

1 A procedure to estimate cutoff wall transport properties from
2 monitoring wells

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8 **Abstract**

9 Owing to their capability in limiting the transport of pollutants in the subsoil, cutoff walls
10 are popular solutions for the confinement of contaminants. These barriers are often made of
11 soil-bentonite or cement-bentonite mixture, which are characterized by low hydraulic conduc-
12 tivity, low hydrodynamic dispersion and long-term durability. However, the aggressive chemical
13 environment to which these walls are subjected might negatively impact on their performance.
14 Assessment of their performance with time is thus a crucial issue in wall design. The use of
15 dedicated monitoring wells, cast in place inside the wall during construction when the bentonitic
16 mixture is still fluid, can be particularly suitable for both intercepting and detecting the fluids
17 flowing through the barrier. In this research, the results of a numerical study aimed at provid-
18 ing a methodology to estimate the transport properties of the backfill material at the site scale
19 are presented. The methodologies relies on abaci and only requires the flow and concentration
20 within a monitoring well inside the barrier to be known.

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1 Introduction

In both geotechnical and geoenvironmental applications, cutoff walls are aimed at controlling groundwater flow and pollutant migration, providing at the same time a negligible long-term environmental impact. Initially, cutoff walls were used for hydraulic applications, *e.g.* to prevent piping in dams (Cermak *et al.*, 2012) and groundwater flow into excavations (Opdyke and Evans, 2005). Subsequently, they started being applied also to remediation and securing of polluted areas, *e.g.* waste disposal and contaminant insulation (Jefferis, 1981; Rowe, 2005). For both applications, low hydraulic conductivity and long-term durability are required. These requirements are fulfilled by constructing the barriers by using either soil-bentonite or cement-bentonite mixtures (Ryan, 1985; Evans, 1993; Rumer and Mitchell, 1995). The site efficiency of the barrier system, however, depends not only on the properties of the mixture, but also on many other factors, such as the construction method, the presence of impurities and defects, and chemical interactions between the mixture and the pollutant (Joshi *et al.*, 2009).

The construction methods for soil- or cement-bentonite slurry trench cutoff walls are well-established (Ryan, 1987; Evans, 1993). A narrow (typically 0.5 to 1 m wide) slurry filled trench is first excavated in the subsurface. The slurry ($\sim 5\%$ bentonite and $\sim 95\%$ water) is employed to maintain trench stability as the excavation proceeds downward from the ground surface. For soil-bentonite filling, as the excavation proceeds longitudinally, the trench is backfilled by displacing the slurry with a mixture of soil, bentonite-water slurry, and occasionally dry bentonite (Malusis and McKeehan, 2013). The soil used in the backfill may be soil excavated from the trench, borrow soil imported from off-site, or a mixture of both, depending upon grain size characteristics, the presence/absence of contamination and hydraulic conductivity requirements. For the cement-bentonite mixtures, the slurry incorporated with cementitious binder (usually containing Portland cement but often blended with ground blast furnace slag or pulverised fuel ash) is left to harden in place, *i.e.* without a backfill soil, to form the cutoff wall (Jefferis, 2012). Cement-bentonite may be the backfill choice where strength considerations indicate the need for a material stronger than a soil-bentonite backfill (Jefferis, 1981).

Specifications in the U.K. (Institution of Civil Engineers, 1999) require the hydraulic conductivity of the backfill material at 90 days to be less than 10^{-9} m/s for at least 80% of laboratory

50 cured samples. However, the hydraulic conductivity of the barrier in the field depends on many
51 factors which, in general, cause it to be higher than the one measured in the laboratory (Bar-
52 venik and Ayres, 1987; Ryan, 1987; Trivedi *et al.*, 1992; Evans, 1993, 1994; Manassero, 1994;
53 Sanetti, 1998; Filz *et al.*, 2001; Britton *et al.*, 2005; Sanetti, 2006). The complexity and the
54 variability of these factors –*i.e.* as defects and fractures related to the construction, to the
55 oscillation of the groundwater level and interaction with the atmosphere, chemical changes in
56 the material fabric due to the aggressive ground conditions– imply that an *a priori* estimate of
57 the *in situ* hydraulic conductivity is not possible, even if laboratory test results are available
58 (Fratolocchi *et al.*, 2006; Du *et al.*, 2015; Evans *et al.*, 2017).

59 *In situ* measurements and monitoring of barrier properties are, therefore, an important issue,
60 and the field hydraulic barrier performance should always be verified after installation. Due to
61 the difficulties related to the collection of high-quality solid core samples from the constructed
62 walls, Manassero (1994) and Takai *et al.* (2016) proposed a site evaluation of the hydraulic con-
63 ductivity of the backfill material by means of cone penetration (CPTU) testing. The techniques
64 adopted, however, may induce a permanent damage to the barrier, due to the penetration of
65 the device into the solid backfill. Among non-invasive techniques, the solution generally used
66 consists in excavating monitoring wells outside the diaphragm (Sanetti, 2000), in the area to be
67 protected from pollution. However, in this case, the contaminant is detected only after migra-
68 tion through the barrier. An alternative solution has been introduced by Grisolia and Napoleoni
69 (1997), who performed *in situ* constant-head hydraulic conductivity tests via a piezometer in-
70 stalled in the barrier when the trench mixture is still in the slurry state. Accordingly, Sanetti
71 (1998) proposed the installation of monitoring/measuring wells in the liquid slurry, avoiding the
72 perforation of the wall and the related damage, and allowing to test the performance of a large
73 volume of the system. The goal of these wells is to collect the fluid passing through the barrier
74 before the leachate contaminates the surrounding environment, allowing an early identification
75 of contaminants and an assessment of the efficiency of the barrier. Monitoring wells can also be
76 used to check undesired permeability changes and an early identification of transport species
77 (Trivedi *et al.*, 1992; Fratolocchi *et al.*, 2006).

78 This paper explores a systematic numerical analysis of the process of contaminant transport
79 throughout cutoff walls that contain monitoring wells. In particular, a procedure to exploit

80 measurements performed in the wells to estimate the hydro-chemical properties of the barrier
 81 is presented. The procedure is synthesized in terms of dimensionless quantities, facilitating
 82 different geometrical configurations of both barrier and well, and different hydraulic conditions
 83 at the inlet and outlet of the wall.

84 2 Cut-off wall with monitoring wells: geometrical 85 scheme

86 In this study, the cutoff wall is assumed to be installed to insulate a polluted area from a
 87 freshwater aquifer (Figure 1). The combination of the wall and the contaminated area are
 88 considered to be large relative to the wall thickness, such that end effects can be disregarded
 89 along the longitudinal direction of the wall. The water level in the polluted area was considered
 90 to be either higher (worst case scenario, such as reported in Figure 1), or lower than the aquifer
 91 hydraulic head, providing diffusive and advective fluxes in either the same (as is shown in Figure
 92 1) or opposite direction.

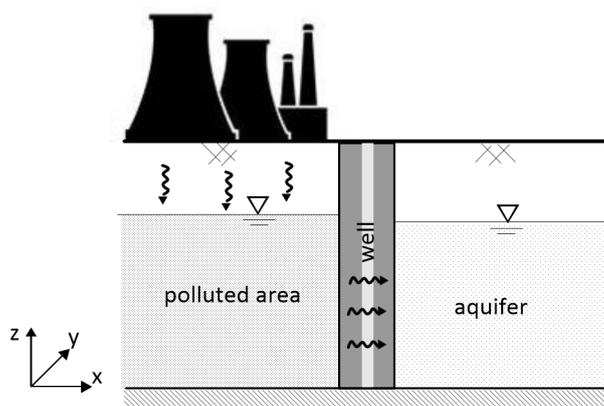


Figure 1: Schematic view of the cutoff wall, with a monitoring well.

93 The simulation of contaminant transport and water flow through a barrier without wells
 94 is generally performed under one-dimensional conditions, since (i) the barrier length is much
 95 larger than its thickness, l , and (ii) the flux direction across the barrier, according to the *Dupuit*
 96 assumption, is assumed to be horizontal. A three-dimensional scheme for the barrier with wells
 97 is shown in Figure 2. In the presence of wells of diameter d , transversally centered and spaced
 98 a distance s apart, the flow of water can be still treated as horizontal but two-dimensional in

99 the horizontal plane. Nonetheless, by assuming homogeneity of the barrier, the symmetry of
100 the problem allows to study just a segment of the wall centered in the wells, s long and l thick
101 (dark area in Figure 2).

102 From the geometrical point of view, the system can thus be described, for a given thickness
103 l , by just two geometrical dimensionless variables: (i) the normalized diameter d/l of the well,
104 and (ii) the normalized spacing s/l . In the numerical study, different geometrical configurations
105 of the system were considered.

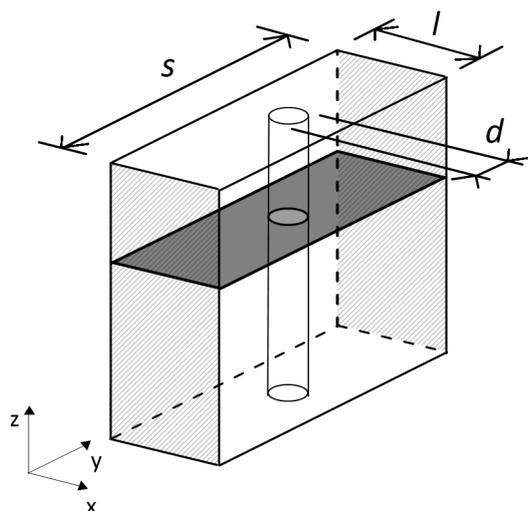


Figure 2: Three-dimensional scheme of the cutoff wall with monitoring well.

106 **3 Field equations for dilute contaminant transport**

107 **across the barrier**

108 **3.1 Modelling assumptions**

109 The flux of a dissolved contaminant in a saturated cutoff wall is considered. Accordingly, the
110 solution of two field equations is required, namely the water mass balance equation and the
111 contaminant mass balance equation. The model was kept as simple as possible, while trying
112 to find a compromise between reproduction of all the relevant physical processes, the need
113 for the inverse problem to be well-posed, and robustness of application in engineering design.
114 Accordingly, the backfill material was assumed homogeneous and isotropic in terms of transport
115 properties, *e.g.* hydraulic conductivity and hydrodynamic dispersion. Progressive enhancement

116 (or otherwise, decay) of the hydraulic properties of the backfill were not considered. The
 117 evidence exists that the permeability of some mixtures continues to decrease over long times,
 118 *e.g.* the permeability of the blast furnace slag cement with an activated Na-bentonite tested
 119 by Fratalocchi *et al.* (2006) reached a stable value only after 300 days in tap water. However,
 120 introducing such time dependency in the numerical simulations would not significantly change
 121 the contaminant breakthrough time, as showed in the sensitivity analysis presented in Appendix
 122 1. Also, any physico-chemical interaction between the constituents of the barriers and the
 123 contaminated ground water were not considered. Overall, the present methodology aims to
 124 provide a tool for a quick and proper evaluation of the current transport parameters at the field
 125 scale. Both anomalously high values and values that increase with time are key indicators of
 126 malfunctions of the barrier.

127 3.2 Water mass balance equation

128 Water flow across the barrier is governed by the Darcy law (Equation 1):

$$\mathbf{v} = -K\nabla h, \quad (1)$$

129 where \mathbf{v} is the water velocity field in the domain, K is the hydraulic conductivity and ∇h is
 130 the gradient of the hydraulic head. Under the assumption of constant water density, negligible
 131 porosity changes, isotropic and homogeneous hydraulic conductivity, the two-dimensional mass
 132 balance is expressed by the Laplace equation:

$$\nabla^2 h = \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0, \quad (2)$$

133 where x and y are the horizontal reference coordinates.

134 The imposed hydraulic boundary conditions adopted are illustrated in Figure 3a. A constant
 135 hydraulic head h_{in} is applied at the inlet boundary (in contact with the polluted area), while
 136 a constant hydraulic head h_{out} is applied at the outlet boundary (in contact with the aquifer).
 137 These head values are set accordingly to the *in situ* water table levels. Lateral flow is not
 138 permitted ($\partial h / \partial y = 0$). At the well boundaries, a hydraulic head h_{well} is imposed. All the
 139 results are presented in terms of the non-dimensional quantity h_{well}^* , defined as follow:

$$h_{well}^* = \frac{h_{well} - h_{out}}{h_{in} - h_{out}}. \quad (3)$$

If $h_{well}^* = 0$ the hydraulic head in the well coincides with that at the outlet, $h_{well} = h_{out}$. If, in contrast, $h_{well}^* = 1$, the hydraulic head in the well coincides with that at the inlet, $h_{well} = h_{in}$.

3.3 Contaminant mass balance equations

The flux \mathbf{j} represents the quantity of contaminant passing through a unitary area of porous medium in a time increment, and is expressed as:

$$\mathbf{j} = c\mathbf{v} - D\nabla c. \quad (4)$$

The first term represents the advective contribution, depending on the hydraulic gradient through *Darcy* velocity \mathbf{v} (Equation 1). The second term represents the diffusive contribution, related to the gradient of contaminant concentration via the hydrodynamic dispersion D , accounting for both molecular diffusion and mechanical dispersion (Shackelford, 1990; Della Vecchia and Musso, 2016).

The transport of a dissolved contaminant in water is governed by the contaminant mass balance equation. Assuming negligible changes in porosity and complete saturation, the contaminant mass balance is expressed by the advection-diffusion equation (*e.g.* Rowe *et al.* 1995; Bear 2013), according to which the variation in contaminant concentration c with time is related to the divergence of the contaminant mass flux \mathbf{j} :

$$R \frac{\partial c}{\partial t} + \nabla \cdot \mathbf{j} = 0, \quad (5)$$

where R is the retardation factor. If no adsorption of contaminant occurs, R is equal to 1. For solutes subject to reversible, linear and instantaneous (equilibrium) adsorption reactions during diffusive transport (Smith and Jaffe, 1994), R is greater than 1, representing a retard action on the contaminant migration.

The barrier is assumed to be initially free of contaminant, or:

$$c(x, y, t = 0) = 0. \quad (6)$$

The choice of the appropriate boundary conditions for contaminant flux into barriers is not straightforward (Rabideau and Khandelwal, 1998; Prince *et al.*, 2000; Li *et al.*, 2017). However, it could have a strong impact on numerical results, especially when advection dominates diffusion, *i.e.* for values of the *Peclet* number $Pe = |\mathbf{v}|l/D \geq 20$ (Van Genuchten and Parker, 1984).

Figure 3b shows the chemical boundary conditions adopted for the barrier in accordance to Brenner (1961) suggestion:

- at the inlet boundary, a *Robin* boundary condition is imposed, in order to guarantee contaminant mass conservation between the polluted area and the cutoff wall (Van Genuchten and Parker, 1984):

$$\mathbf{v}c - D\nabla c = \mathbf{v}c_0, \quad (7)$$

where c_0 is the contaminant concentration in the polluted area;

- at the outlet boundary, according to Brenner (1961), solute concentration is assumed to be continuous between the barrier and the aquifer: $\partial c/\partial x = 0$. Following the observation by Rabideau and Khandelwal (1998), the case of a perfectly flushing boundary condition was also considered, and it did not have significant impact on the methodology results (see Appendix 2);
- across the lateral sides of the domain, symmetry requires the imposition of no flux conditions, *i.e.* $\partial c/\partial y = 0$.

Finally, a proper boundary condition is needed for the contaminant at the well boundary Γ_d . This condition was chosen by imposing the mass balances to the well system. As for the water, in order to maintain a constant hydraulic head in the well (Section 3.2), the water flow entering the well q_{well}^{in} must be equal to the water flow pumped outside the well q_{well}^{out} :

$$q_{well}^{in} = q_{well}^{out} = q_{well}, \quad (8)$$

182 where $q_{well} = v_{av}\pi d$, being v_{av} the average Darcy velocity of water across well boundary Γ_d
 183 whose normal unit vector is \mathbf{n} : $v_{av} = 1/(\pi d) \int_{\Gamma_d} (\mathbf{v} \cdot \mathbf{n}) d\Gamma$. Solute mass balance implies that the
 184 variation in contaminant mass inside the well is ruled by the difference between the inlet (j_{well}^{in})
 185 and the outlet ($j_{well}^{out} = cv_{av}$) average contaminant flux:

$$V_{well} \frac{\partial c}{\partial t} = S_{well} (j_{well}^{in} - cv_{av}), \quad (9)$$

186 where $V_{well} = \pi d^2/4$ is the volume of the well per unit depth, and $S_{well} = \pi d$ is the lateral
 187 surface of the well per unit depth. Note that q_{well} is greater than zero only when $h_{well}^* < 0.5$
 188 (Scelsi *et al.*, 2019). Further, although low hydraulic conductivity barriers ($K \sim 10^{-9} m/s$) are
 189 considered, a measurable quantity of the outflow can be obtained in reasonable range of time
 190 (see the Examples on Sect.5.3).

191 The contaminant mass balance differential equation (Eq. 9) rules the variation with time of
 192 the concentration within the well. This concentration is then imposed at the boundary between
 193 the well and the barrier, leading to a concentration inside the well that is updated at every time
 194 step.

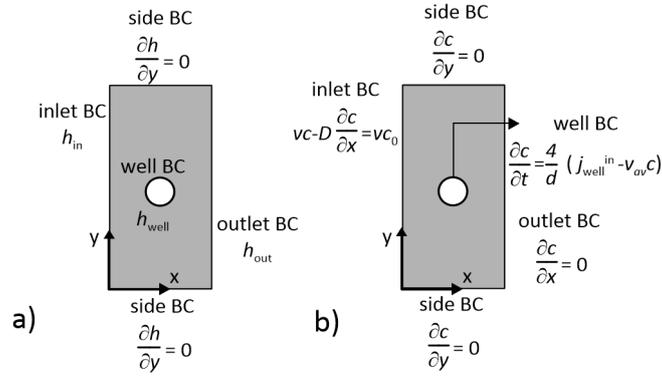


Figure 3: a) Boundary conditions for the water mass balance equation. b) Boundary conditions for the solute mass balance equation.

195 4 Scenarios analyzed in the numerical simulations

196 The system of Equations 2 and 5 was solved numerically by employing the Finite Element
 197 Method using Comsol Multiphysics.

198 Two hydraulic scenarios (Britton *et al.*, 2005; Neville and Andrews, 2006) were considered
 199 (see Figure 4):

- 200 • Hydraulic Scenario 1 (HS1, Figure 4a): the hydraulic head in the polluted area is greater

201 than at the outlet, and $\Delta h = h_{out} - h_{in} < 0$. In this scenario, the boundary conditions

202 are such that both the advective and the diffusive fluxes are in the same direction in

203 the absence of the well, *i.e.* from the contaminated area to the aquifer. The normalized

204 hydraulic head inside the well h_{well}^* should be lesser than 0.5, in order to avoid water flow

205 from the well to the aquifer (Scelsi *et al.*, 2019).
- 206 • Hydraulic Scenario 2 (HS2, Figure 4b): the hydraulic head in the polluted area is kept

207 lower than the one in the aquifer, and $\Delta h = h_{out} - h_{in} > 0$. The boundary conditions are

208 such that the advective and the diffusive fluxes are in opposite directions. This scenario

209 reduces the contaminant flux towards the aquifer.

210 HS1 is the most unfavourable for the containment of the contaminant, because both advection

211 and diffusion drive the pollutant toward the aquifer. In HS2, advection and diffusion may

212 partially balance and depending on the Peclet number one dominates over the other.

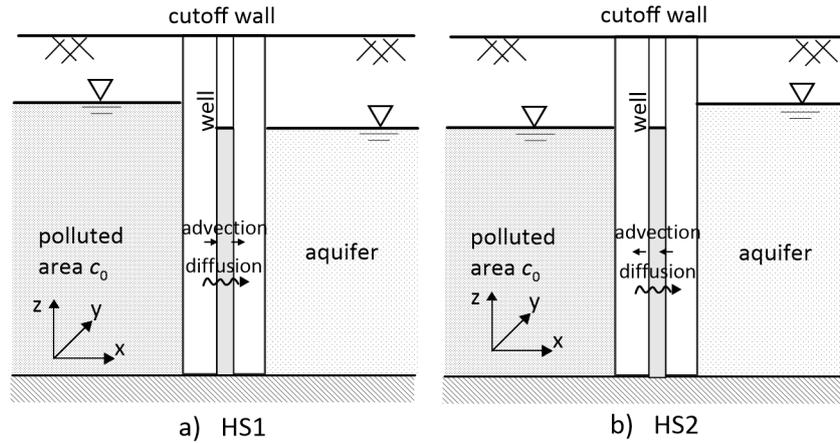


Figure 4: Scheme of the different hydraulic scenarios analyzed: a) Hydraulic Scenario 1 (HS1) with $h_{well}^* = 1$ and b) Hydraulic Scenario 2 (HS2) with $h_{well}^* = 1$.

213 5 Exploiting monitoring wells to estimate in situ trans-

214 port properties

215 Scelsi *et al.* (2019) proved that, for certain geometrical configurations and boundary conditions,

216 the presence of the wells may contribute to mitigate and retard the contaminant flux through

217 the barrier. For instance, when $h^* \leq 0.5$, water is drained by the well and the transport

218 of contaminant towards the aquifer is retarded by this drainage. This paper focuses on the
 219 information which can be obtained exploiting the wells for monitoring purposes. The most
 220 evident application is the detection of the presence of contaminants crossing the barrier, by
 221 analysing the chemical composition of the flowing water. The detection of a contaminant halfway
 222 towards the aquifer *–i.e. many years in advance–* is certainly beneficial. However, the paper
 223 proposes a further use of monitoring wells: if the water discharge into the wells is measured
 224 and the water chemistry is analyzed, this information can be exploited in order to estimate
 225 the *in situ* transport properties of the barrier via back analysis. In particular, water flow
 226 into the well allows for the hydraulic conductivity to be estimated, while the breakthrough
 227 curves of contaminant flux into the well can be used to obtain the hydrodynamic dispersion
 228 and the retardation factor. Once determined via the measurements performed at the site, the
 229 current values of hydraulic conductivity and hydrodynamic dispersion may be used to update
 230 the predictions of the barrier performance.

231 5.1 Hydraulic conductivity estimate

232 The water flow entering into the well q_{well} depends on the difference between the hydraulic head
 233 in the well and the hydraulic heads at the inlet and the outlet, *i.e.* on the normalized hydraulic
 234 head in the well h_{well}^* . Figure 5 shows an example of the role of h_{well}^* on the normalized hydraulic
 235 head distribution $h^*(x, y)$ inside the barrier and on the flow lines. $h^*(x, y)$ was defined as:

$$h^*(x, y) = \frac{h(x, y) - h_{out}}{h_{in} - h_{out}}. \quad (10)$$

236 The two-dimensional flow path induced by the well is evident for $h_{well}^* = 0$, (Figure 5a).
 237 For $h_{well}^* = 0.4$, the flow path is less affected by the presence of the well. For given hydraulic
 238 boundary conditions and a given geometrical configuration of the wells, then the water flow
 239 entering into the well q_{well} can be directly linked to the hydraulic conductivity K of the barrier.
 240 To this aim, several FEM numerical analyses solving the stationary water mass balance equation
 241 were performed, with the aim of creating the non-dimensional plots of Figure 6. These abaci
 242 provide the evolution of the non-dimensional group $K|\Delta h|/q_{well}$ with the non-dimensional well
 243 spacing s/l for different well diameters d/l and well normalized hydraulic heads h_{well}^* . Only

244 cases with water flow entering into the well ($q_{well} > 0$) are exploited. This means that for HS1,
 245 $h_{well}^* = h_{HS1}^* < 0.5$, while for HS2 $h_{well}^* = h_{HS2}^* > 0.5$.

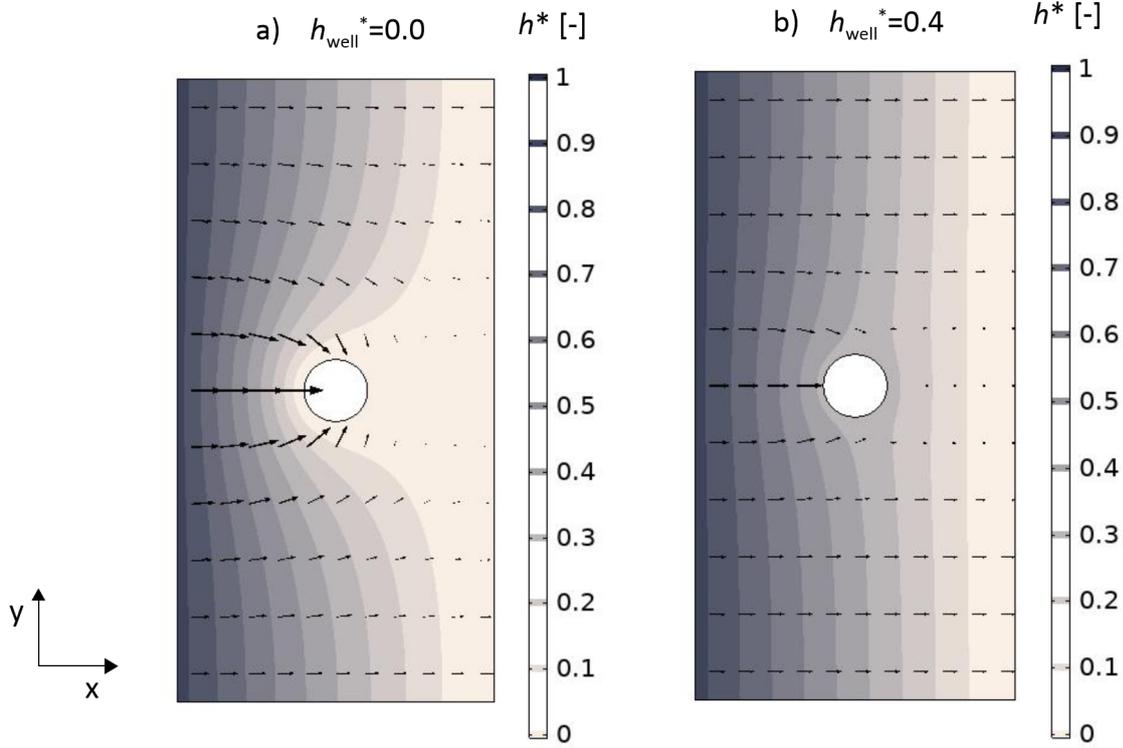


Figure 5: Contour plots of the hydraulic head and water velocity field for two different normalized water heads inside the well: a) $h_{well}^* = 0.0$ and b) $h_{well}^* = 0.4$, for $d/l = 0.2$ and $s/l = 2$.

246 The procedure to estimate the hydraulic conductivity of the barrier K is:

- 247 1. Select the relevant value of the hydraulic head h_{well} and compute the corresponding di-
- 248 mensionless value h_{well}^* ;
- 249 2. Select the relevant value of well spacing s/l and diameter d/l ;
- 250 3. Obtain from the abacus in Figure 6 the corresponding non-dimensional value of $K|\Delta h|/q_{well}$;
- 251 4. Measure *in situ* the water flow in the well q_{well} ;
- 252 5. Estimate K by multiplying the non-dimensional ratio by $q_{well}/|\Delta h|$.

253 A block diagram summarizing the logical steps of the procedure to estimate K is reported in
 254 Figure 7. From Figure 6, as s/l increases, the corresponding variation in $K|\Delta h|/q_{well}$ becomes
 255 negligible. This is related to the fact that, for values s/l larger than 2, the discharge q_{well} starts
 256 to be independent from well spacing, due to the finite area of influence of the well.

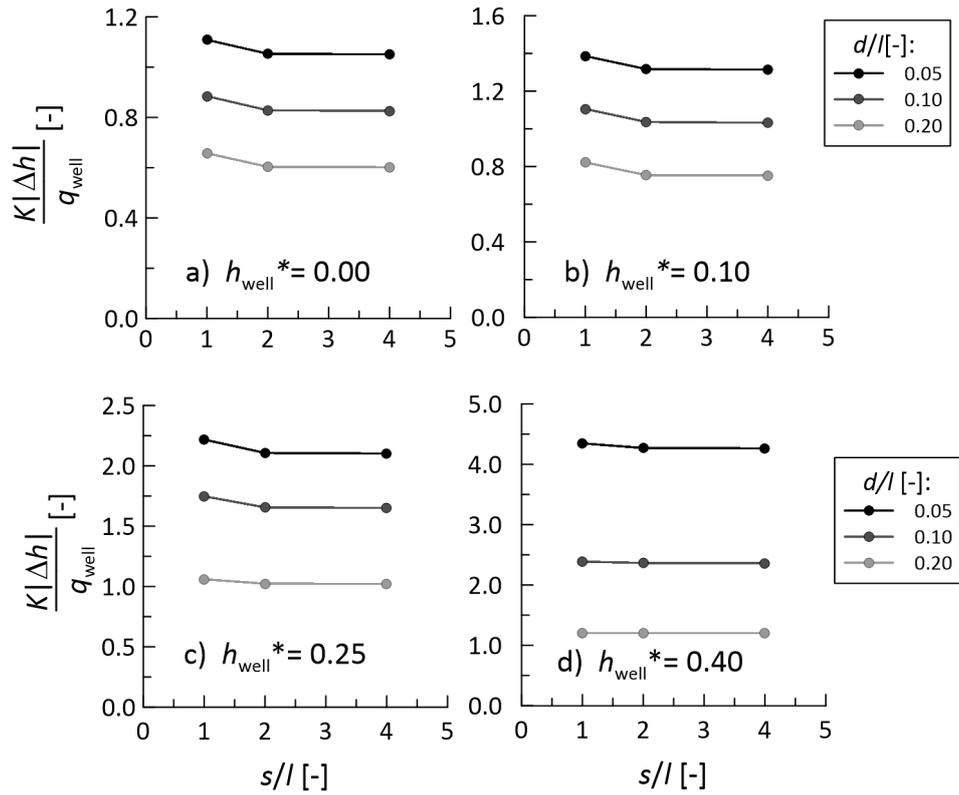


Figure 6: Non-dimensional abaci to determine the *in situ* hydraulic conductivity of the barrier.

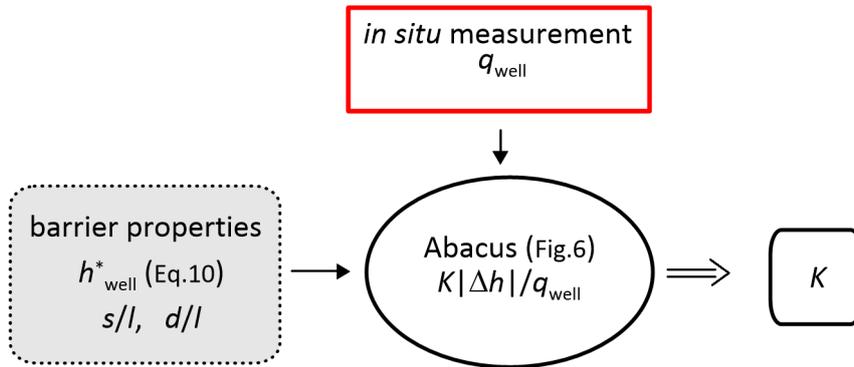


Figure 7: Procedure for the assessment of K .

257 5.2 Hydrodynamic dispersion and retardation factor estimate

258 Once the hydraulic conductivity is known, the contaminant concentration measured within the
 259 well can be exploited to estimate the hydrodynamic dispersion and the retardation factor of the
 260 barrier.

261 Parametric analyses were performed to identify the effects of the hydrodynamic dispersion
 262 D and retardation factor R on concentration breakthrough curves in the well, accounting for
 263 both hydraulic scenarios HS1 and HS2. The results are summarized in Figure 8, where just
 264 non-dimensional variables were used, namely

- The normalized concentration c/c_0 , *i.e.* the ratio of current concentration with respect to the constant concentration of the polluted area;
- The Peclet number $Pe = K|\Delta h|/D$, describing the relative role of advection with respect to diffusion, *i.e.* of hydraulic conductivity with respect to the hydrodynamic dispersion;
- The non-dimensional time $T = K|\Delta h|t/l^2$.

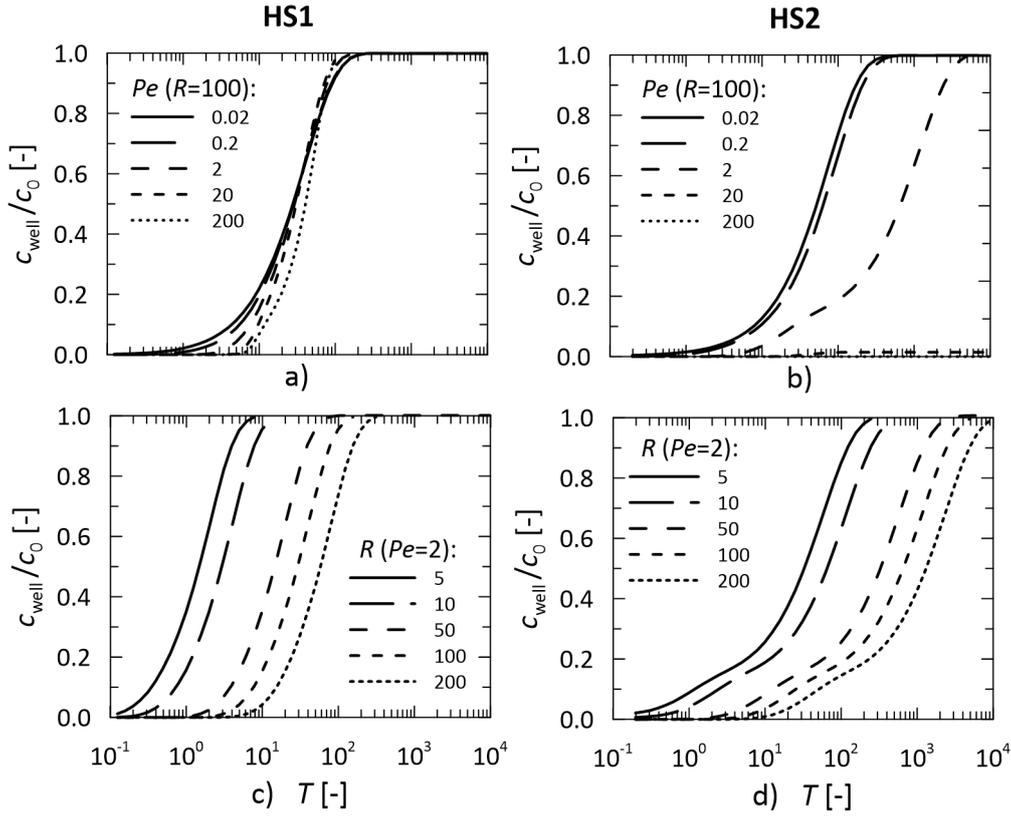


Figure 8: Concentration evolution into the well for scenarios HS1 and HS2, by varying a,b) the Peclet number Pe and c,d) the retardation factor R . $h_{well}^* = 0$, $s/l = 2$ and $d/l = 0.2$ were assumed in all the simulations.

In Figure 8a, corresponding to hydraulic scenario HS1 and $h_{well}^* = 0$, the arrival of the contaminant at the well, in terms of non-dimensional time, increases for increasing Peclet number, as well as the slope of the breakthrough curve in the log time scale. The effect of an increase in the retardation factor R (Figures 8c) is to shift the breakthrough curve toward longer times while maintaining the same shape. This observation is also valid for hydraulic scenario HS2 (Figure 8d). As for the role of the Peclet number in scenario HS2 (Figure 8b), it is worth noting that the contaminant does not get the well when advection dominates diffusion, *i.e.* for $Pe \geq 20$. The role of advection in modifying the shape of breakthrough curves of contaminant in the well is however already evident for $Pe = 1$.

279 In order to estimate the hydrodynamic dispersion, the rate of concentration change in the
 280 well evaluated in a log time scale, being almost independent on the retardation factor, appears
 281 as an appropriate variable (to measure in site). For practical purposes, the slope θ can thus be
 282 defined as:

$$\theta = \frac{1}{c_0} \frac{c_{well}(t_1) - c_{well}(t_0)}{\log\left(\frac{t_1}{t_0}\right)}, \quad (11)$$

283 where t_0 and t_1 are the times corresponding to a given value of the non-dimensional concentration
 284 ratio c_{well}/c_0 : *i.e.* equal to 1% and 5%, respectively.

285 Figure 9 and 10 report abaci of the relation between θ and D for different spacing s/l ,
 286 diameters d/l and normalized hydraulic head inside the well h_{well}^* for both hydraulic scenarios.
 287 As expected, for hydraulic scenario HS1 (Figure 9), a lower value of D corresponds in general to
 288 larger values of θ . When the transport mechanism is dominated by the hydrodynamic dispersion
 289 (*i.e.* $D/(K|\Delta h|) > 5$, corresponding to $Pe < 0.2$), the curves are the same regardless of the
 290 value of h_{well}^* . When the dominant transport mechanism is the advection, the value h_{well}^*
 291 becomes more relevant. For Scenario HS2, as shown in Figure 10, only Peclet numbers lower
 292 than 1 are considered (*i.e.* $D/(K|\Delta h|) > 1$). For larger Peclet numbers, the advective flux
 293 (directed from the outlet to the inlet) limits the diffusive flux of contaminant (directed from
 294 the inlet to the outlet), leading to a reduction in θ with increasing $D/(K|\Delta h|)$ up to invalidate
 295 the parameter estimate procedure. The role of spacing s/l in the abaci of Figure 9 and 10 is
 296 limited, with a slight tendency to provide an increase in slope θ for increasing s/l , especially for
 297 low D . The influence of diameter d/l is more relevant, because it changes the distance between
 298 barrier boundaries and the well. In particular, for HS1, the larger the well diameter d/l , the
 299 smaller the slope θ (Figure 9). In contrast, an increase in d/l implies an increase in θ for HS2
 300 (Figure 8).

301 Figure 11 and 12 report abaci of the relation between the dimensionless ratio of $D/(K|\Delta h|)$
 302 and the non-dimensional group $Rl^2/(t_0K|\Delta h|)$. Also in this case, different well configurations
 303 in terms of spacing s/l , diameter d/l and well normalized hydraulic head h_{well}^* are considered.
 304 Given the time of contaminant detection t_0 , the retardation factor decreases with D , and sta-
 305 bilizes for $D/(K|\Delta h|) < 10^{-1}$. The spacing between wells does affect significantly the trend for

306 HS1, as is shown in Figure 9, but is more relevant for HS2.

307 The procedure for assessing the chemical transport properties of the barrier, by the use of
 308 the abaci of Figs. 9–12 as follow is illustrated in Fig.13:

- 309 1. monitor the time evolution of contaminant concentration inside the well and determine the
 310 time t_0 , corresponding to a non-dimensional contaminant concentration ratio $c(t_0)/c_0 =$

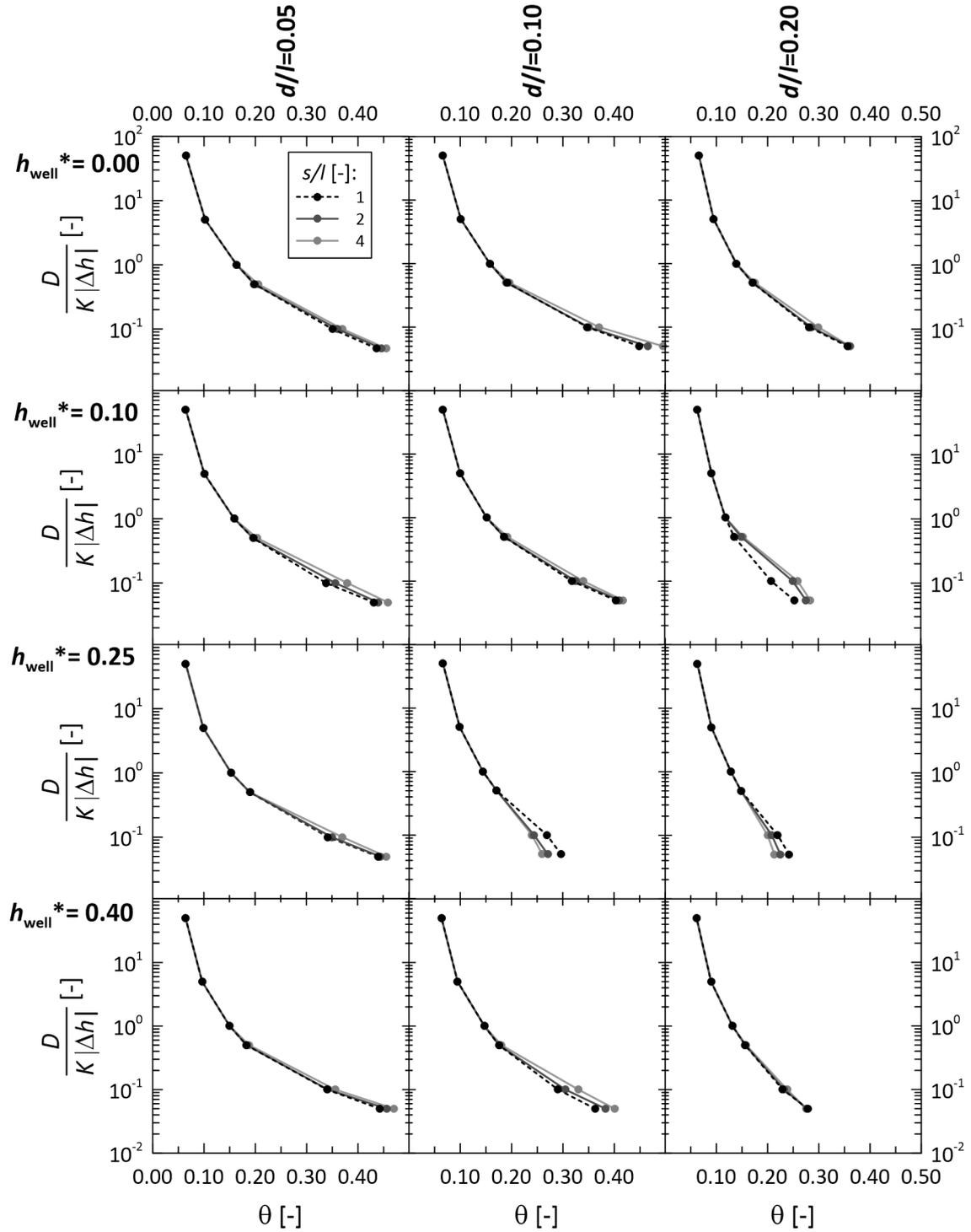


Figure 9: Abacus for the estimation of the hydrodynamic dispersion D as a function of d/l , s/l and h_{well}^* for a hydraulic regime HS1.

311

1%;

312

2. determine the time t_1 , corresponding to a non-dimensional contaminant concentration

313

ratio $c(t_1)/c_0 = 5\%$;

314

3. calculate by means of Eq. 11 the value of θ ;

315

4. given the well spacing, diameter and hydraulic head, estimate from Figure 9 or 10 the

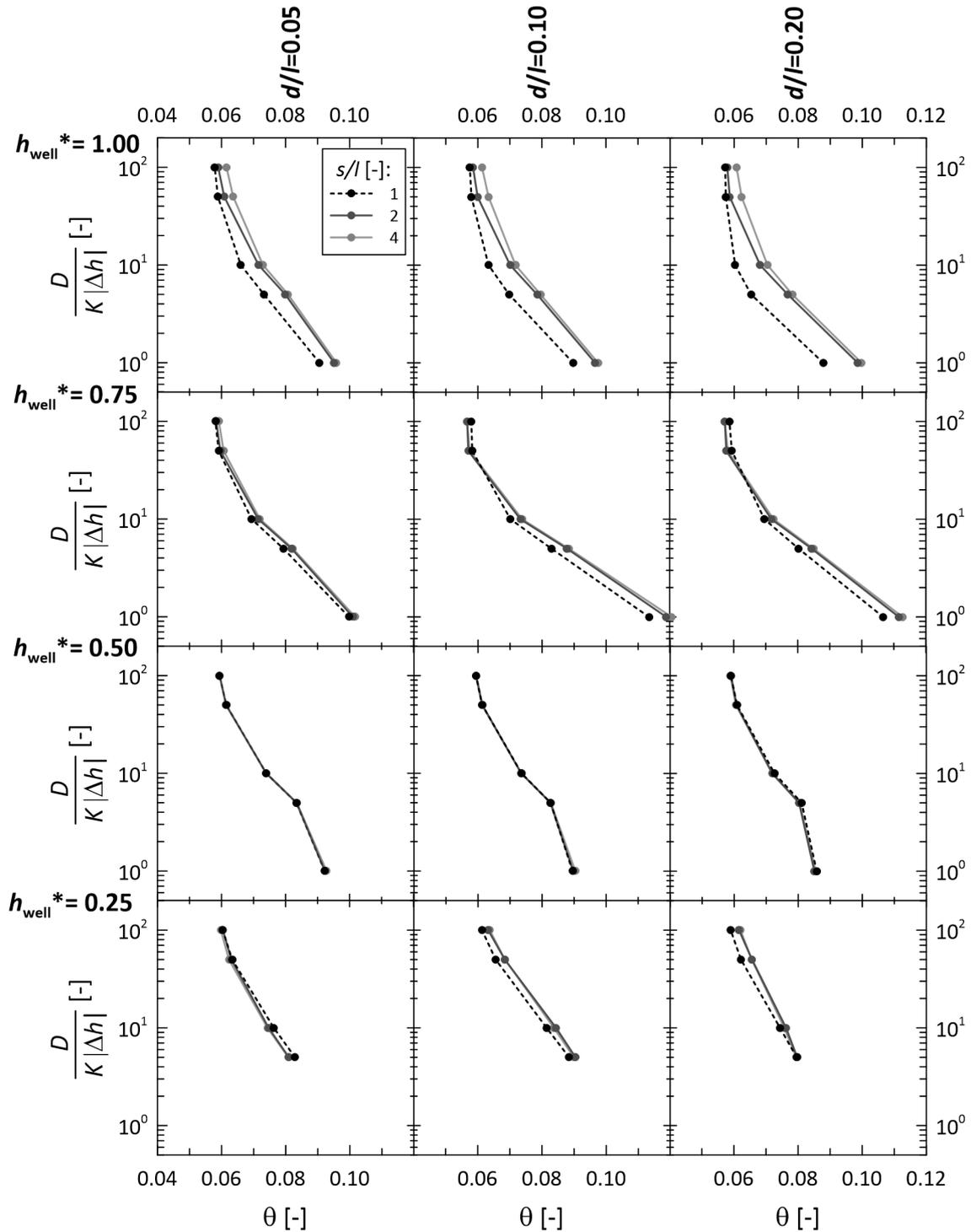


Figure 10: Abacus for the estimation of the hydrodynamic dispersion D as a function of d/l , s/l and h_{well}^* for a hydraulic regime HS2.

value of the non-dimensional group $D/(K|\Delta h|)$;

5. after determining K (from the procedure described in Sect. 5.1) and Δh , estimate the

hydrodynamic dispersion, D ;

6. with D , determine the value of $D/(K|\Delta h|)$ and use Figure 11 or 12 to estimate the non-

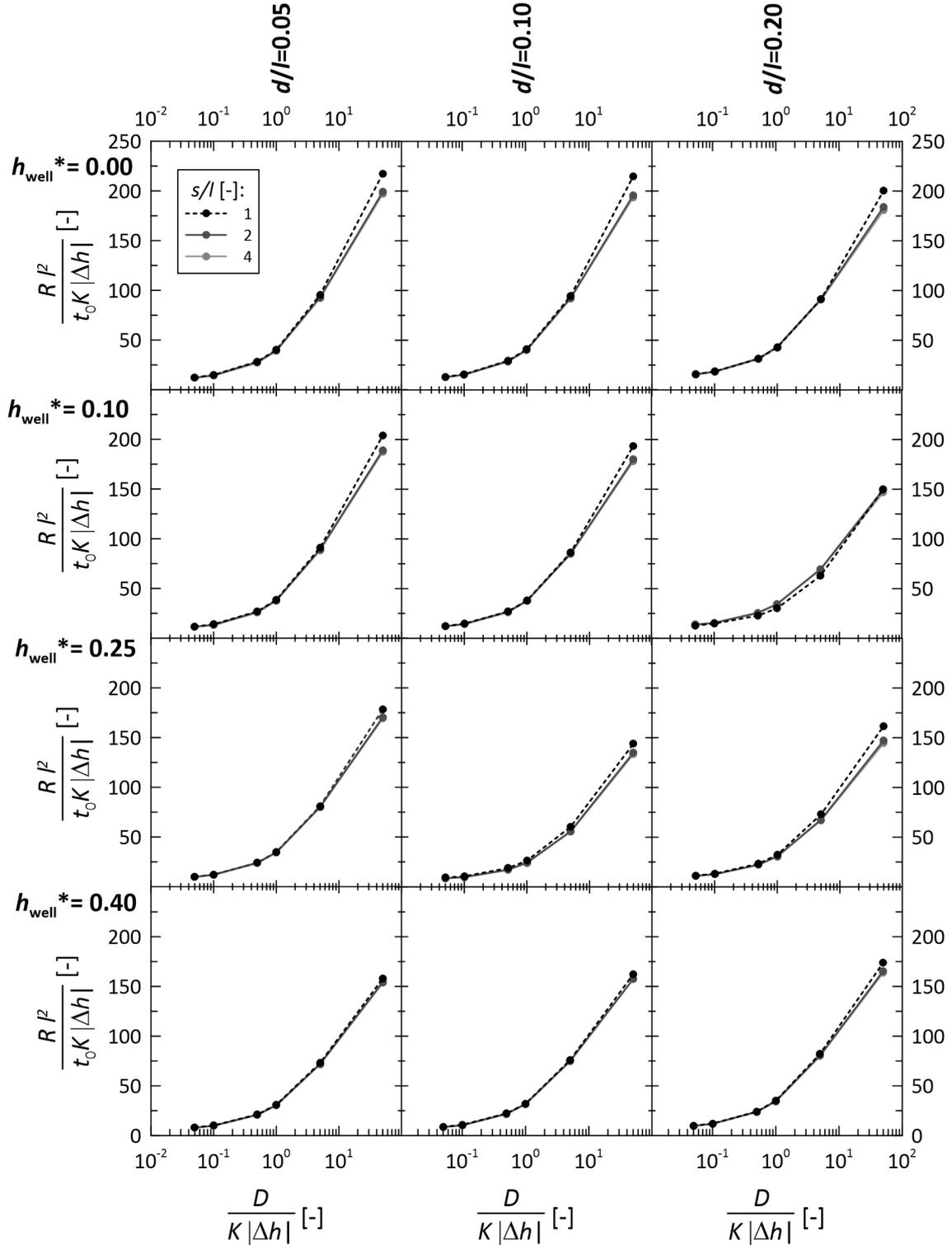


Figure 11: Abacus for the assessment of R as a function of d/l , s/l and h_{well}^* (hydraulic regime HS1).

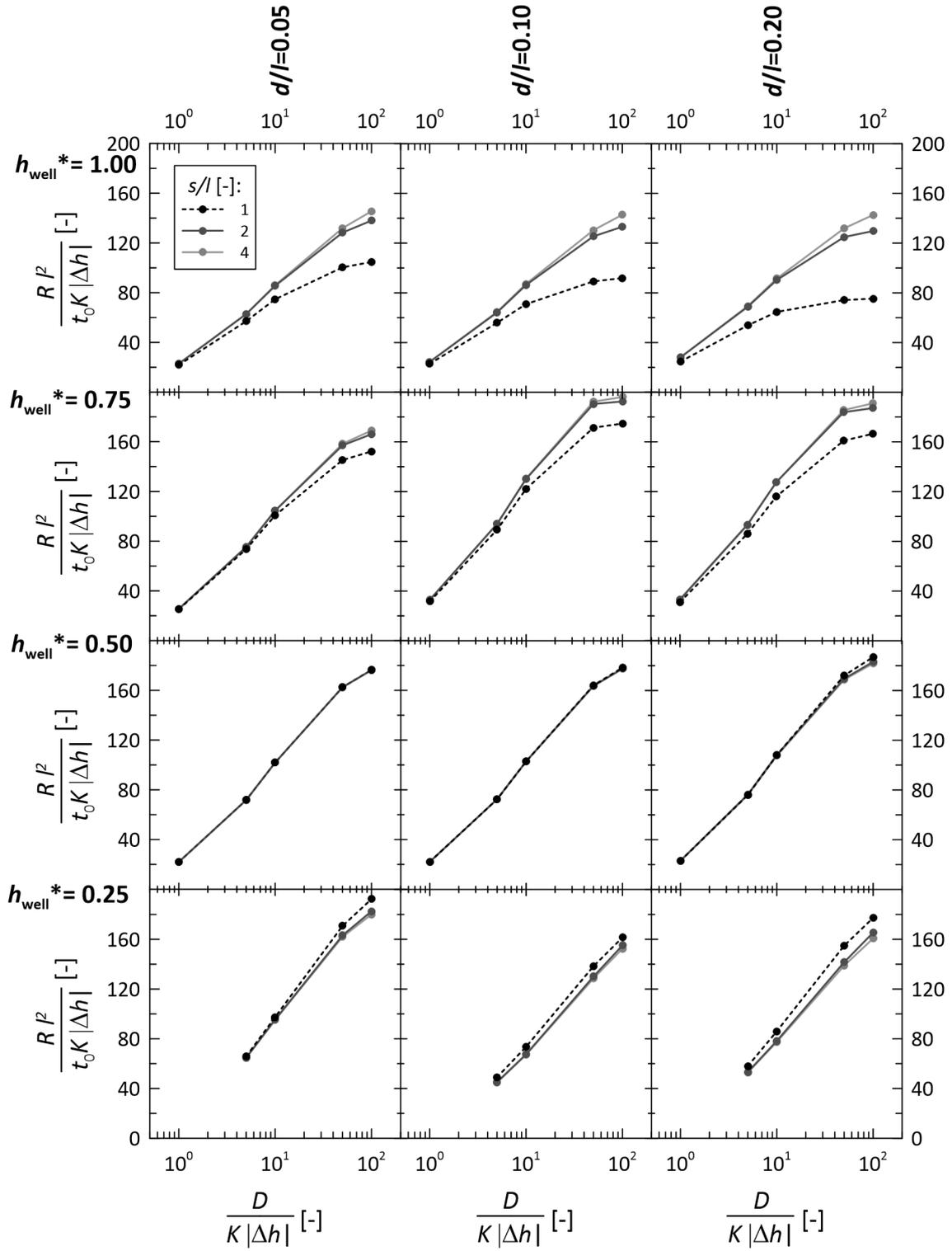


Figure 12: Abacus for the assessment of R as a function of d/l , s/l and h_{well}^* (hydraulic regime HS2).

320

dimensional group $Rl^2/(t_0K|\Delta h|)$;

321

7. with the known t_0 , estimate the retardation factor as $R = \frac{t_0K|\Delta h|}{l^2}$.

322

The same non-dimensional results reported in the abaci of Figures 9-12 apply also for the flushing

323

zero concentration boundary condition of no contaminant in the aquifers (see Appendix 2 for

324

the detailed explanation).

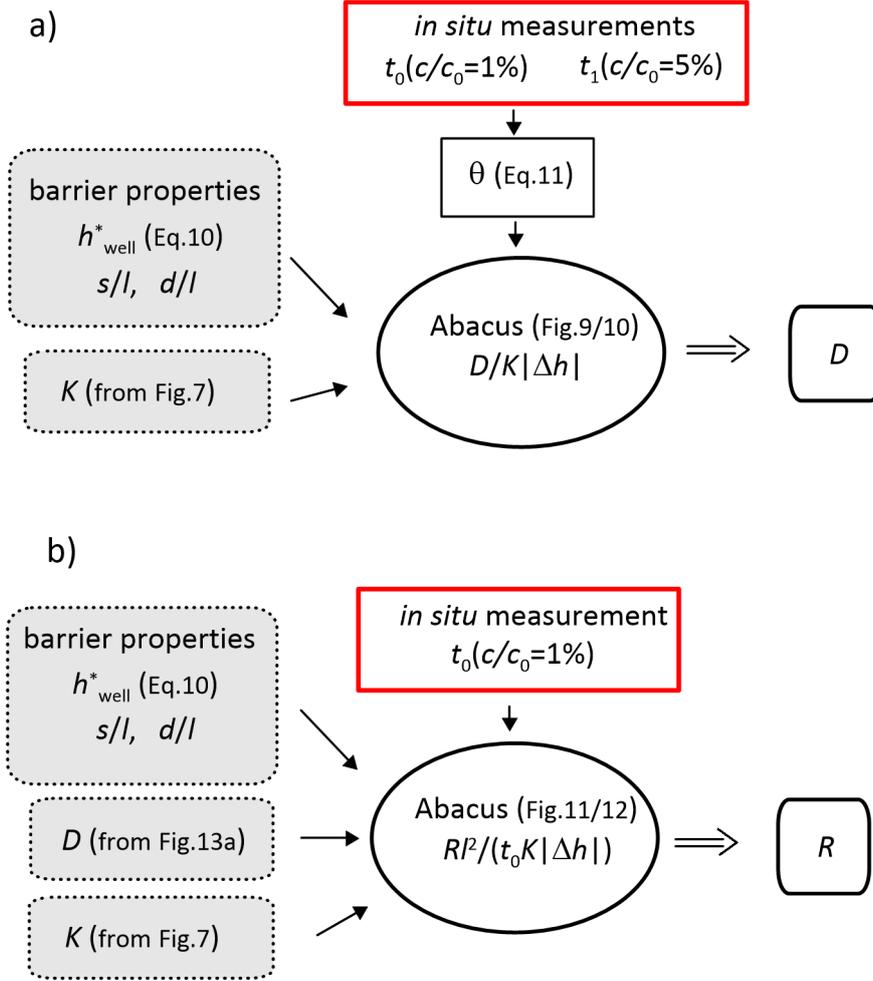


Figure 13: Procedure for the assessment of a) D and b) R .

5.3 Examples of application of the procedure

In this section, two numerical examples applied to a cutoff wall characterised by different hydraulic scenarios (HS1 and HS2) are illustrated. The geometrical properties and boundary conditions are given in Table 1.

Table 1: Geometry and material parameters used in the numerical examples (Scelsi *et al.*, 2019).

Wall thickness	l	0.4	m
Well diameter	d	8	cm
Well spacing	s	0.8	m
Inlet hydraulic head	h_{in} (HS1/HS2)	3.0/2.0	m
Outlet hydraulic head	h_{out} (HS1/HS2)	2.0/3.0	m
Well hydraulic head	h_{well}	2.0	m
Concentration in the polluted area	c_0	10	mol/m ³

Assume that for an hydraulic scenario HS1 a steady-state discharge of $q_{well} \sim 15.0$ cm²/d

per unit depth is measured *in situ* . Since the level of hydraulic head in the monitoring well is maintained constant, a pump will remove the exceeding volume of water. Measuring the volume lifted over the range of time, is possible to extrapolate a value of the water discharge q_{well} .

The procedure proposed in Figure 7 can be applied to estimate the hydraulic conductivity K :

1. once known the hydraulic head in the well and the hydraulic heads at the inlet and at the outlet (Tab.1), the normalized water head in the well $h_{well}^* = 0$ is calculated according to Eq. 3;
2. the dimensionless spacing $s/l = 2$ and diameter $d/l = 0.20$ of the well are calculated from the relevant values of barrier geometrical properties (Tab. 1);
3. by entering in the abacus of Figure 6a with $d/l = 0.20$ and $s/l = 2$, a value of the dimensionless ratio $\frac{K|\Delta h|}{q_{well}} \sim 0.6$ is obtained;
4. and the hydraulic conductivity of the barrier is $K = 0.6 \frac{q_{well}}{|h_{in} - h_{out}|} \sim 10^{-8}$ m/s.

Monitoring the value of hydraulic conductivity is thus possible and economical. This would allow evaluating with continuity the hydraulic performance of the barrier. If it changes with time, some process affecting the properties of barrier material is likely to be ongoing, *e.g.* due to chemo-mechanical interaction, and the engineers can take it as an alert to proceed with other interventions.

In the case of scenario HS1, the diffusive and advective fluxes combined and the condition is more critical the higher the hydraulic conductivity of the wall. However, foreseeing the time required for the contaminant to reach the aquifer requires the hydrodynamic dispersion and the retardation factor of the barrier, that can be evaluated by following the procedure described in Section 5.2. The time needed for the contaminant to cross the wall is generally considered of the order of tens of years (Manassero and Shackelford, 1994), thus the shorter the time needed to estimate backfill parameters, the longer the time available to update predictions and to implement maintenance operations. Assuming that, after a time $t_0 = 3$ years a concentration $c_{well} = 0.1$ mol/m³ (corresponding to $0.01c_0$) is measured, while a concentration $c_{well} = 0.5$ mol/m³ (corresponding to $0.05c_0$) is measured after $t_1 = 5$ years, the application of the procedure depicted in Figure 13 is as follow:

1. slope $\theta \sim 0.18$ is evaluated (from Eq.11);
2. from Figure 9, for $d/l = 0.2$ and $h_{well}^* = 0.0$ (first row, third column), the slope leads to a value of the non-dimensional group $D/(K|\Delta h|) \sim 0.5$;
3. multiplying the non-dimensional group by $K|\Delta h|$, the value of the hydrodynamic dispersion is obtained, $D \sim 5 \times 10^{-9} \text{ m}^2/\text{s}$;
4. from Figure 11, in correspondence to the plot of $d/l = 0.20$ and $h_{well}^* = 0.0$ (first row, third column), it is found that when $D/(K|\Delta h|) = 0.5$ the non-dimensional group $Rl^2/(t_0K|\Delta h|) \sim 30$;
5. by introducing the current values of t_0 , K and Δh , the value of the retardation factor is obtained $R \sim 180$.

In this example, the time required to have a complete characterization of material transport properties can be considered equal to 2 yrs ($= t_1 - t_0$), the numerical model could then be run with the transport parameters estimated *in situ* and the time required by the contaminant to cross the barrier updated. However, when the hydraulic scenario is the HS1 with $h_{well}^* < 0.5$, the time required for the contaminant to reach the aquifer, estimated when the average concentration of contaminant at the outlet $c_{out}/c_0 > 5\%$, is lower than if the well was not present (10 yr with well *vs.* 13 without well).

The same procedure is repeated for a case corresponding to hydraulic scenario HS2, in which $h_{in} = 2.0 \text{ m}$ and $h_{out} = 3.0 \text{ m}$ (see Table 1). In this scenario, the water flows towards the polluted area and the contaminant can reach the aquifer only if $D/(K|\Delta h|) > 1$, *i.e.* diffusion dominates over advection.

Although the direction of flow is the opposite, the geometry and type of boundary conditions for the hydraulic problem are the same as in the previous example, thus $K|\Delta h|/q_{well} = 0.60$. By assuming a steady-state discharge $q_{well} \sim 1.5 \text{ cm}^2/\text{d}$, the resulting hydraulic conductivity of the barrier is $K \sim 10^{-9} \text{ m/s}$. Assume, then, that a contaminant concentration $c_{well} = 0.1 \text{ mol/m}^3$ (corresponding to $0.01c_0$) is measured after $t_0 = 7$ years, while a concentration $c_{well} = 0.5 \text{ mol/m}^3$ (corresponding to $0.05c_0$) is measured after $t_1 = 19$ years. This sequence implies $\theta \sim 0.09$ (from Eq.11) and by using Fig. 10, *i.e.* the plot corresponding to $d/l = 0.2$ and $h_{well}^* = 1.0$ (first row, third column), to a value of the non-dimensional group $D/(K|\Delta h|) \sim 2$

388 from which the hydrodynamic dispersion $D \sim 2 \times 10^{-9}$ m²/s. From Figure 12 with $d/l = 0.20$
389 and $h_{well}^* = 1.0$ (first row, third column), $Rl^2/(t_0K|\Delta h|) \sim 55$ which gives $R \sim 60$.

390 The procedure requires the presence of the contaminant within the well, and for a given set
391 of parameters the migration of the contaminant is retarded when HS2 scenario is of concern.
392 Determination of the transport parameters requires longer times for this scenario (about 12 yrs
393 for the examined example).

394 Further, contrary to what seen for HS1, the presence of the well retards the arrival of the
395 contaminant to the aquifer in respect to the case of barrier without wells (23 yrs with well *vs.*
396 18 yrs of the case without well).

397 6 Conclusions

398 The site behaviour of cutoff walls is difficult to predict, due to the complexity and the variability
399 of the factors potentially modifying the transport properties of the backfill material. Presence of
400 defects and impurities related to the construction stage, cracks induced by the oscillation of the
401 groundwater level and chemical interactions related to the aggressive groundwater conditions
402 may serve as examples. This situation implies that *in situ* measurements of barrier transport
403 properties is fundamental for reliable predictions about contaminant transport. In this study,
404 the suitability of using monitoring wells cast in place when the backfill material is still in a
405 slurry state is analyzed, with the aim of providing a methodology to estimate the transport
406 properties of the barrier from site measurements. To this aim, several two-dimensional, finite-
407 element numerical simulations were performed, solving the water and contaminant mass balance
408 equations for boundary conditions relevant for cutoff walls and monitoring wells. In particular,
409 two hydraulic scenarios were considered: one corresponding to advection and diffusion both
410 directed from the inlet to the outlet, and the other with advection directed from the aquifer
411 into the contaminated area. Numerical results were then performed in order to develop non-
412 dimensional abaci, as fast and practical tools to estimate barrier transport properties from
413 periodic measurements performed in the monitoring wells, regardless of the type of mixture
414 adopted. In particular, the methodology allows for the determination of the average (field scale)
415 values of the barrier hydraulic conductivity, hydrodynamic dispersion and retardation factor,

416 as a function of (i) quantities easily measurable on site, like the discharge and the contaminant
417 concentration in the monitoring well; (ii) hydraulic head in the well, in the polluted area and in
418 the aquifer; (iii) the thickness of the cutoff wall; (iv) the spacing and the diameter of the wells.
419 The evolution of such field scale values might help practitioners and agencies in recognizing
420 a detrimental impact, that may require additional remedial actions to preserve the barriers
421 functionality.

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Appendix 1: Curing effect

In this Appendix, the effects of the time evolution of cement-bentonite hydraulic conductivity due to curing are considered. Inspired from the experimental data presented by Fratolocchi *et al.* (2006), the time needed for the backfill material to achieve a stable asymptotic hydraulic conductivity can be estimated to be ~ 300 days. According to this evidence, an empirical evolution law for hydraulic conductivity, calibrated on the experimental data of Fig.14 was implemented in the numerical simulations:

$$K(t) = K_f + K_0 t^{-\alpha} \quad (12)$$

where $K_0 = 10^{-5}$ m/s is the initial value of the hydraulic conductivity of the liquid slurry, $\alpha = 2.3$ describes the maturation velocity of the mixture and K_f is the asymptotic value of the hydraulic conductivity (equal to 10^{-9} m/s; 10^{-10} m/s; 10^{-11} m/s respectively for $K_1(t)$, $K_2(t)$ and $K_3(t)$ as shown in Fig.14). Fig.14b shows the breakthrough curves at the outlet of the barrier (having adopted material properties as reported in Tab. 1, boundary conditions as described in Fig.3, $D = 5 \times 10^{-10}$ m²/s and $R = 100$) by considering the time evolution of K (dashed lines), as well as the same simulation run considering a constant value of K , equal to the asymptotic one. In all the cases, simulating the curing process does not provide any appreciable difference in terms of model predictions: the curing time is in fact negligible with respect to the time of contaminant transport across the barrier ($\sim 10^2$ years for $K = 10^{-11}$ m/s, ~ 5 years for $K = 10^{-8}$ m/s).

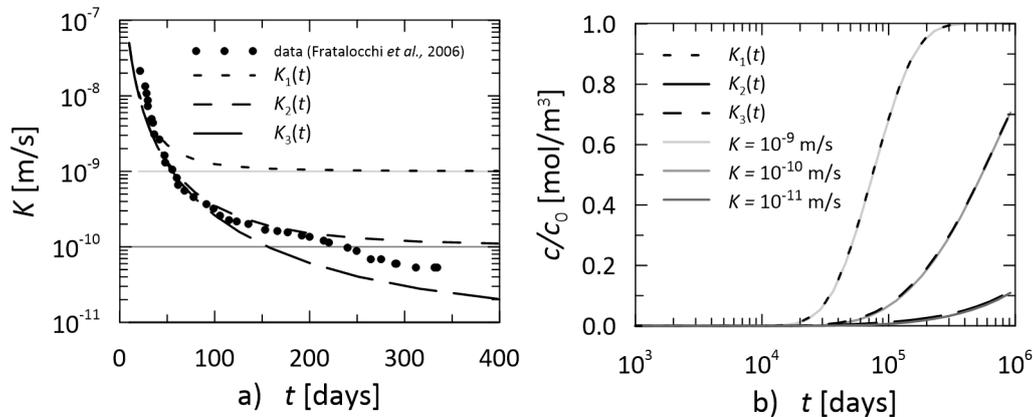


Figure 14: Effect of curing in cement bentonite mixtures: a) Evolution of hydraulic conductivity with time, b) breakthrough curves at the outlet of the barrier using different evolution laws for the hydraulic conductivity.

Appendix 2: Outlet boundary conditions

The choice of the boundary conditions to adopt for the solute mass balance equation was inspired by Brenner (1961) and Van Genuchten and Parker (1984), and are described in details on Section 3.3. However, Rabideau and Khandelwal (1998) also asserted that the most conservative boundary conditions to adopt for the solute transport through a cutoff wall are: a constant concentration BC at the inlet ($c(x = 0, t) = c_0$) and a zero concentration BC at the outlet ($c(x = l, t) = 0$), that maximize the concentration gradient between inlet and outlet, and inevitably induce a greater contaminant outflow flux. Assuming a Robin boundary condition at the inlet boundary, that ensures the conservation of mass contaminant, Figure 15 shows the results of simulations adopting a zero concentration gradient BC, compared to zero concentration BC at the outlet for the both hydraulic scenarios considered in the study. Figs 15a,b show the trends of the average value of the flux of contaminant across the outlet boundary with time: they are greater in the case of zero concentration BC, in accordance to what asserted by Rabideau and Khandelwal (1998). Figs 15c,d show the trend of the average contaminant concentration at the well boundary as a function of time. When a zero concentration BC is adopted at the outlet, the average value of contaminant concentration at the well boundary is as much mitigated as Pe is low. However, it is interesting to note that the first part of the trends, on which the method relies, for both the hydraulic scenarios and the Pe numbers, is not influenced by the type of outlet BC adopted. This leads to the univocal determination of the time t_0 and t_1 and of the slope θ of the trends (Eq.11), that are computed between $c_{dev}/c_0 = 1\% - 5\%$. Consequently the abaci presented in Figs. 9-12 are the same for both the type of outlet boundary conditions.

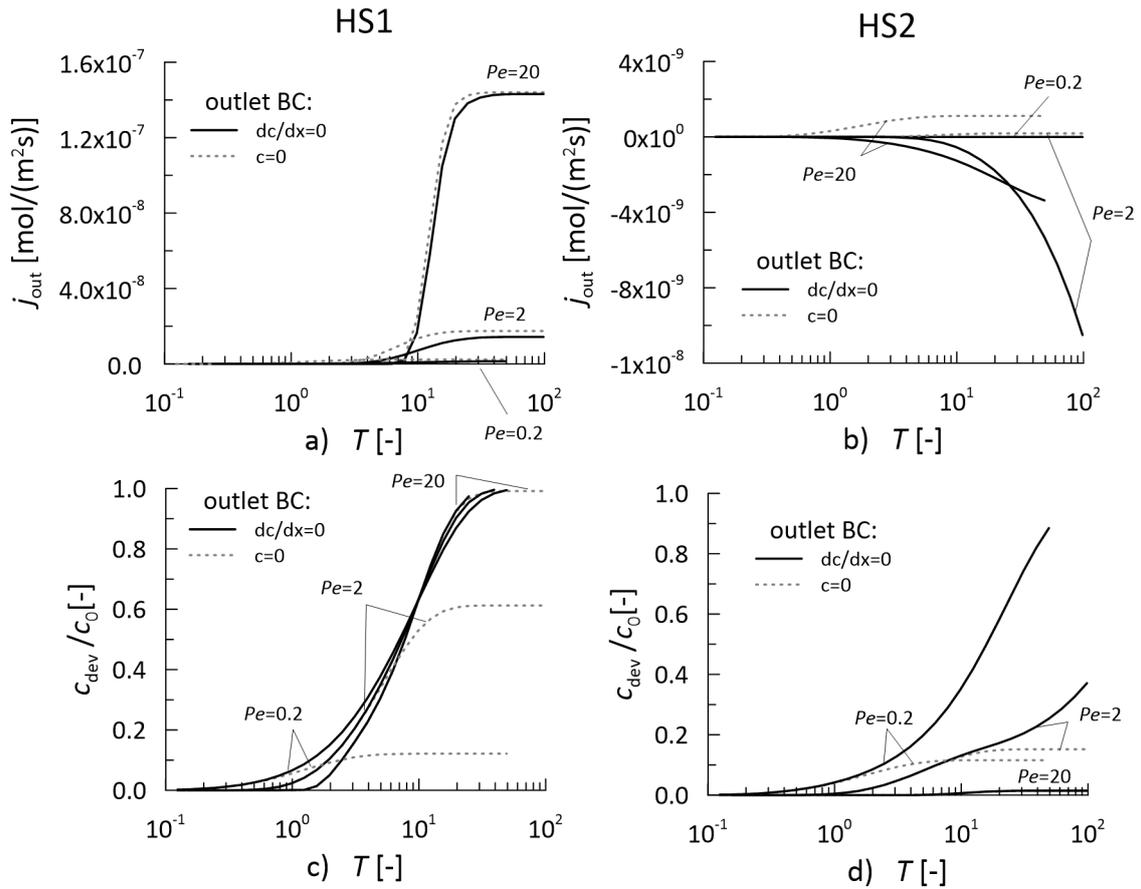


Figure 15: Comparisons results between zero gradient concentration and zero concentration boundary condition at the outlet. a,b) Trends of the average flux of contaminant along the outlet boundary for the hydraulic scenario a) 1 and b) 2; c,d) Trends of the average concentration of contaminant along the well boundary for the hydraulic scenario c) 1 and d) 2.