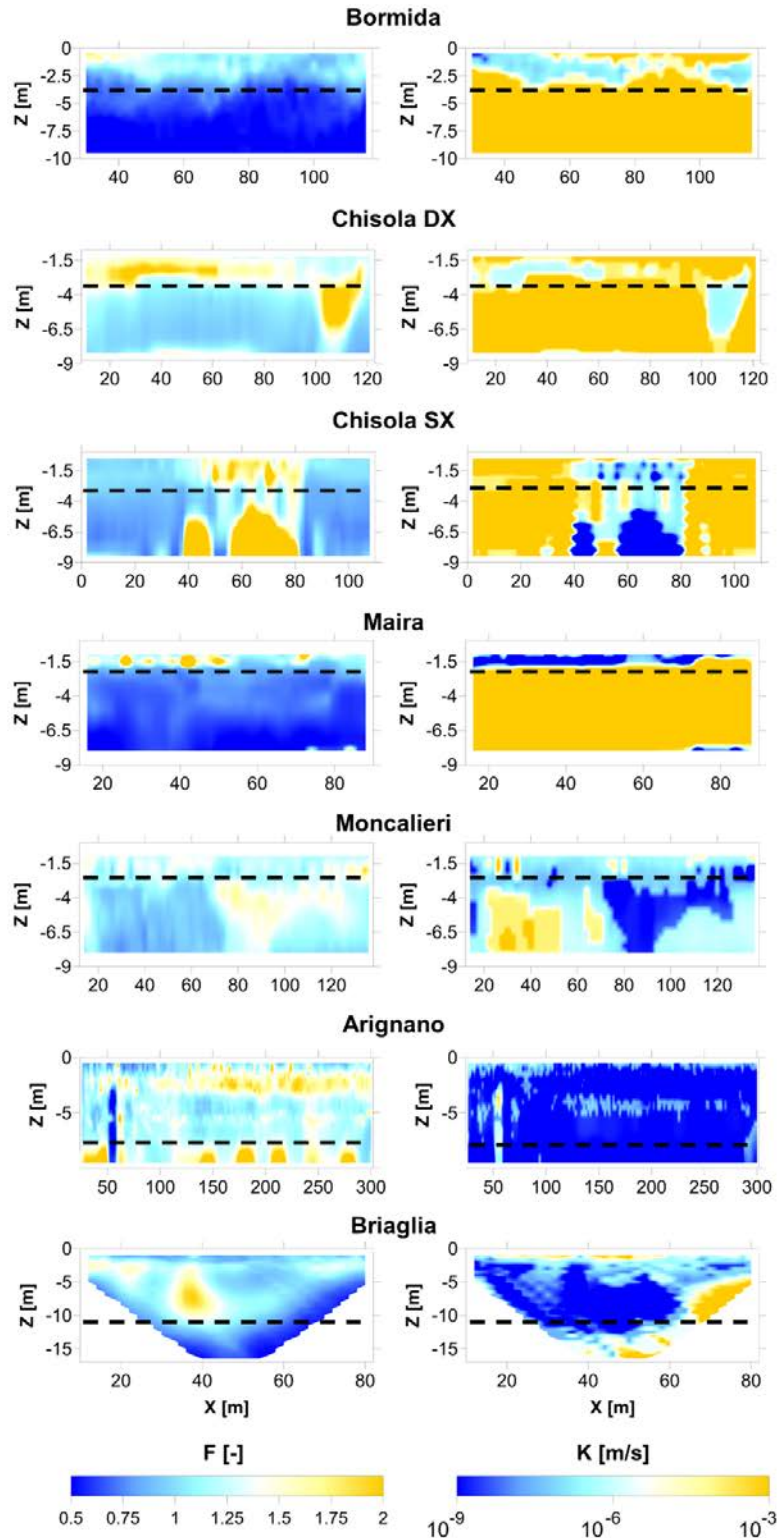
446 In Fig. 7 the seepage index, F, and hydraulic conductivity, K, distributions for each case study are
 447 shown. F and K are intimately linked each other since they provide information on embankment
 448 hydraulic conditions and possible sectors prone to piping and seepage phenomena.

449 As suggested by Chen et al. (2006), the empirical coefficients k_S and k_R depend on the overall
 450 geophysical and geotechnical conditions and they may in turn be calibrated on V_S and R
 451 distributions. In this study, since no evidence of seepage phenomena were previously detected, k_S
 452 and k_R were evaluated on the basis of the minimum V_S and R values observed in the surveys.

453 The values estimated for k_S and k_R in each test site are reported in Table 3.

454 The left column of Fig. 7 shows portions of the embankments with forecasted F values higher than
 455 2 (yellow colour). In these portions there are no matches with previous geotechnical investigations
 456 of potential seepage phenomena. However, some of the reported high F values are located at the
 457 interface between embankment body and foundation (e.g Moncalieri, Maira, Chisola DX and
 458 Bormida), therefore from a theoretical point of view, their susceptibility to seepage and piping
 459 may be considered moderate to high. Conversely, Chisola SX embankment exhibits high F values
 460 ($F > 2$) in correspondence of the restored portion of the levee. In this sector, compacted clays were
 461 used as construction material. Seepage susceptibility may be expected at the interface between

462 natural and restored soil but hopefully not within the latter. Therefore in this situation the field-
463 based approach fails in identifying a strong variation in material properties attributing the R and
464 Vs variations to potential piping effects not reflecting the real state of the embankment.
465 Contrary to the field-based approach, the theoretical approach allows the detection of sharp
466 variations of K (right column of Fig. 7), with the main advantage of a rapid identification of the
467 interfaces between soil with different hydraulic and geotechnical features. For instance, the
468 presence of the brick channel along the Arignano dam (at about 50 m in longitudinal direction and
469 at 3 m depth) is detected as a sector of high hydraulic conductivity compared to the surrounded
470 clayey and silty soil with very low K values. This hydraulic contrast may be responsible of
471 potential seepage and piping around the channel. The corresponding F distribution in this test site
472 doesn't highlight this possibility (no F values higher than 2 are forecasted around the channel).
473 Analogous observations can be extended to Chisola SX embankment where the restored soil is
474 detected as a sector with very low K values, accordingly to the design material used during
475 restoration works.
476



477
 478 **Figure 7.** Distributions of the seepage index F (left columns) and the hydraulic conductivity K
 479 (right columns) for each analysed case study. In each plot black dashed lines identify the transition
 480 from the embankment body to foundation soil.
 481

482 **Table 3.** List of k_S and k_R used for the evaluation of the seepage index F for each case study.

Site	k_S [m/s]	k_R [Ωm]
Bormida	145	22
Chisola DX	172	47
Chisola SX	136	19
Maira	165	11
Moncalieri	150	79
Arignano	50	24
Briaglia	49	25

483

484 **5. Discussions**

485 From the results reported in the paper it was observed that integrated seismic and electrical
 486 methods can be considered potentially useful tools for the characterisation of soil layering and
 487 related geotechnical parameters since they can be linked to soils stiffness (seismic properties) and
 488 water and clay content (electric properties), allowing for a preliminary classification as a function
 489 of soil fraction and providing indirect correlations with other important geotechnical parameters
 490 (e.g. hydraulic conductivity).

491 Notwithstanding this potentiality, some differences were observed in the obtainable results among
 492 the different adopted approaches, in comparison with available borehole data. The statistical
 493 approach discrepancies between predicted and observed fine fraction values can be related to the
 494 empirical and site-specific nature of this formulation. In fact, it was developed from measurements
 495 performed on Japanese earth retaining structures that might be slightly different, both in terms of
 496 geological and geotechnical features, from the embankments analysed in this work. Consequently,
 497 a devoted calibration of the polynomial coefficients in its formulation should be performed for
 498 optimizing the fit between estimated and observed parameters. For this calibration, however, a
 499 relevant number of independent geotechnical data and several case histories would be required.

500 On the contrary, the theoretical approach has a universal application, but it might be limited due
 501 to numerous assumptions necessary with respect to the parameters inherent in its formulation (such
 502 as clay and sand resistivity, interstitial water resistivity, critical porosity, saturation degree, etc.).
 503 At the same time, this approach allows punctual calibration with geotechnical observations, even
 504 if available in a limited number, for a detailed profiling of the retaining structure.

505 Apart from the limitations due to the soil characteristic assumptions, the main advantage of the
 506 theoretical model is its versatility since it can be employed in different saturation and soil
 507 conditions. Moreover, this approach also considers the confinement and the soil layering (in terms
 508 of depth and soil density). If borehole logs are available, the theoretical approach can be calibrated
 509 for estimating both the fine fraction content, C , and the hydraulic conductivity, K , distributions;
 510 on the other hand, it can forecast their distributions based on average reliable parameters.

511 Particularly, the possibility of estimating the hydraulic conductivity distribution along an earth
 512 retaining structure from geophysical data is fascinating. It must be however underlined that several
 513 constituting properties of the clay particles, such as its mineralogy and cation exchange capacity,
 514 are not explicitly considered in the theoretical formulation. These properties have been shown to
 515 have a paramount importance in the resulting hydraulic conductivity (e.g. [Revil et al. 1999](#)). With
 516 this respect, the electrical resistivity alone cannot be considered as an exhaustive parameter since
 517 electrical resistivity depends on both electrolytic conduction (fluid saturation and ionic
 518 composition) and surface conduction (in presence of clay particles or organic matter). The

519 contributions of these two entities are not easily distinguishable in survey results from the only
520 resistivity. Indeed, the conduction mechanisms from soil surface charge are usually mainly
521 associated to Induced Polarisation (IP). Several applications of IP surveys to the characterization
522 of dams and river embankments can be found in literature (e.g. [Abdulsamad et al. 2019](#); [Soueid et al. 2020](#)) exploiting this technique for a more comprehensive characterization.

524 Nevertheless, the electrical resistivity measurements are still often adopted as a first
525 characterization tool since their execution is significantly less time consuming than IP. Performing
526 IP measurements with the same instrumentation adopted in the paper would indeed require longer
527 current injection times, strongly increasing the survey time. In the aims of the present work, this
528 is considered as a drawback since the study was focused on providing fast characterization tools
529 for a first screening of the investigated structures. Further detailed characterization with
530 geotechnical tests and/or with the same IP measurements will be required, particularly in
531 correspondence of the location of the detected anomalies.

532 With this respect the provided hydraulic conductivity distributions must be considered more as a
533 tool for identifying anomalous zones within the embankments than as an attempt to strictly
534 quantify the hydraulic properties. In comparison with the empirical approach through the seepage
535 index F, developed for the same aim, again the theoretical approach showed increased
536 correspondence with available observations and a more comprehensive characterization at the
537 different test sites reported in this paper. Particularly, at the Arignano earth dam, independent tests
538 were performed to locally estimate the hydraulic conductivity (i.e. both variable-head hydraulic
539 conductivity tests and laboratory oedometer tests). The results of these tests were observed to be
540 in very good agreement with the ones from the distributions evaluated through the theoretical
541 approach, with hydraulic conductivity values always within the same order of magnitude ([Vagnon et al. 2021](#)).

543

544 **6. Conclusions**

545 The comparison between the analysed procedures for geotechnical parameters estimation through
546 electric and seismic data focused on strongpoints and limitations in forecasting earth structures
547 characteristics in comparison with previously available geotechnical investigations.

548 The electric and seismic streamer surveys and the analysed methods for geotechnical profiling
549 represent a good compromise between quality of the estimated data, costs and surveying time. The
550 theoretical approach, notwithstanding the limitations inherent in the calibrating parameter
551 necessary for its formulation, proved to be more effective in geotechnical estimation of the main
552 earth retaining structure properties. However, all the described methodologies are thought for a
553 first screening of earth retaining structures: consequently, independent geotechnical investigations
554 are essential for calibrating and validating obtained results. Whenever direct geotechnical data are
555 available at some profiles along the retaining structure, geophysical models should be properly
556 calibrated and can then be used to extend punctual direct information to the whole structure. Once
557 relevant anomalies are identified along the investigated structures with the proposed methods more
558 detailed geophysical investigations (e.g. Induced Polarization measurements) or direct
559 geotechnical investigations are necessary to allow a more precise definition of the geotechnical
560 parameters of interest.

561

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568

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