

1 **Title:** *Instability of flood embankments due to pore water pressure build-up at the toe: lesson*
2 *learned from the Adige river case study.*

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14

15 **Abstract**

16 The paper presents the case study of the Adige river embankment, a segment of which
17 experienced instability of the landside slope due to the development of uplift pressures. Soil
18 profile and hydro-mechanical properties of the embankment and foundation materials have
19 been assessed via site investigation, laboratory testing, and field monitoring for two cross
20 sections, within and outside the failure segment respectively. The hydro-mechanical model
21 developed thereof was first validated against its ability to reproduce the probability of failure
22 for the two sections with a FOSM-based approach. Comparison of water flow regimes between
23 the two sections was then used to highlight the importance of the hydraulic properties of the
24 material on the landside for the development of uplift pressures at the toe of the embankment.
25 The lesson learnt from this case study is that the hydraulic response of the ground on the
26 landside may play a critical role on the stability of flood embankments and its characterisation
27 should therefore not be overlooked when planning site investigation.

28

29 **Keywords**

30 *flood embankments; slope stability; uplift pressure; landside; case study*

31

32 **Introduction**

33 Flood embankments are essential structures in flood defence systems and their failure can lead
34 to devastating consequences. One of the most critical failure mechanisms is represented by the
35 instability of the landside slope triggered by the development of high uplift pressures at the toe
36 of the embankment (Phoon, 2008), often accompanied by the formation of sand boils (CIRIA;
37 French Ministry of Ecology; USACE, 2013). This is frequently the case when embankments
38 are built on top of foundation layers having significantly higher hydraulic conductivity.

39 The development of uplift pressure at the toe of the embankment can lead to failure by
40 triggering two different failure mechanisms; one is the piping process caused by seepage and
41 internal erosion, while the other is the instability caused by the increase of pore water pressure
42 and consequent decrease of shear strength of the soil (Dyer, 2004). The main failure mechanism
43 depends on whether the embankment is built directly on top of a permeable foundation soil, or
44 an intermediate impermeable layer is interposed between the permeable subsoil and the
45 embankment; in the first case, piping tends to be the prevailing failure mechanism, while in the
46 second case instability triggered by uplift pressures tends to be the dominant one (Hird, et al.,
47 1978).

48 In the literature there are only a few case studies documenting the development of uplift
49 pressure as the primary cause of instability of embankments. The first case study is found in
50 Cooling and Marsland (1953), who showed that the failure occurred in the embankment at
51 Dartfort Creek in 1953 was caused by the development of high pore water pressure in the
52 underlying layers of permeable sandy gravel that lead to a decrease in shear strength and
53 therefore instability of the landside slope. Uplift induced failure is also estimated to be the most
54 likely failure mechanism in the Western region of the Netherlands, where many flood
55 embankments are built on top of a very permeable sand layer (Bauduin, et al., 1989; Van, et
56 al., 2005). Although this kind of stratigraphy is not uncommon, Van et al. (2005) suggest that

57 one of the reasons why only two case studies are reported in the literature is that this failure
58 mechanism has not been recognised in other occasions, either because of the damage resulting
59 from the failure or because of unawareness.

60 An exception in more recent years is represented by the catastrophic breach on the North
61 London Canal in New Orleans during Hurricane Katrina in 2005, which resulted in the flooding
62 of the most densely populated area of New Orleans. The instability was caused by uplift
63 pressures at the toe of the embankment, which was built over a foundation layer of loose sand
64 (Seed, et al., 2008). While Kanning et al. (2008) pointed out that there is still some uncertainty
65 about the reasons why the failure occurred exactly at that location and not in any other sections
66 with a similar soil profile, Seed et al. (2008) concluded that the only explanation was to be
67 found in the subtle difference in soil profile in the foundation layer at the toe of the
68 embankment. In the failed section, the interface between the layers had a gentle slope and was
69 almost horizontal, while on the opposite bank, the presence of a steeper interface between the
70 layers altered the shape of the slip surface and enhanced the stability of the embankment.

71 The aim of this work is to show that the high contrast in hydraulic conductivity between a
72 pervious foundation layer and a relatively impervious embankment material does not represent
73 per se a critical condition for the development of uplift pressures and to highlight the key role
74 played by the soil profile on the landside even outside the footprint of the embankment.

75 This paper presents a case study associated with a segment of the Adige River embankment
76 subject to instability during a flood in 1981 for the first time ever since construction, between
77 1860 and 1890 (Werth, 2003). This segment experienced instability with a failure mechanism
78 on the landside (Pozzato, et al., 2014), likely associated with uplift pressures as boiling is often
79 observed during intense flood events. Instability was characterised by the formation of a scarp,
80 but the embankment did not experience a full collapse. As a result, the materials and soil profile
81 to date are exactly the same as at the time of failure. This offers the unique chance to

82 characterise the soil profile and hydro-mechanical properties of the embankment and its
83 foundation as they were at the time of failure. This is rarely the case as instability is often
84 accompanied by a breach with the embankment and the foundation layers swept away, making
85 soil profile and material characterisation impossible to reconstruct a posteriori. Field and
86 laboratory testing was carried out to characterise stratigraphy and soil's hydro-mechanical
87 properties for two cross sections, one within and one outside the failure segment. The hydro-
88 mechanical model was first validated against its capability to reproduce realistic probability of
89 failure within and outside the failure segment and then used to highlight critical aspects of the
90 failure mechanism.

91 **The case-study**

92 The embankments on the Adige River¹ were built at the end of 19th century to straighten the
93 river path. The traces of the ancient meanders are still visible along the alluvial valley and are
94 easily recognisable from aerial photographs and satellite images (Angelucci, 2013). These
95 resources can be coupled with historical cartography to reconstruct the ancient meandering
96 path of the river (Fig. 1).

97 During an intense flood event in July 1981, a 230 m segment of the embankment near the
98 village of San Floriano experienced instability and a 50 cm deep scarp was observed on the
99 crest of the embankment². The probability of failure has been assessed for this segment as well
100 as for a section in the south outside the failure segment ('stable' in Fig. 1).

101 **Soil profile**

102 Soil profile has been inferred from boreholes and Dynamic Probing Heavy (DPH) tests from
103 the crest at different locations along a 500 m segment, which includes the failure segment and
104 the stable zone south of the failure segment. Layer boundaries inferred from visual inspection

¹ Northern Italy in Figure S1.

² Picture of the scarp in Figure S2.

105 of core samples and DPH blow number are shown as diamonds and circles respectively in Fig.
106 2. The soil profile was then cross-checked via the grain-size distribution (GSD) of samples
107 taken from the identified soil horizons (black rectangles in Fig. 2).

108 The body of the embankment is made of two different layers. The upper layer is a gravelly
109 shell, whereas the core of the embankment is a brown sandy silt. The thickness of the
110 embankment core layer is fairly constant (~6 m) along the 500 m segment. The thickness of
111 the gravelly shell is slightly larger in the area located north of the failure zone, closer to San
112 Floriano Bridge, but it is fairly constant (~1.1 m) in the area of the failure segment and outside
113 the failure segment in the south.

114 Two layers form the embankment foundation. The first layer is a brown-grey sandy material,
115 with significant coarse fraction and rounded particles. This material has alluvial origin and
116 corresponds to the ancient riverbed where the Adige River flowed before being straightened.
117 This material is also encountered outside the ancient riverbed projection as derived from aerial
118 photographs and historical maps (Fig. 1). This is because the alluvial deposit extends beyond
119 the ancient riverbed on the inner side of the meander due to deposition phenomena. The deep
120 foundation layer is a dark grey sandy deposit, with local lenses of finer material. It constitutes
121 the glacial lacustrine deposit where the Adige River formed its meandering path.

122 The two cross sections examined in this study are located at chainage km 122.25 (within the
123 failure segment) and at chainage km 122.42 (outside the failure segment). After the failures
124 observed in 1981 the entire segment of embankment has been reinforced by a berm on the
125 landside slope³ which has not been included in the geotechnical model in this study.

126 Only two boreholes, B131 and B132 for the sections within and outside the failure segment
127 respectively³, have been drilled on the landside at the toe of the embankment down to 4m.

³ Topography in Figure S3.

128 Additional information was therefore required to characterise the soil profile on the landside.
129 Investigation was carried out using EM profiling based on Slingram method (Nabighian, 1992)
130 using the device GEM2 (GEOPHEX USA) along the toe of the embankment. Results are shown
131 in Fig. 3. The sharp local anomalies of resistivity (chainage km 122.19, 122.28, 122.35) are
132 associated with the presence of artificial metal objects on the surface. Within the failure
133 segment, apparent resistivity is essentially constant with a slightly increasing trend from
134 chainage km 122.24 to km 122.39. Outside the failure segment the resistivity increases, more
135 markedly from chainage km 122.41. This is associated with the appearance of the sandy
136 alluvial deposit generated by the ancient river, which is close to the embankment in the south
137 section. The alluvial deposit on the landside therefore appears in the south but not in the failure
138 segment, as reflected in the soil profile for the two cross-sections (in Fig. 4).

139 Grain size distributions for the identified materials are shown in Fig. 5. The embankment core
140 is fairly homogeneous within and outside the failure segment. The alluvial material and the
141 lacustrine material show larger variability along the longitudinal profile (Fig. 5b).

142 The grain size distribution of the sample collected at a depth of 3.5m from the only borehole
143 on the landside in the section outside the failure segment (B132B) is consistent with the grain
144 size distribution of the alluvial material. This confirms that the alluvial deposit extends beyond
145 the toe of the embankment in the section located south of the failure segment.

146 **Hydro-mechanical characterisation**

147 Laboratory testing was carried out to investigate shear strength, saturated hydraulic
148 conductivity, and water retention behaviour of the embankment material. Cell piezometers and
149 tensiometers were installed in the zone below and above phreatic surface respectively and their
150 measurements were used to characterise the hydraulic properties of the embankment and the
151 shallow foundation layer by inverse analysis of hydraulic flow.

152 ***Deterministic hydraulic characterisation***

153 *Water retention behaviour of embankment material*

154 Water retention behaviour of the embankment material was determined from loose samples
 155 reconstituted in the laboratory by compaction at target dry density of 1.53 g/cm³ consistent
 156 with the estimated field value. The density index of the embankment material was first
 157 estimated based on SPT and DPH tests (EN 1997-2, 2007). The density index was found to be
 158 in the range 0.19-0.32. Assuming a minimum and a maximum void ratio $e_{\min}=0.30$ and
 159 $e_{\max}=0.90$ (Lambe & Whitman, 1969), the dry density was therefore estimated to be in the
 160 range 1.50-1.57 g/cm³. Specimens 100 mm diameter and 100 mm high were compacted to
 161 gravimetric water contents ranging from 9% to 21%. After moisture equilibration overnight,
 162 matric suction was measured using a High-Capacity Tensiometer (Tarantino & Mongiovi,
 163 2003). Water retention data points of compacted samples can be associated with the main
 164 wetting curve (Tarantino & Tombolato, 2005), which is reasonably representing water
 165 retention behaviour for infiltration associated with the flood.

166 The experimental data points have been fitted with van Genuchten model for water retention
 167 (van Genuchten, 1980). The effective saturation S_e is defined by Eq. 1:

$$S_e = \frac{\theta - \theta_{res}}{\theta_{sat} - \theta_{res}} = \frac{1}{(1 + (\alpha s)^n)^m} \quad \text{Eq. 1}$$

168 where s is the suction, θ , θ_{sat} and θ_{res} are the current, saturated, and residual volumetric water
 169 contents respectively, α and n are soil parameters ($\alpha=0.07$ kPa⁻¹ and $n=1.438$) and $m = 1-1/n$.

170 The relative hydraulic conductivity was derived from S_e as per Eq. 2 (van Genuchten, 1980).

$$k_{rel} = S_e^{-0.5} \left(1 - \left(1 - S_e^{\frac{1}{m}} \right)^m \right)^2 \quad \text{Eq. 2}$$

171 *Saturated hydraulic conductivity of foundation layers via laboratory testing*

172 Two specimens 80 mm diameter and 20 mm high were cut from samples taken from the alluvial
173 deposit and lacustrine deposit at the depths of 7.3-7.7 and 12.7-13.00 respectively (Borehole
174 B551). The specimens were consolidated in an oedometer to 160kPa and 250 kPa for the 7.3-
175 7.7 and 12.7-13.00 samples respectively to reproduce field effective stress (Aldegheri, 2009).
176 Constant head hydraulic conductivity tests were carried out by connecting the base of the
177 oedometer to a water reservoir placed on a balance to measure water flow rate. The values
178 obtained for the saturated hydraulic conductivity were $8 \cdot 10^{-6}$ m/s and $4 \cdot 10^{-8}$ m/s for alluvial and
179 lacustrine deposit respectively. The values were not considered representative of the field
180 hydraulic conductivity in the sense that differences of 1-2 order of magnitude are generally
181 observed between laboratory and field values (Herzog & Morse, 1986). Nonetheless, the ratio
182 between these two laboratory values was assumed to be representative of the same ratio of field
183 values.

184 *Saturated hydraulic conductivity of embankment and upper foundation via inverse analysis*
185 *of piezometer and tensiometer data*

186 A monitoring system has been installed at the section outside the failure segment consisting of
187 i) 6 pressure transducers (B1-MOD level transducer from Tecnopenta) installed via bayonet
188 fitting at the bottom of standpipe piezometers to measure pore-water pressure below the
189 phreatic surface mainly in the alluvial deposit and ii) 5 tensiometers (T8 tensiometer from
190 UMS) to measure suction above the phreatic surface (Fig. 4).

191 The calibration of the TecnoPenta pressure transducers was verified in the field by moving the
192 pressure transducer at different locations in the standpipe piezometers (filled with water once
193 the piezometer was removed from its bayonet fitting). The calibration curve of the UMS

194 tensiometers was verified in the field by submerging the porous tip into a water-filled container
195 and imposing vacuum via a hand-operated pump.

196 Measurements recorded by the instruments were used to characterise the saturated hydraulic
197 conductivity of the embankment and the alluvial deposit by inverse analysis. The time selected
198 for the inverse analysis is a two-week window in summer 2016, where the Adige River
199 recorded its highest level since the installation of the instruments⁴.

200 Measurements by the shallow tensiometer T6, closer to the ground surface, were clearly
201 affected by the rainfall, whereas measurements by the intermediate and deep tensiometers were
202 not⁵. Atmospheric boundary conditions on the crest of the embankment did not have a
203 significant effect on the seepage in the embankment core below shallow depths. At the same
204 time, intermediate and deeper tensiometers clearly responded to the fluctuation of the river
205 level, which was considered as the hydraulic boundary condition in the inverse analysis.

206 The saturated hydraulic conductivities of the embankment and the alluvial deposit were
207 selected to allow for the best matching between simulated and observed data. In lack of data,
208 the ratio between the hydraulic conductivities in vertical and horizontal direction has been
209 assumed equal to $k_V/k_H=0.1$ for the foundation layers (Lancellotta, 2009).

210 The comparison between simulated and measured pore-water pressure values was considered
211 satisfactory. The maximum difference between measured and simulated values was less than 4
212 kPa for the piezometers and less than 3 kPa for the tensiometers. A sample showing results for
213 two of the tensiometers and one piezometer is reported in Fig. 6⁶. The values of saturated
214 hydraulic conductivity derived from the inverse analysis are $2 \cdot 10^{-6}$ m/s and $4 \cdot 10^{-3}$ m/s for the
215 embankment material and alluvial deposit respectively. It is worth observing that the saturated
216 hydraulic conductivity for the alluvial deposit material derived from the inverse analysis is

⁴ River level variations in Figure S4.

⁵ Measurements in Figure S5.

⁶ Full set of data in Figure S6.

217 greater than the one measured in the laboratory tests by about two orders of magnitude, which
218 is a difference typically encountered when comparing laboratory and field measurements of
219 hydraulic conductivity.

220 ***Deterministic shear strength characterisation***

221 Shear strength behaviour of the embankment material was determined on two sets of
222 specimens. Two specimens from within the failure segment (B551, depth 4.20 - 4.80m) were
223 reconstituted from slurry. After flooding the shear box container with water to submerge the
224 sample, the two specimens 20 mm high were initially consolidated to a maximum vertical stress
225 of 75 kPa and 150 kPa respectively. The target vertical stress was attained in stages and the
226 specimens were allowed to consolidate fully under each vertical stress increment. The time
227 required to achieve primary consolidation was always less than 1 min. Specimens were sheared
228 at shear displacement rate of 2mm/h. This rate was sufficiently slow to ensure shear under
229 drained conditions (Aldegheri, 2009). Both specimens showed a monotonic increase in shear
230 strength until the ultimate state with compressive behaviour.

231 Two specimens from outside the failure segment (B739, depth 3.0-3.3 m) were compacted into
232 the shearbox to vertical stress of 200 kPa and water content of 18% and 21% respectively to
233 achieve a target dry density similar to the specimens prepared for the water retention behaviour.
234 Specimens were then unloaded to 100 kPa and 50 kPa vertical stress respectively to simulate
235 field stress conditions. Afterwards, the specimens were saturated by flooding the shearbox
236 external container sheared at displacement rate of 1.6 mm/h. All specimens showed a
237 monotonic increase in shear strength until the ultimate state with compressive behaviour. The
238 specimens from within and outside the failure segment aligned to the same failure envelope in
239 the Mohr-Coulomb plane (Fig. 7) characterised by a friction angle equal to 28.9°.

240 ***Probabilistic modelling of hydro-mechanical properties***

241 Material properties have to be characterised in probabilistic terms in order to calculate the
242 probability of failure of the embankment. The experimental characterisation in this study did
243 not provide sufficient data to develop a full probabilistic model for the hydro-mechanical
244 properties of the embankment. The values obtained from the experimental hydro-mechanical
245 characterisation were therefore assumed as mean values, while the standard deviation was
246 estimated on the basis of published values of coefficients of variations COV (ratio of the
247 standard deviation to the mean value) as suggested by Duncan (2000).

- 248 • *Friction angle*: COV equal to 13%, in the range 2-13% suggested by Duncan (2000).
- 249 • *Saturated hydraulic conductivity*: COV equal to 90%, consistent with literature values
250 ranging from 10% (Nguyen & Chowdhury, 1985) to 160% (Zhang, et al., 2005).
- 251 • *Water retention curve*: COV of 60% and 9% were assumed for the α and n parameters in
252 van Genuchten model respectively after Likos et al. (2014). The parameters α and n were
253 assumed to be independent consistently with their physical meaning, being α related to the
254 largest pore size in the material and n to the pore size distribution.

255 The saturated hydraulic conductivity of the foundation was not considered in probabilistic
256 terms because preliminary studies on the embankment in San Floriano⁷ showed that its
257 variability has little influence on the variability of the Factor of Safety. The mechanical
258 properties of the foundation layers do not play any role in the stability analysis because slip
259 surfaces are not deep enough to reach the foundation layers; a deterministic friction angle equal
260 to 33° has been assumed for both materials.

261 A LogNormal distribution was assumed for all variables. Normal or LogNormal distributions
262 are usually selected for the probability distribution of soil hydro-mechanical properties (Arnold

⁷ Amabile, A.; Cordão-Neto, M.P.; Pozzato, A.; Tarantino, A. *An accessible approach to assess the probability of failure of flood embankments taking into account transient water flow*. Submitted paper.

263 & Hicks, 2011), (Suchomel & Mašín, 2010), (Malkawi A.I.H., 2000). The LogNormal
 264 distribution has the advantage of never taking any negative values (Uzielli, et al., 2007), which
 265 is consistent with most soil properties.

266 **Deterministic assessment of embankment stability**

267 *Seepage analysis*

268 The finite element software SEEP/W (GEO-SLOPE, 2004) was used to analyse transient
 269 saturated/unsaturated seepage in the embankment and its foundation. The governing equation
 270 in SEEP/W is Richards equation (Richards, 1931), which describes two dimensional flow in
 271 unsaturated soils as shown in Eq. 3:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial h}{\partial y}\right) + Q = \frac{\partial \theta}{\partial t} \quad \text{Eq. 3}$$

272 Where x and y are spatial coordinates, θ is the volumetric water content, h is the hydraulic head,
 273 k_x and k_y are a function of θ and represent the hydraulic conductivities in the x and y direction
 274 respectively, Q is water flux and t is the time.

275 The boundary condition on the riverside was represented by the time-dependent hydraulic head
 276 (i.e. the hydrograph) recorded during the flood event in 1981, as shown in Fig. 8. The vertical
 277 boundary on the landside and the bottom boundary were modelled as impermeable boundaries.

278 The distance of the landside vertical boundary from the toe of the embankment was set equal
 279 to 80m, large enough so that the pore-water pressure distribution up to 10 m from the toe of
 280 the embankment is not affected by the boundary condition on the landside. The vertical
 281 boundary on the riverside is modelled as an impermeable boundary being an axis of symmetry.

282 The crest of the embankment, the landside slope and the ground surface are modelled as
 283 potential seepage faces, where pressure can never exceed atmospheric pressure.

284 The initial condition has been obtained from a steady-state seepage analysis associated with
 285 the initial river level for the flood event. In the steady state analysis the far field boundary

286 condition on the vertical boundary on the landside is represented by a constant head equal to
287 211.45m. This corresponds to the average measurement of water level in a well located 80m
288 from the toe of the embankment observed in winter when the baseline river level was lower
289 than the level in the well. The level in the well was therefore attributed to the far-field
290 groundwater table.

291 An unstructured mesh of quadrilateral and triangular elements was adopted for the entire
292 domain. The mesh density in the regions where higher gradients develop was optimised by
293 reducing the element size until no significant change in pore-water pressure was observed (~0.5
294 kPa). For the embankment core, the alluvial deposit and the gravelly shell, elements with size
295 equal to 0.4m were adopted, while 1m elements were used for the lacustrine foundation layer.
296 A constant time step of one hour was used for the entire duration of the seepage simulation
297 (four days). The optimal time step duration was selected with the same approach adopted for
298 the mesh density, by reducing an initial time step of 3 hours until no significant change in pore
299 water pressure was observed (~0.5 kPa).

300 ***Stability analysis***

301 The stability analysis was carried out using Bishop's simplified method (Bishop, 1955). The
302 iterative procedure to calculate the Factor of Safety was completed with the software
303 SLOPE/W (GEO-SLOPE, 2004). The pore-water pressures from the transient seepage analysis
304 were used to calculate the evolution of the Factor of Safety over the duration of the flood event.
305 The unsaturated shear strength model (Vanapalli, et al., 1996) implemented by the software
306 was considered appropriate for the embankment coarse-grained material (Tarantino & El
307 Mountassir, 2013). The critical slip surface was assumed circular and then refined with the
308 optimisation algorithm based on the segmental technique.

309 **Probabilistic assessment of embankment stability**

310 The probability of failure of the embankment has been calculated with a probabilistic approach
 311 based on the First Order Second Moment (FOSM) method (Wolff, et al., 2004). The Factor of
 312 Safety is derived as a Taylor's series expansion with the first order terms of the series used to
 313 calculate the mean and variance as a function of the input variables X_1, \dots, X_n . For uncorrelated
 314 input variables the mean $\mu[FS]$ and the variance $\sigma^2[FS]$ are given by Eq. 4 and Eq. 5
 315 respectively:

$$\mu[FS] \cong FS(\mu[X_1], \mu[X_2], \dots, \mu[X_n]) \quad \text{Eq. 4}$$

$$\sigma^2[FS] \cong \sum_1^n \left(\frac{\partial FS}{\partial X_i} \right)^2 \sigma^2[X_i] \quad \text{Eq. 5}$$

316 The partial derivatives in Eq. 5 were estimated numerically with a finite difference method by
 317 choosing the increment of each input variable equal to its standard deviation. The probability
 318 of failure corresponds to the probability of having a Factor of Safety lower than or equal to
 319 unity. The application of the FOSM method on its own only provides information about the
 320 mean and variance of the Factor of Safety. A probability distribution function of the Factor of
 321 Safety must be assumed a priori in order to calculate the probability of failure, very often
 322 normal (Baecher & Christian, 2005) or LogNormal (Duncan, 2000). In order to overcome this
 323 limitation, the appropriate probability distribution function for the Factor of Safety has been
 324 selected with the same approach proposed in Amabile et al.⁷. The selection of the probability
 325 distribution function is based on the application of the Monte Carlo method for a single input
 326 variable. Different probability distribution functions were fitted to the Monte Carlo-derived
 327 empirical distribution function and the Normal distribution function, which returned the best
 328 match of the value of the empirical probability of failure, was selected.

329 **Application and results**

330 The number of simulations required for the application of the FOSM method is $2n+1$, where n
331 is the number of independent input variables X_i . Four independent input variables were
332 considered in this study, corresponding to as many material properties (k_{sat} , ϕ' , α , n). In each
333 simulation the minimum value of the Factor of Safety over time FS_m has been obtained⁸. For
334 both cross sections, the minimum Factor of Safety FS_m is fairly linear with respect to the input
335 variables, thus complying with the implicit assumption of the FOSM method that considers
336 only first order terms of the Taylor's series expansion. The mean value of the minimum Factor
337 of Safety $\mu[FS_m]$ corresponds to the result of a deterministic analysis and is associated with the
338 simulation where all input variables are taken with their mean value (Eq. 4). In the remaining
339 $2n$ simulations one variable at a time is increased or decreased by adding or subtracting its
340 standard deviation $\sigma[X_i]$ to its mean value $\mu[X_i]$. The values of minimum Factor of Safety FS_m
341 obtained from these simulations are used to calculate the variance of the minimum Factor of
342 Safety $\sigma^2[FS_m]$ (Eq. 5). The mean and variance of the minimum Factor of Safety FS_m within
343 the failure segment are equal to 1.162 and 0.044 respectively, while outside the failure segment
344 they are equal to 1.631 and 0.112 respectively.

345 The value of the probability of failure is calculated as the probability to have a Factor of Safety
346 lower than or equal to one and is graphically represented by the shaded areas in Fig. 9. The
347 values of the probability of failure calculated for the section within and outside the failure
348 segment are equal to 22.1% and 2.96% respectively. These values are consistent with the
349 expected probabilities of failure. The probability of failure of the section outside the failure
350 segment is one order of magnitude lower and within the acceptable values reported in the
351 literature for similar cases (Chowdhury, 2010). The probability of failure of the section within

⁸ Values in Table S1.

352 the failure segment, on the other hand, is well outside the acceptable range, thus confirming
353 the validity of the model.

354 **Discussion**

355 Results of the seepage analysis in terms of hydraulic head contours are reported in Fig. 10.
356 Results are shown for the time step corresponding to the peak of the hydrograph in the
357 simulation that resulted in the minimum value of Factor of Safety for both sections. For the
358 section within the failure segment the phreatic surface (bold line in Fig. 10) reaches the ground
359 surface on the toe of the embankment, while for the cross section outside the failure segment
360 positive pore water pressure does not reach the toe of the embankment as the phreatic surface
361 remains below the ground surface by about 0.5m.

362 The shape of the phreatic surface is also different for the two sections. The phreatic surface
363 lowers down monotonically outside the failure segment whereas it 'bulges' in proximity of the
364 embankment toe within the failure segment. This bulging is due to the upward flow originating
365 from the foundation layer as shown by the velocity vectors in Fig. 10. In turn, this is generated
366 by the sharp contrast in hydraulic conductivity between the alluvial material below the
367 embankment ($k=4\cdot 10^{-3}$ m/s) and the lacustrine material on the landside ($k=2\cdot 10^{-5}$ m/s). The
368 lacustrine material acts as a barrier diverting the water flow upward towards the embankment
369 toe. On the other hand, water flow is not diverted in the section outside the failure segment
370 because there is no contrast in hydraulic conductivity in the horizontal direction.

371 The different water flow pattern affects the distribution of hydraulic head at the embankment
372 toe. In the failure segment, hydraulic head is not dissipated in the alluvial material below the
373 embankment because its hydraulic conductivity is much higher than the adjacent lacustrine
374 deposit. This makes available the full hydraulic head at the base of the embankment and water
375 flow therefore occurs upwards with relatively high hydraulic gradients. This does not occur

376 outside the failure segment as the hydraulic head dissipates uniformly in the homogenous
377 alluvial deposit.

378 In turn, the different distribution of hydraulic head and, hence, pore-water pressures affect the
379 stability of the embankment. In the section within the failure zone, the build-up of pore water
380 pressure at the toe of the embankment leads to decrease in the shear strength and lower Factor
381 of Safety. Results of the stability analysis resulting in the minimum Factor of Safety FS_m are
382 shown in Fig. 11.

383 The critical slip surface obtained in the mean value (deterministic) simulation has been
384 considered in all the other simulations (Phoon, 2008). Its shape and position compare very
385 favourably with the scarp observed during the flood in 1981, when a cut of about 50 cm of
386 depth was observed on the crest of the embankment.

387 The variation of Factor of Safety over time for the mean value simulation is shown in Fig. 12.
388 The minimum Factor of Safety FS_m is not attained at the same time in both sections. It
389 corresponds to the time of maximum river level ($t = 1.125$ days) for the section within the
390 failure segment and $t = 1.500$ days for the section outside the failure segment. This can be
391 explained by the results of the seepage analysis. The Factor of Safety depends on the value of
392 the pore water pressure along the slip surface. In the section within the failure segment the
393 variation of pore water pressure is immediately affected by the variation of river level because
394 of the high hydraulic transmissivity of the confined alluvial layer beneath the embankment.
395 This is shown by the fact that the maximum pore water pressure for a point at a depth of 0.5 m
396 below the toe of the embankment corresponds to the peak of the hydrograph (Fig. 12). In the
397 section outside the failure segment pore water pressure along the slip surface is less affected
398 by variations in river level, even if the foundation material is extremely permeable, because
399 water tends to flow towards the landside. The value of pore water pressure along the slip surface
400 increases more slowly because water flow is taking place in the embankment from the river,

401 not from the foundation layer, therefore the seepage process is governed by the hydraulic
402 conductivity of the unsaturated embankment material. For this reason it takes some time for
403 the water front to reach the slip surface, resulting in a delay between the peak of the hydrograph
404 and the time when the maximum pore water pressure and, hence, the minimum Factor of Safety
405 is attained.

406 The conclusion drawn by this comparison is that the stability of the embankment is strongly
407 affected by the water flow regime, which is in turn strongly controlled by the hydraulic
408 conductivity of the material on the landside of the embankment (and not just by the hydraulic
409 conductivity of the material directly beneath the embankment). In the section outside the failure
410 segment the presence of a permeable foundation layer on its own is not a decisive cause for the
411 development of uplift pressure.

412 These findings seem to be consistent with previous literature. Seed et al. (2008) emphasized
413 the key role played by the soil profile beneath the toe of the North London Canal embankment
414 in New Orleans when comparing the behaviour of two sections with very similar soil profiles.
415 Although both embankments were built on very permeable foundation layers, only one of them
416 collapsed during Hurricane Katrina. Their results highlighted that a subtle difference in soil
417 profile even beyond the toe of the embankment can have catastrophic consequences in terms
418 of embankment stability. The results presented in this work about the case study of the Adige
419 River embankment confirm that instability triggered by uplift pressures is a complex failure
420 mechanism and the critical role played by the foundation material on the landside cannot be
421 adequately captured by approaches that assume horizontally layered soil profiles.

422 **Conclusions**

423 The paper has presented the case study of the embankment instability along the Adige River.
424 A segment of this embankment experienced instability in the form of a scarp without
425 collapsing, thus offering the chance to investigate soil profile and material properties as they

426 were at the time of instability. Soil profile within and outside the failure segment has been
427 inferred from boreholes, DPH tests and EM scanning. The hydro-mechanical properties of the
428 embankment and its foundation have been characterised through laboratory tests and inverse
429 analysis of water flow based on field measurements below and above phreatic surface.

430 The hydro-mechanical model has been validated by calculating the probability of failure within
431 and outside the failure segment with a FOSM-based approach. The calculated probability of
432 failure has shown a good agreement with the expected probability of failure for the sections
433 within and outside the failure segment.

434 The analysis of the flow regime within the segment that experienced failure has shown that the
435 contrast in hydraulic conductivity in the foundation layers on the landside leads to an upward
436 diversion in the water flow and build-up of pore-water pressures at the toe of the embankment.
437 On the other hand, the zone outside the failure segment is characterised by homogeneous
438 foundation layers and the water flow towards the landside is undisturbed, with dissipation of
439 hydraulic head taking place in horizontal direction in the foundation layers beneath the
440 embankment and on the landside.

441 These results show that the high hydraulic conductivity of the foundation layer does not
442 represent per se a critical condition for the development of uplift pressures at the toe of the
443 embankment. The material outside the embankment footprint can play indeed a key role on the
444 water flow regime and, hence, on the stability of flood embankments. The role of the soil profile
445 on the landside is often overlooked and should be addressed by site investigation.

446

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455 **Supplemental data**

456 Supplemental data are provided in the attached file.

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562 Captions of Figures

- 563 Fig. 1. Ancient meanders of the riverbed before the construction of flood embankments (Werth, 2003).
564 Fig. 2. Longitudinal soil profile: layer boundaries identified from visual inspection of the borehole logs
565 (diamonds) and DPH tests (circles), samples collected for grain size analysis (black rectangles) and studied
566 sections within and outside the failure segment (dashed lines).
567 Fig. 3. EM measurements along the longitudinal profile taken on the landside at the toe of the embankment.
568 Fig. 4. Soil profile for the section within (left) and outside (right) the failure segment.
569 Fig. 5. Grain size distributions for (a) embankment core, (b) alluvial deposit, (c) lacustrine deposit (grey =
570 samples within failure segment; white = samples outside failure segment).
571 Fig. 6. Comparison between measured (continuous lines) and simulated (dashed lines) values of (a) pore
572 water pressure in tensiometers and (b) hydraulic head in piezometers.
573 Fig. 7. Failure envelope obtained from direct shear tests on two sets of specimens from the embankment
574 core.
575 Fig. 8. Hydrograph recorded during the flood event in 1981.
576 Fig. 9. Probability of failure for the sections within and outside the failure segment.
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578 within (left) and outside (right) the failure segment at the time of peak.
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582 (depth = 0.5 m) compared to flood hydrograph.
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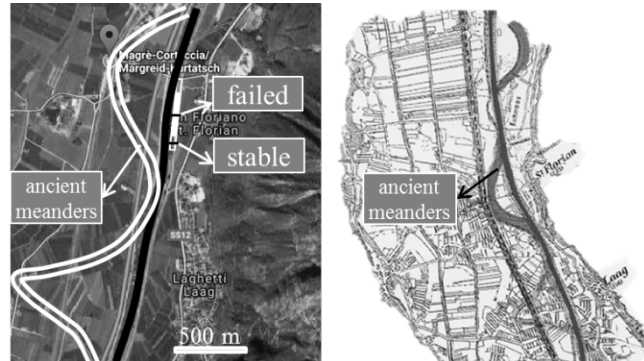


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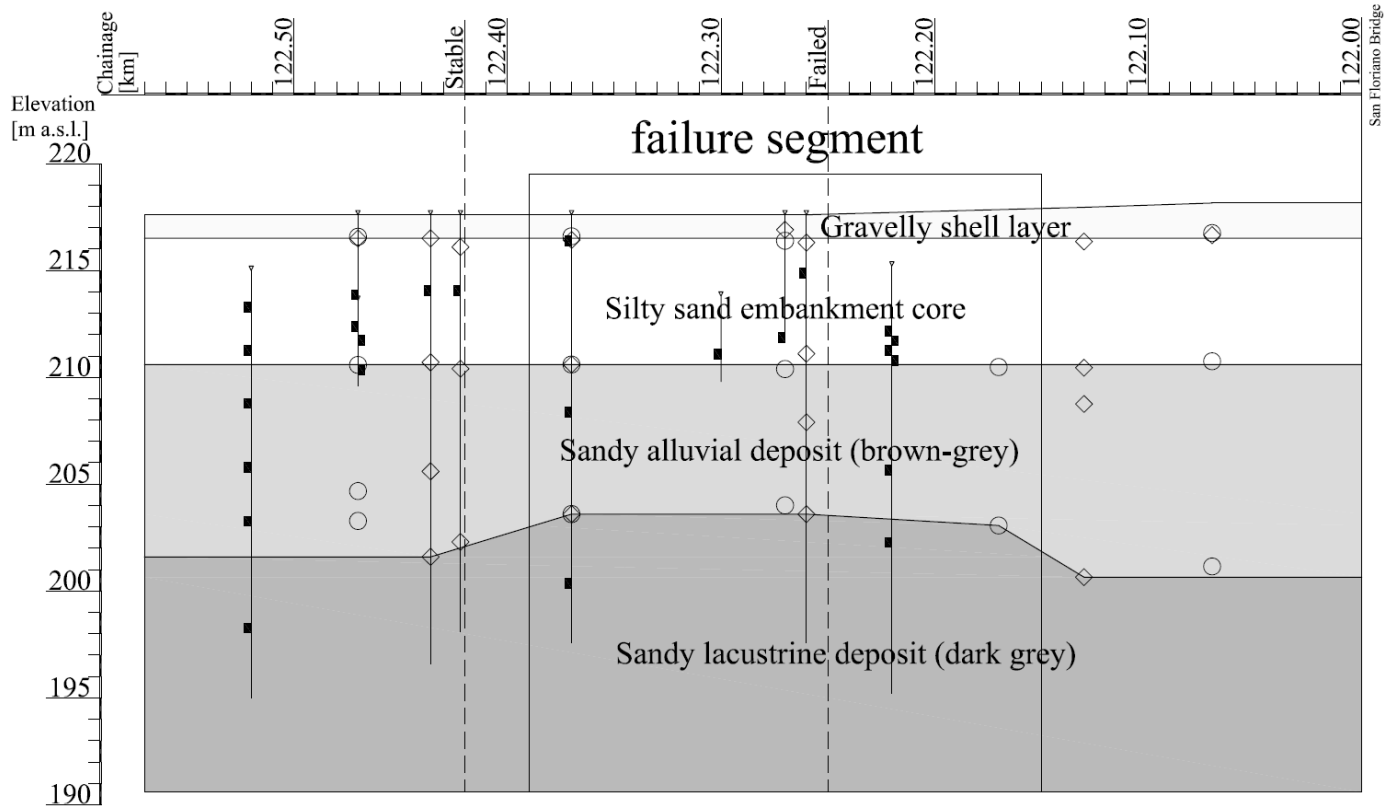


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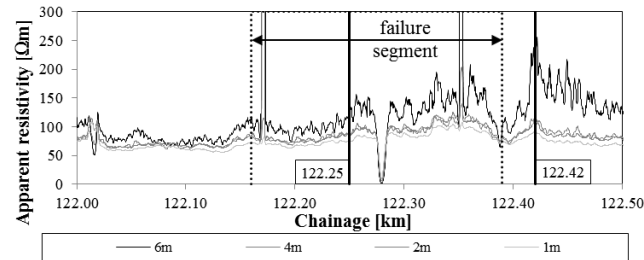


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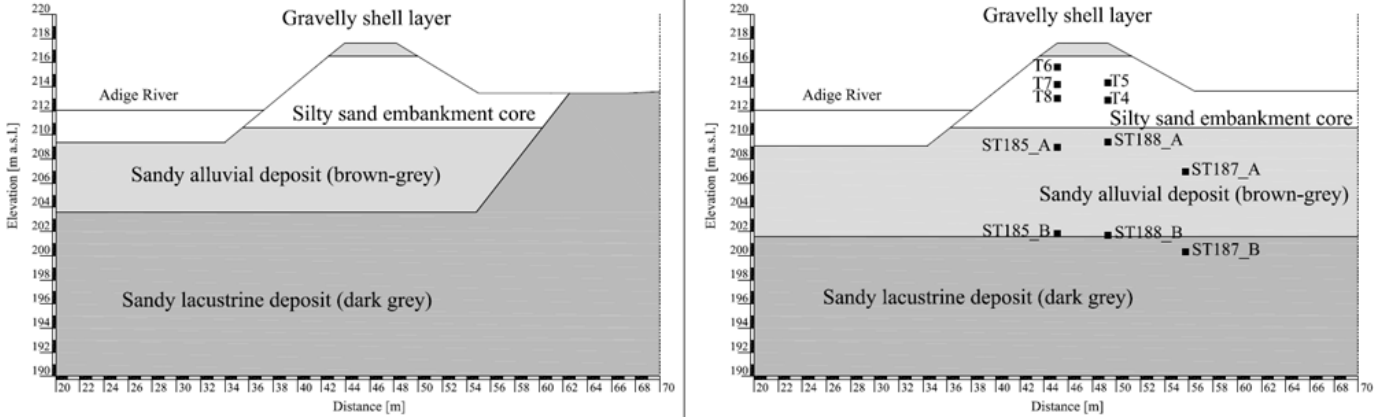


Fig. 4. Soil profile for the section within (left) and outside (right) the failure segment.

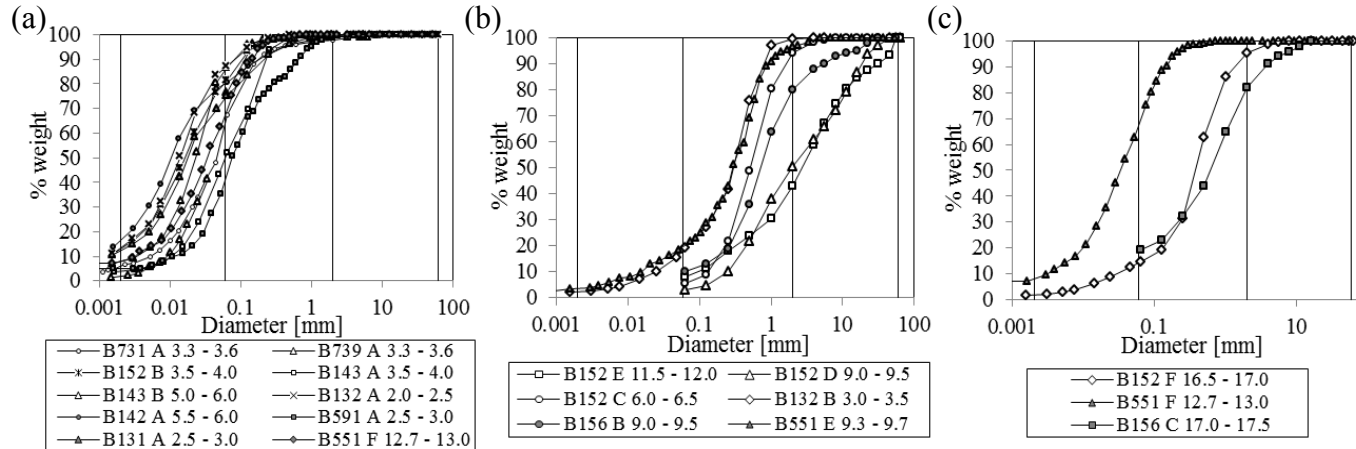


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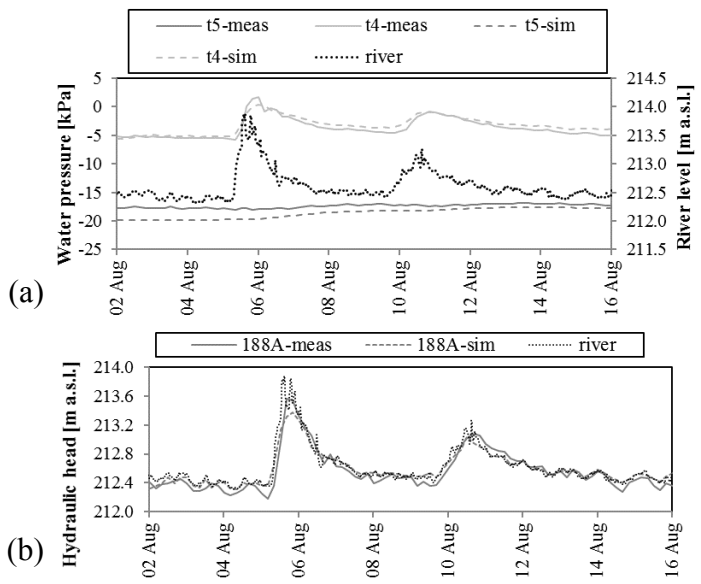


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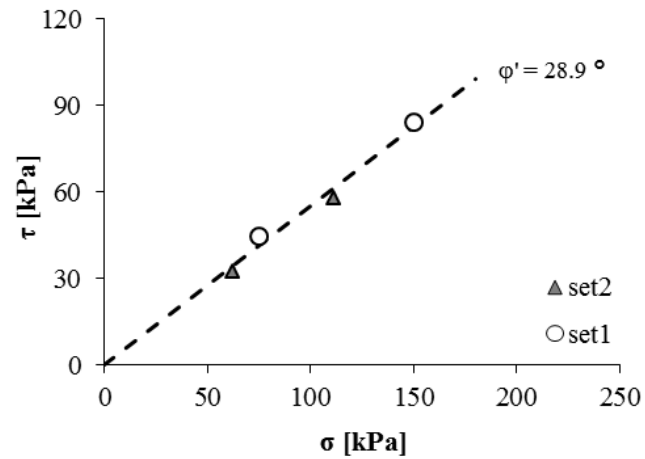


Fig. 7. Failure envelope obtained from direct shear tests on two sets of specimens from the embankment core.

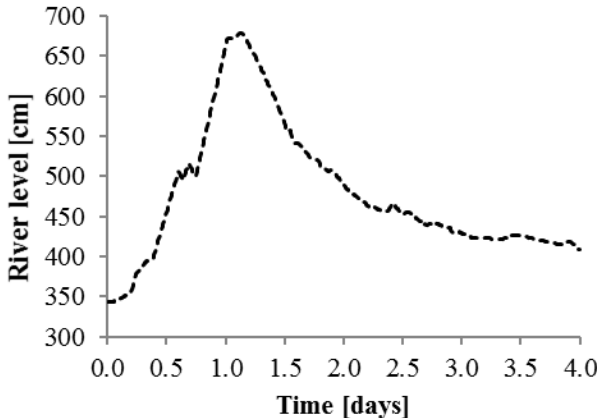


Fig. 8. Hydrograph recorded during the flood event in 1981.

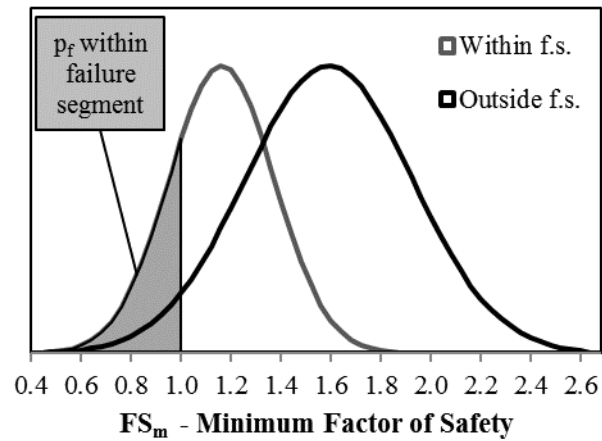


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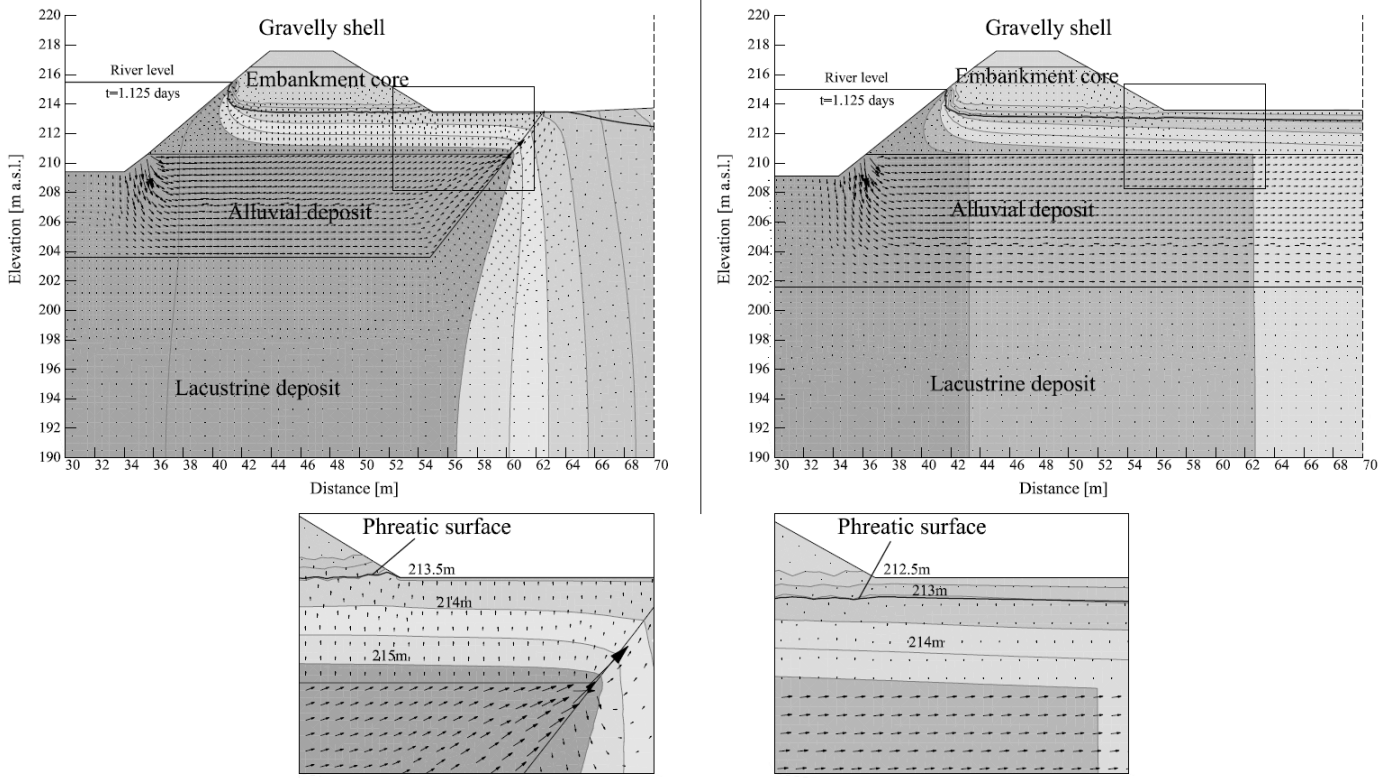


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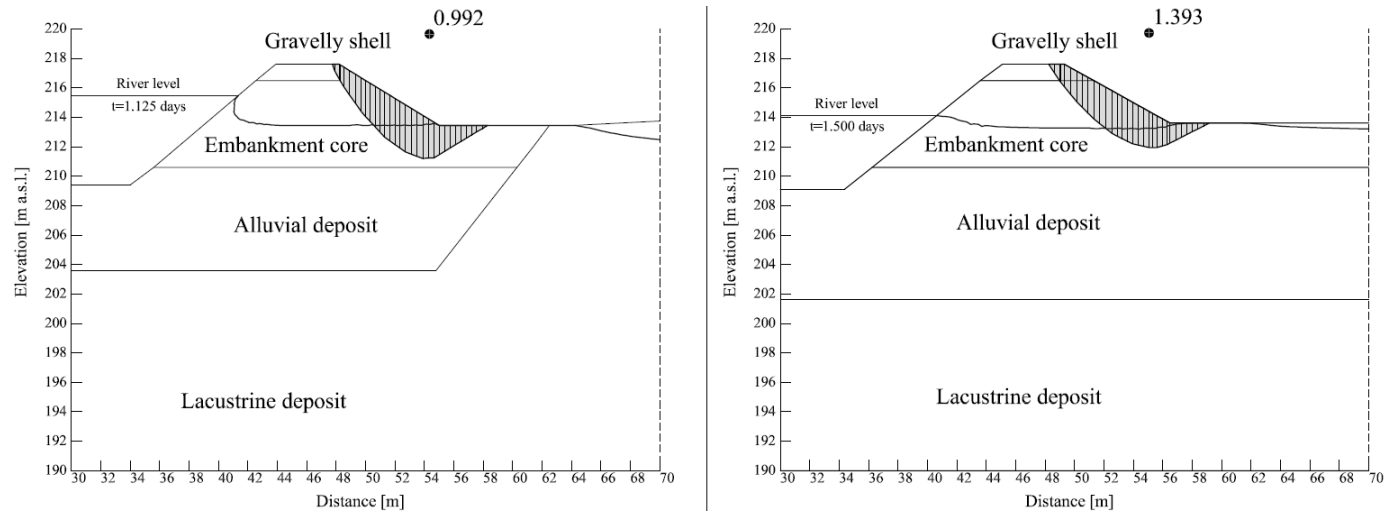


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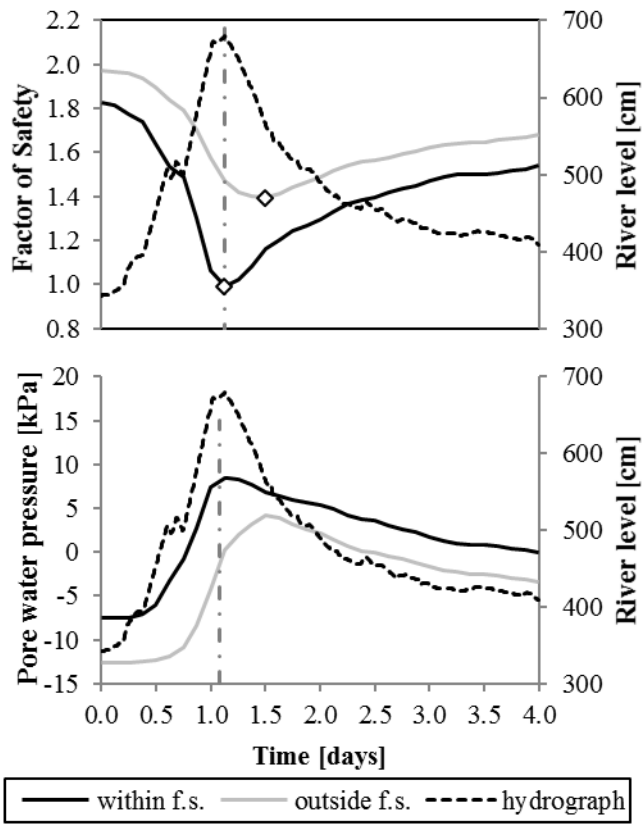


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