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Abstract

A 2D saturated-unsaturated unsteady-flow numerical study has been carried out to analyze the behavior of levees stressed by flood events. The investigation has involved: i) simulation of the seepage process in a simplified levee over a long period of river flows; ii) the use of a synthetic design hydrograph to be utilized as an alternative to a long-term history of river stages and iii) the influence of the unsaturated parameters on the maximum saturation depth in the levee soil. The results of the analysis show that the statistical properties of the maximum annual piezometric levels are different from those of the corresponding river levels, and that the tested synthetic design-hydrograph is able to guarantee a well-balanced, conservative margin. The analysis shows that the role of the unsaturated zone is also very important. Furthermore, a comparison of the piezometric levels, computed by means of the numerical model, with those computed through simplified solutions, shows that the latter ones may not be conservative.

Keywords: Earthen levees; Flows through porous media; Levee piezometric levels; Modeling unsaturated zone; Synthetic Design Hydrograph

1 Introduction

River levees are important devices to control floods and protect the territory. The design of levees requires both geotechnical and hydraulic requirements to be taken into consideration For instance, the phreatic line should not cut the downstream side of an embankment, in order to avoid the triggering of erosive phenomena, which may reduce the water containment efficiency and compromise the stability of the embankment.

Many analysis have been carried out to understand the complex processes that levees undergo during flood events. For instance, fragility curves have been developed to consider the multiplicity of aspects that stress levees and cause their failure (hydraulic, geo-hydraulic and global static failures). Fragility curves are drawn up on the basis of physically-based and empirical process formalization (Vorogushyn et al., 2009) or experimental analyses (Hewett et al., 1987). Fragility curves constitute an important tool that can be used to support vulnerability and risk analyses (Camici et al., 2017; Mazzoleni et al., 2019), as, when combined with stochastic models of hydraulic loads, they allow the probability of levee failure to be computed. In this regard, the definition of the hydraulic loads for levee analysis is not a trivial matter, and it is a research topic of great interest: for instance, a copula-based model, which considers both the peak flow discharge and flow duration, has been proposed for the estimation of the structural residual hazard (Balistrocchi et al., 2019) and the use of a Synthetic Design-Hydrograph (SDH) has been suggested for levee design purposes (Butera and Tanda, 2006).

When dealing with river levees, one of the most important aspects is the identification of the phreatic line. To this aim, geometric and empirical criteria were developed in the past to identify the location of the phreatic line (e.g. Shaffernak, 1917; Casagrande, 1940; Kozeny, 1931; USACE, 1993).

Apart from resorting to geometric and empirical criteria, accurate and site specific descriptions of the phreatic line location can also be obtained by means of numerical models. The currently used numerical models, in fact, allow seepage phenomena through a levee to be simulated by taking into account the geometry of the embankment, the soil properties and appropriate boundary and initial conditions. The reliability of the numerical results depend on an accurate definition of the hydraulic head boundary condition and the adoption of adequate soil parameters. It is usual practice to consider steady-state conditions in these models, assuming a water level that is constant over time at the river side of the embankment and equal to the river stage of the discharge value of the design return period. However, a flood event produces an unsteady flow, and a phreatic line that changes over time. The design of an embankment under steady conditions can lead to an oversizing of the embankment (and therefore to a non-economic design, USACE, 2013; Butera and Tanda 2006) and, even more worrying, cannot account for possible instabilities due to changes in the water level in the river (e.g. Rinaldi et al., 2004; Kwang Seok Yoon, 2005; Stark et al., 2014; Jafari et al., 2019). Such instabilities in some cases can be acceptable, if controlled, as stated in Lupiano et al. (2020) where dams have been designed with backfilling, through the implementation of a steady-state numerical model, to ensure that the failure occurs at an appropriate water level.

A transient analysis is of fundamental importance to assess slope stability, and a fully coupled unsteady flow-mechanics analysis (e.g. Pinyal et al., 2008; Voltz et al., 2017), in which attention is paid to the composition of the soil and to the soil parameter values (e.g. Elkholy et al., 2015), is desirable. The drawdown effect on the riverside can in fact be quite risky (e.g. Mitchell and Hunt, 1985), and an analysis under steady state conditions is not able to handle such a case.

The use of numerical models under unsteady conditions allows not only the modifications in time of the phreatic line to be understood and taken into account, but also the role of the hydraulic content in the unsaturated zone of the levee. The role of the unsaturated zone and its effect on the piezometric levels reached during flood events is a topic which, to the best of the Authors' knowledge, has received very little attention.

Traditional approaches that deal with the issue of the piezometric levels reached in a levee and the problem of levee dimensions under unsteady conditions did not consider the impact of the unsaturated zone, that is, they considered that the soil above the piezometric surface was completely dry. Supino (1955) and Marchi (1957) suggested relatively simple solutions to compute, under a few hypotheses, the location of the phreatic line in unsteady conditions. It should be mentioned that such semi-analytical solutions are valid for the linearization of the flow equation and assume Dupuit's hypothesis. Giugni and Fontana (1999) then extended the work of Marchi to a nonlinear flow equation and removed Dupuit's assumption.

The present work pertains to the analyses of the seepage process in a levee under unsteady conditions, with particular attention being paid to the maximum annual piezometric levels reached in the levee. A saturated-unsaturated numerical model has been used and the analysis concerns the following three aspects: 1) the statistical characterization of the piezometric levels reached in the levee; 2) the use of synthetic hydrographs for the analysis of the seepage in the levee; 3) the sensitivity of the saturated-unsaturated dynamics in the levee to the unsaturated soil parameters, i.e. the impact of soil retention and the relative hydraulic conductivity curves.

The analysis has been carried out at a real site: the Pontelagoscuro Po River section (Ferrara, Italy). Public Agencies, devoted to hydrological surveying and to the planning and management of the Po River, have recorded the river water levels in Pontelagoscuro for many years. The daily water levels and hourly observations during flood events are in fact available for this hydrograph station for the years 1951 to 2016. Furthermore, synthetic hydrographs are also available for the Pontelagoscuro section: Maione et al. (2003) developed special design hydrographs (SDH – Synthetic Design-Hydrograph) for Po River sections that are useful for numerical simulations of flood routing; these SDHs can be used for the prediction of the maximum water levels while taking into account the storage due to the inundation of the floodplains. The possibility of deriving SDHs from a regional analysis (e.g. Tomirotti and Mignosa, 2017), without the necessity of historical records, suggests testing the suitability of SDHs for levees design.

The manuscript is organized as follows: a brief description of the mathematical statement of the problem is presented, and this is followed by a description of the data and the numerical model. The first part of the analysis concerns a statistical characterization of the piezometric level in the levee, which is followed by the evaluation of the impact of the use of SDHs for the hydraulic load. The analysis concludes with the treatment of the role of the unsaturated zone. The work is completed with a discussion of the results and some conclusions.

2 Mathematical statement of the problem

Darcy’s law and continuity equations govern seepage phenomena through an embankment: inserting Darcy's law into the continuity equation, for a homogeneous and variously saturated medium, one obtains the following equation:

$$\frac{\partial}{\partial x}\left(K(\theta_w)\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K(\theta_w)\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K(\theta_w)\frac{\partial h}{\partial z}\right)=S_0\frac{\partial h}{\partial t} \qquad (1)$$

where h is the piezometric head inside the levee, and θ_w, K and S_0 are the water content, the hydraulic conductivity and the specific storage coefficient of the soil, respectively. Eq. (1) is completed with the relations that describe the link between the piezometric height and the water content of the soil (i.e. the retention curve $h=h(\theta_w)$) and the relationship between the hydraulic conductivity and the water content of the porous matrix ($K=K(\theta_w)$).

The Van Genuchten model (Van Genuchten, 1980) for unsaturated soil has been used in the present work:

$$\theta_e = \left[1+\left(\alpha \cdot \psi\right)^n\right]^{-m} \qquad (2)$$

where the effective water content, θ_e, is related to the irreducible water content, θ_r, and to porosity n through the following equation:

$$\theta_e = \frac{\theta_w - \theta_r}{n - \theta_r} \qquad (3)$$

The symbol ψ in (2) stands for the suction in the ground (or capillary head), which is defined as the opposite of the piezometric height:

$$\psi = -\frac{p_w}{\gamma_w} \qquad h = z + \frac{p_w}{\gamma_w} = z - \psi \qquad (4)$$

In expression (4), p_w and γ_w are the pressure and the specific weight of the water, respectively. The coefficients α and n in (2) have to be determined experimentally, while

$$m = 1 - \frac{1}{n} \qquad (5)$$

The relationship between the hydraulic conductivity and the water content is defined by introducing the relative hydraulic conductivity coefficient, K_{rr}, which represents the ratio between the hydraulic conductivity of the soil of a generic water content with respect to the saturated hydraulic conductivity:

$$K_r = \frac{K\left(\theta_w\right)}{K_{sat}} \qquad (6)$$

Van Genuchten stated (1980) that:

$$K_r = \theta_e^{1/2}\left[1-\left(1-\theta_e^{1/m}\right)^m\right]^2 . \tag{7}$$

The boundary and initial conditions define the solution of the differential equation (1).

The 3D problem defined by relations (1) to (7) is complex, and some simplifications of the problem were proposed in the past that allowed analytical or semi-analytical solutions to be obtained. These solutions can capture the main features of the phenomena, but do not consider, for instance, the role of the unsaturated zone. In this article, we refer, in some of the comparisons, to the semi-analytical model of Marchi (1957), as already used by Butera and Tanda (2006).

3 The data and their processing

The case study deals with the Pontelagoscuro section of the Po River (Italy). The catchment area of the basin is 70091 km^2. The considered data pertain to the water levels observed in the 1 January 1951 to 31 December 2016 period. The water level is recorded and published daily in yearbooks, although, upon request, hourly step data can be supplied.

Some morphological changes occurred in the river during the examined period; in particular, a lowering of the river bed was detected (Marchetti, 2002) which caused modifications of the geometry of the river section and, for this reason, the observed water levels cannot be considered to constitute a homogeneous time series. In order to obtain results with the usual statistical analysis tools for stationary time series, we modified the observed water level data with the procedure described hereafter.

The stage data were converted into discharge data using the rating curve considered reliable during the observation period (96 relations in the considered period) and all the obtained discharge values were then back-converted to stage values using the same rating curve, that is, the 1982 rating curve, which was chosen arbitrarily. The thus obtained water levels were interpolated to obtain a one-hour time step sequence to use in the numerical simulations. The achieved dataset may be considered as homogeneous, and is referred to, in the following, as the rearranged historical stage time series (rearranged stage history –RSH– in short).

Figure 1 shows the frequency of occurrence of the stage values in the RSH, which was obtained by processing the 66 years of rearranged data: the abscissa value for a given stage in the ordinate axis describes the number of days for which that stage value is exceeded in an average year. The line depicted in Fig. 1 is the stage-duration curve: the minimum value is 1.01 m a.s.l., the maximum is 12.11 m a.s.l., the median value is 3.30 m a.s.l and the mean value is 3.66 m a.s.l..

HERE FIGURE 1

The synthetic design hydrographs (in short SDH) were obtained for the same Pontelagoscuro section (Maione et al., 2003), by processing the data available for different return periods -T_r- (T_r = 2, 5, 10, 20, 50, 100, 200 and 500 years). The duration of the hydrographs was set equal to 953 hours, which corresponds to the 95% percentile of the durations of the hydrological events whose water levels are higher than the level that corresponds to the 80% percentile of the historical water level series. These percentile values were set so that the duration of the SDHs was representative of the flood event durations. The SDHs were transformed, through the 1982 rating curve, into time patterns of the water levels, and Synthetic Design Level Diagrams, in short SDLDs, were thus obtained (Butera and Tanda, 2006). The obtained SDHs and SDLDs are shown in Fig. 2 for the 2016 updated observations.

The RHS and the SDLDs were used as boundary conditions for the upstream edge of the levee, i.e. the river side, both in the semi-analytical model and in the numerical one. The legend of the different curves in Fig. 2b reports the return period of the SDH that was used to create the SDLD, although, in principle, it cannot be assumed as the return period of the SDLD.

HERE FIGURE 2

4 Numerical model

The FEMWATER code (Lin et al., 1997) was used for the numerical model of the seepage. A rectangular-shaped prism model was built with the dimensions and physical parameters defined according to the main characteristics of the Pontelagoscuro levee, although a greatly simplified geometry was assumed (Fig. 3). The dimensions of the model in the horizontal plane are: 500m in the *x* direction, orthogonal to the river, and 1m in the *y* direction parallel to the river. The extension of the model in the *x* direction was considered long enough to reduce the impact of the downstream boundary condition (Fig. 3). Only one column of elements, whose size was fixed at 1m, was considered in the *y* direction; since the surfaces of the vertical planes at

y=0m and y=1m were set as impervious, the thus built 3D model behaves like a 2D model in the vertical plane.

HERE FIGURE 3

The vertical dimension of the model is 66.38m. The model elements change size along the x and z locations: they are smaller where higher variations of the piezometric head can be expected, that is, upstream close to the river, and in the upper zone of the model where the phreatic line moves in response to the transient water levels in the river. The side of the elements varies between 1m and 7m along the x direction and between 1m and 5m along the z direction.

The water levels in the embankment were analyzed at 10 sections at different distances from the upstream face (riverside); their locations are summarized in Table 1.

HERE TABLE 1

As far as the boundary conditions are concerned, the bottom of the model is a horizontal and impervious plane located at -50 m a.s.l, the RHS, or alternatively the SDLDs, represent the boundary condition at the riverside, while a constant total head with a value equal to that of the initial condition was given to the downstream boundary. As mentioned above, impervious boundary conditions were adopted on the vertical planes that delimit the model in the y direction. Moreover, the upper horizontal plane of the model was assumed impervious, i.e. no recharge or evaporation was considered possible through the soil surface during the simulations. A static condition, whose value influences the distribution of the humidity in the unsaturated zone, was assumed for the initial conditions over the entire domain.

The initial condition was set equal to the first value of the water level series (i.e. 3.78 m a.s.l., January 1st 1951) in the RHS simulations, so that the initial depth of the aquifer was set equal to 53.78 m. Preliminary runs, showed that the memory of the initial condition in the analysis of the RHS (66 years long) is limited: differences in the initial condition equal to 2.7 m after 2.5 months of simulation resulted in maximum changes of 0.18 m.

According to the technical reports on the Pontelagoscuro levees (e.g. SISMAPO project, 2015), the soil in the levee was considered as a sandy silt with a total porosity and hydraulic conductivity equal to 0.406 and $5 \cdot 10^{-6}$ m/s, respectively.

Van Genuchten relations were used to describe the physical properties of the unsaturated soil and, due to the absence of specific investigations, the relative parameters were defined according to the procedure introduced by Sleep (2011). The residual water content was assumed equal to 10% of the total porosity and the parameters of equations (2) and (3) were estimated considering different humidity conditions of the soil. Since the value of these parameters changes as a function of the wetting or drying conditions, five different conditions, all-referring to sandy silt soil, were considered, and the estimated parameters are shown in Table 2. The "Average wetting condition (AW)" and the "Average drying condition (AD)" refer to the values averaged over different experiments on sandy silt samples under wetting and drying conditions, respectively. The "Wetting Boundary 90 % confidence condition (WB90)" values are the parameter values of the lower extreme of the 90% confidence interval for wetting condition samples, while those of the "Drying Boundary 90% confidence (DB90) condition" are the parameter values of the upper extreme of the 90% confidence interval for drying condition samples. The parameter values of the Average Wetting-Drying (AW-D) condition are the average values of the Average Wetting condition (AW) and the Average Drying (AD) condition. Figure 4 shows the characteristic curves of the unsaturated soil for the considered conditions; reference can be made to Sleep (2011) for more details.

HERE FIGURE 4

HERE TABLE 2

A Matlab post processor code was written to identify the location of the phreatic line at each monitoring section of the levee (i.e. where the water pressure is equal to the atmospheric pressure). Given the curvature of the streamlines, the pressure distribution cannot be considered hydrostatic in the x-z vertical plane and the location of the piezometric surface therefore cannot be computed as being equivalent to the piezometric head at the computation point. The elevation of the piezometric surface in the levee was computed at each section by means of a bi-linear interpolation of the pressure field, which in turn was determined by means of the Femwater code for the area where the soil conditions change from saturated to unsaturated.

5 Characterization of the levee levels stressed by the RHS

A statistical analysis of the maximum annual water levels reached in the sections considered in Table 1 for the simulation of the 66-year river stage has been carried out. The initial condition was hypothesized as a horizontal piezometric surface at 3.78 m a.s.l., that is, corresponding to the first datum value of the historical water levels, which is equivalent to the water level that is reached for 128 days throughout the average year. The unsaturated soil was described using the average wetting-drying condition (Table 2); the impact of the parameter values on the unsaturated zone is discussed in the following section.

As a first step of the analysis, the return periods of the annual maximum phreatic levels, in the sections listed in Table 1, were compared with the return periods of the annual maximum levels in the river for each year of the RHS simulation.

The maximum annual approach is able to compute the return period of the annual maximum phreatic levels in the levee and the river water levels obtained from the RHS simulation. The maximum value for each year was found for each levee section and for the river; the thus obtained series (66 data for each section) were then processed to identify the statistical distribution that best fitted the data. Six distributions were tested (normal, log-normal, gamma, GEV, the extreme value and the exponential one). It emerged that, according to the Bayesian information criterion, the distribution that best fitted the data in all the sections was the normal one.

Using the parameters of the best-fit statistical distribution, the return period of each annual maximum value was then computed and compared with the return period of the annual maximum water level in the river for the same year. Although it was possible that the values did not refer to the same flood event, any diversity that can be observed in Fig. 5 highlights that the stress degree of a flood event for a levee may have been different from that of the river.

HERE FIGURE 5

Figure 5 shows the results of the analysis: as can be seen, markers located at the 45°-degree line mean that, in a certain year, the river and the levee section underwent events of the same severity. Markers located under the 45°-degree line show that the flood events had been more severe for the river than for the levee; the opposite holds for markers located above the 45°-degree line. In the latter case, the levee is stressed even when the levels in the river are not very high. This is due to the nonlinearity of the process that relates the river levels and the seepage in the levee. In fact, not only does the maximum value of the hydrographs influence the piezometric levels in the levee, but also their shapes (i.e. the duration of the water height in the river that can be linked to the floodwater volume).

These results show the importance of testing the use of SDLDs for the design of a levee under unsteady conditions: SDLDs are, in fact, built considering not only the maximum discharge values, but also the flood volumes.

6 Characterization of the phreatic levels in the levee stressed by SDLDs

In order to test the suitability of the SDLDs to represents the excitations applied to the levee and then to obtain design information, the SDLD obtained from an SDH with a return period of 200 years was applied as a boundary condition at the riverside. An SDH with a return period equal to 200 years was used because this is the main reference value prescribed by Italian Public Agencies devoted to the planning and management of the Po River (e.g. Autorità di bacino del fiume Po, 2010). Such a diagram is here referred to as $SDLD_{\ell 200}$ (SDLD labeled for 200 years).

The results of the computations were compared with the phreatic line level obtained for each levee section by means of the previously mentioned statistical inference with a return period of 200 years (Fig. 6).

As can be seen in Fig. 2b, the used $SDLD_{\ell 200}$ has a high initial value of 7.31 m a.s.l. When a level of 7.31 m a.s.l. is assumed as the initial condition for the piezometric surface in the levee, high levels were reached in the levee during a flood. In fact, much of the levee is under saturated conditions before the beginning of a flood (for $z< 7.31$ m a.s.l.) and the storage capacity of the levee is reduced.

In order to evaluate the impact of the initial level of the horizontal phreatic surface (which also influences the initial water content in the unsaturated zone), an analysis was performed considering different initial conditions, and the results are shown in Fig. 6. Five values, which were obtained by dividing the difference between the first datum of the $SDLD_{\ell 200}$ (7.31 m a.s.l.) and the first datum of the RHS (3.78 m a.s.l.) into five parts, were chosen as the initial condition. In terms of percentiles of the river water level set, the 3.78m hydraulic level corresponds to the 68% percentile, while 7.31 m a.s.l corresponds to the 97.35 % percentile.

It should be pointed out that a different initial level in the aquifer from the starting value of the river hydrograph causes an abrupt change in the river side, which may induce numerical instabilities; reduced time steps were therefore adopted to avoid simulation problems.

Figure 6 shows the hydraulic levels reached in the levee sections when $SDLD_{\ell 200}$, which was derived from the SDH with a return period equal to 200 years, is used and different initial piezometric levels are considered. As expected, the differences in the curves are remarkable, and this underlines that the initial aquifer conditions, which in general are not so well defined, play an important role in the evolution of the phreatic line.

HERE FIGURE 6

The phreatic levels obtained after the inference of the probability distribution of the phreatic levels are compared, in the same figure, with the hydraulic level for a return period equal to 200 years, as computed from the statistical analysis of the maximum annual values. It can be seen that the use of SDLDs, obtained from the SDHs of the return period equal to 200 years, is conservative for all the sections when the initial condition of the level is greater than 4.66 m a.s.l, that is, for the 82% percentile of the RHS stages.

This result seems to be justified by the fact that the phreatic line in the embankment changes quite slowly after a flood and, as a result, it is necessary to adopt moderate initial high water level conditions in the levee domain to simulate severe excitations for the 200 year return period.

7 The impact of the unsaturated zone parameters

The possibility of modeling the unsaturated zone is one of the main reasons for using numerical models instead of semi-analytical solutions. The Femwater code does not reproduce the characteristic hysteresis of the retention curve, and only one curve in Fig. 3 can be used at a time.

In order to test the impact of the parameters that characterize the unsaturated zone, the numerical model was run with the different sets of parameters listed in Table 2. In this analysis, the $SDLD_{\ell 200}$ obtained from the SDH for a return period equal to 200 years was used as the riverside condition and the initial level of the phreatic surface was set equal to the first level of the SDLD series, i.e. 7.31 m a.s.l. The following dimensionless coefficient, which was named infiltration ratio (*IR*), was introduced to analyze the behavior of the water levels in the levee:

$$IR(x,t)=\frac{h(x,t)-h_{il}}{h_{\max}(x=0)-h_{il}} \tag{8}$$

where $h(x,t)$ in (8) is the water level at time t and distance x from the levee riverside, h_{il} is the initial level at distance x and $h_{\max}(x=0)$ is the maximum level reached in the river. The $IR(x,t)$ parameter varies from 0 to 1: a value of IR close to zero means that the flood event in the river does not affect the piezometric level in the levee sections. Higher values of IR indicate a prompt response of the levee aquifer to changes in the water level in the river.

Panels a) to d) in Fig. 7 show the infiltration ratio values as a function of time at different distances from the riverside. The results obtained from the numerical simulations using the parameters listed in Table 2 are shown together with the levels computed with the semi-analytical solution introduced by Marchi (1957) in each panel. It should be pointed out that, when adopting the Marchi solution, the ratio between the rise in the river levels and the initial thickness of the levee aquifer should be less than 0.25 in order to guarantee the reliability of the linearization process.

HERE FIGURE 7

The piezometric surface levels decrease in all the sections as the α parameter in eq. (2) increases. Increasing the α value, for a given suction value (see Fig. 4), means that the unsaturated soil has a low level of humidity and, as a result, the soil has a greater storage capacity, and the piezometric levels of the levee therefore increase less than in the case of drying conditions (a smaller α value). If the distance from the river is increased, the results obtained through the semi-analytical approach (SA in the legend in Fig. 7) are below those obtained by means of numerical modeling. This result can be explained by considering that the semi-analytical approach does not take into account the presence of humidity above the piezometric surface, and thus relies on a greater water storage capacity in the soil pores. The semi-analytical model solution for the levee sections close to the river is not always below the ones provided by the numeric model: the semi-analytical solution, obtained under Dupuit's hypothesis, is less accurate close to the river because of the non-negligible vertical components of the flow field.

Table 3 and Fig. 8 show the maximum levels reached in each monitoring section obtained using the semi-analytical solution and the numerical model with different α values. Remarkable differences can be noticed when different values of the α coefficients are used, and the semi-analytical solution underestimates the piezometric surface levels in most of the tested conditions.

HERE TABLE 3

Figure 8 and Table 3 also point out the role of the water content in the unsaturated zone when the flood wave passes in the river. If the levee is in drying conditions, because a previous flood event has recently occurred, the levee aquifer levels will be higher than those that would be reached if the levee were under wetting conditions. It is in fact known, from field experience, that a levee can collapse in the case of multiple peak floods, when a flood peak occurs, even if it is lower than the previous one, because the levee has a higher initial water content.

8 Conclusions

In this work, a two-dimensional numerical model has been adopted to analyze the water levels in a levee. The analysis mainly concerned three aspects: i) the statistical characterization of the water levels in the levee compared to that of the river, ii) the use of synthetic design level diagrams (SDLDs) derived from synthetic design hydrographs (SDH) and iii) the role of the unsaturated zone in the piezometric levels of the levee.

In order to deal with the first issue, a historical water level series, rearranged to obtain an acceptable homogeneity level, was considered as the riverside condition. The statistical analysis of the annual maximum levels, reached at different distances from the riverside, showed that the maximum return period of the annual maximum of the piezometric levels in the levee is different from that of the river levels. This result confirms that the stresses in the levee may in part be due to factors other than the maximum water level in the river.

The use of SDHs, transformed into SDLDs (Synthetic Design Level Diagrams), has proved to be useful to identify the piezometric surface. The obtained results have shown that the use of the first datum of the SDLDs as the initial condition is appropriate, even though it may appear too precautionary. It has also been shown that $SDLD_{\ell 200}$, labeled for a 200-year return period, can be used to estimate, with a certain approximation, the piezometric levels of the same return period obtained after statistical inference of the values resulting from the simulation of the historical time series of the river levels. Since SDHs can be derived from a regional analysis (Maione et al., 2003; Tomirotti and Mignosa, 2017), without the necessity of historical records, it is the Authors' opinion that SDLD represents an alternative levee design tool. It produces results that are well-balanced between the traditional static design, with the maximum river stage under steady conditions, and those of an analysis under unsteady conditions with a historical time series of the river stages.

It has emerged, from the sensitivity analysis of Van Genuchten's α parameter, that this parameter has a great impact on the maximum piezometric levels. A smaller α value implies higher piezometric levels.

Finally, it has been found that simplified semi-analytical models are not reliable close to the riverside (because Dupuit's formula does not apply) or at a distance from the riverside (because the role of the unsaturated zone is neglected); moreover, their results are often not conservative.

An analysis under transient conditions will be carried out through an integrated hydraulic-geotechnical approach as a future development of the present research, in order to establish the best precautionary design conditions for the stability of the levee which do not lead to an oversized design.

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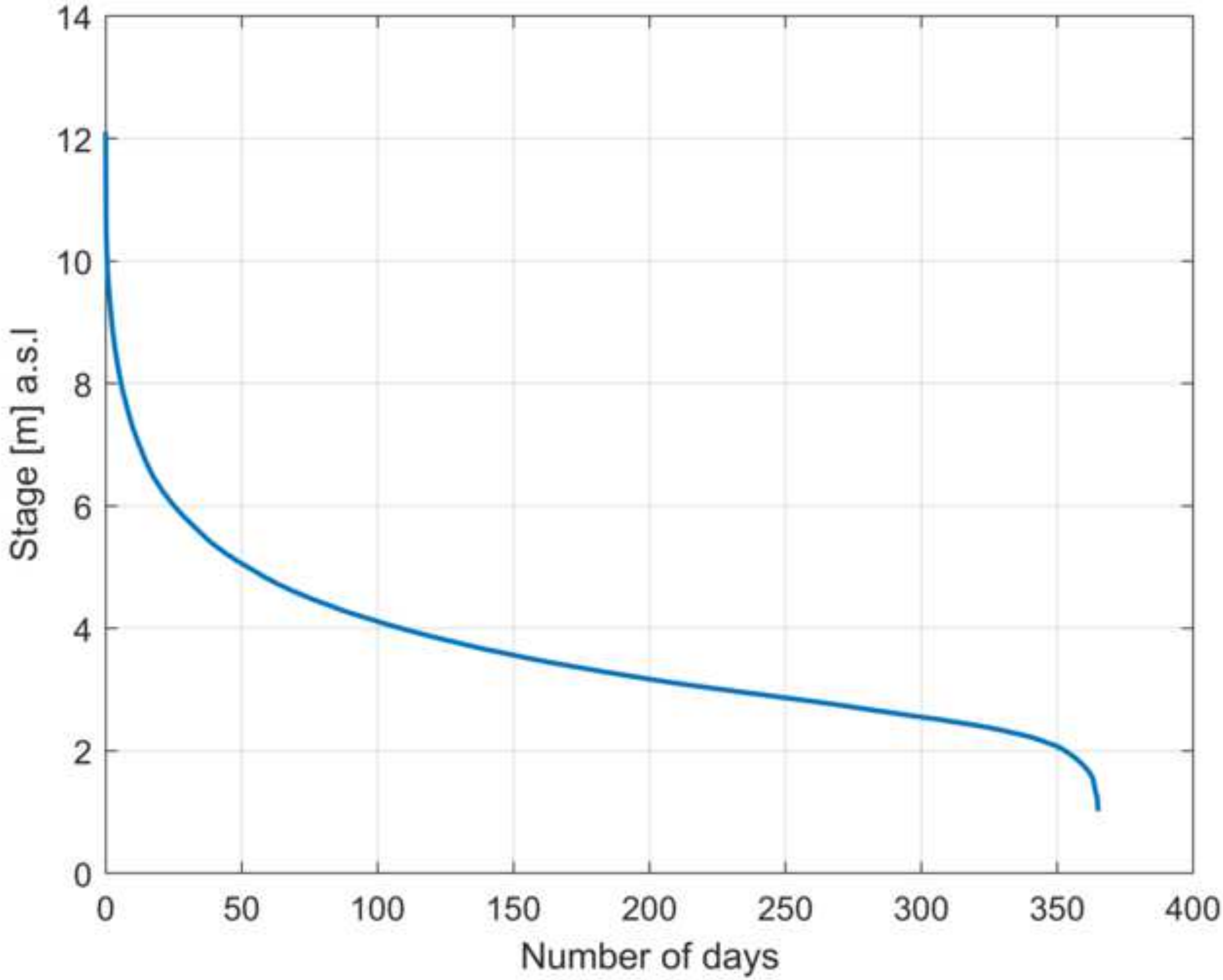

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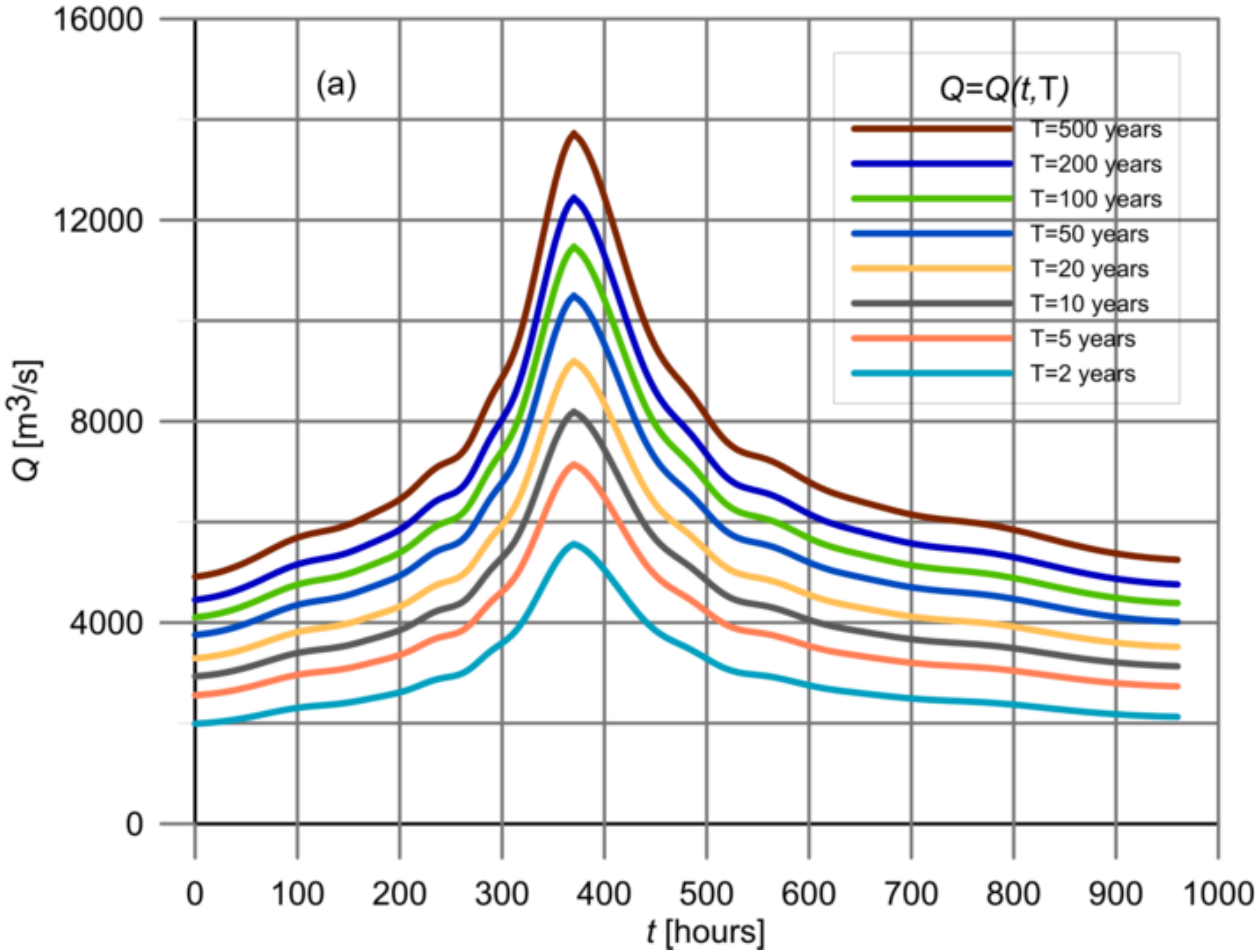

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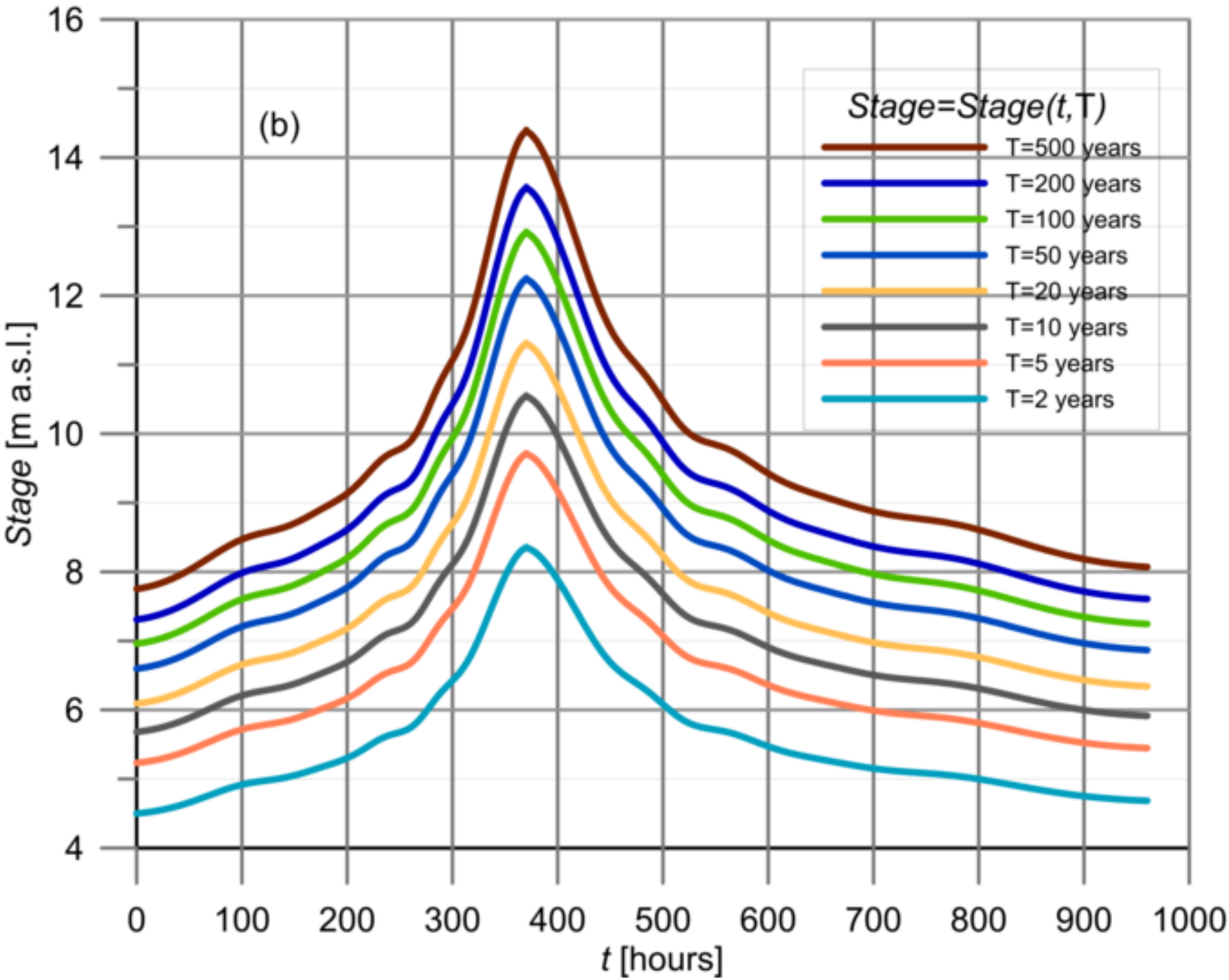

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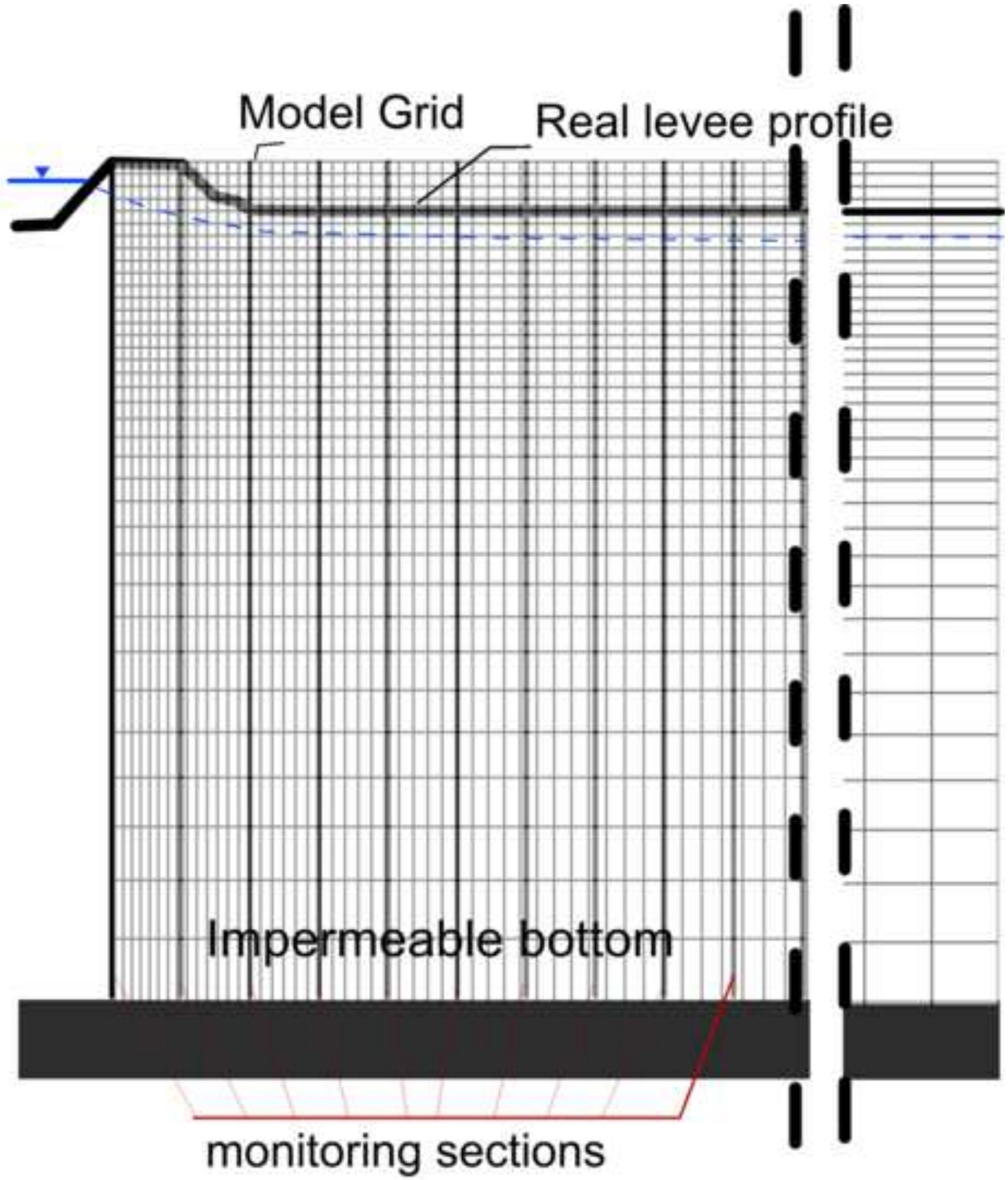

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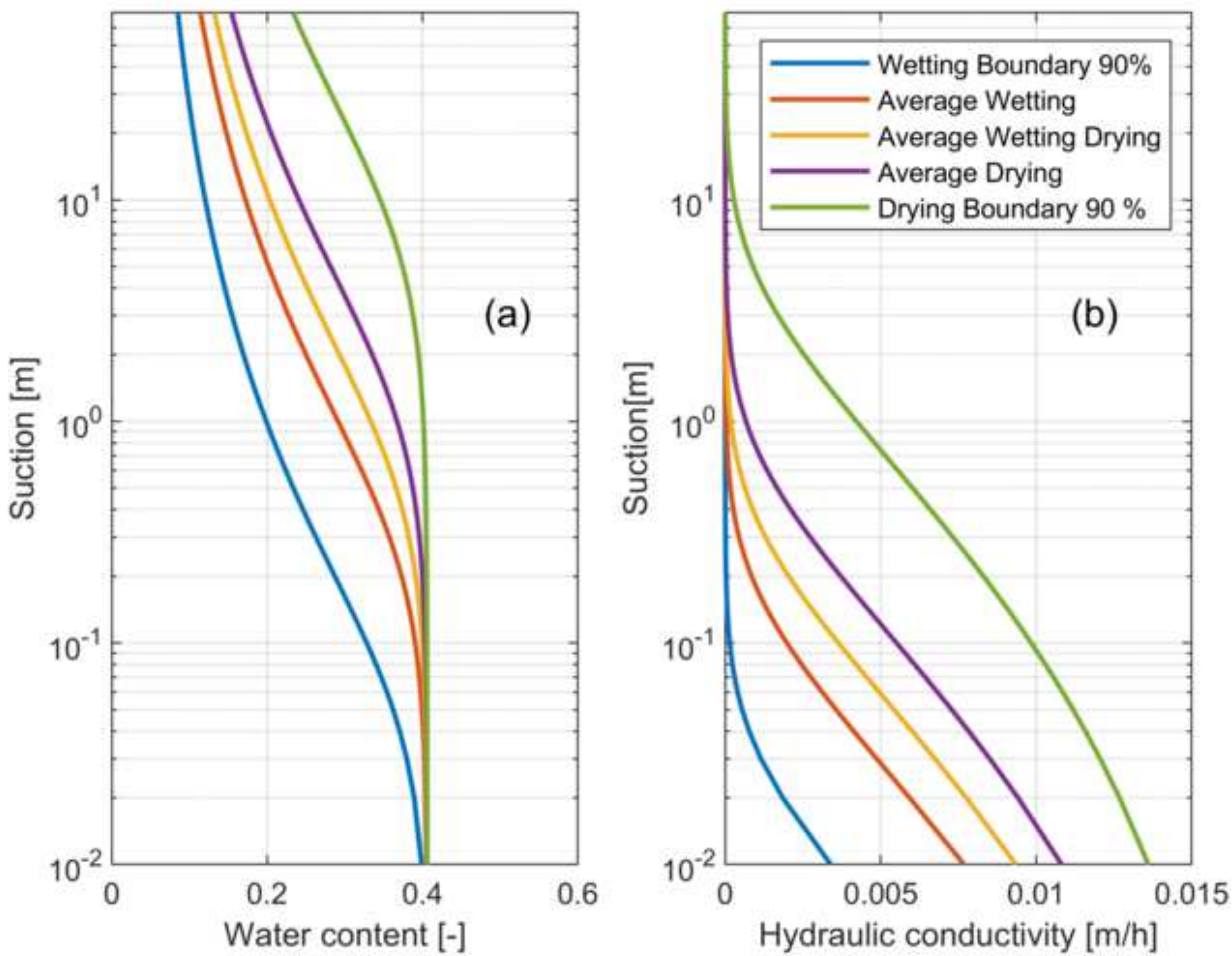

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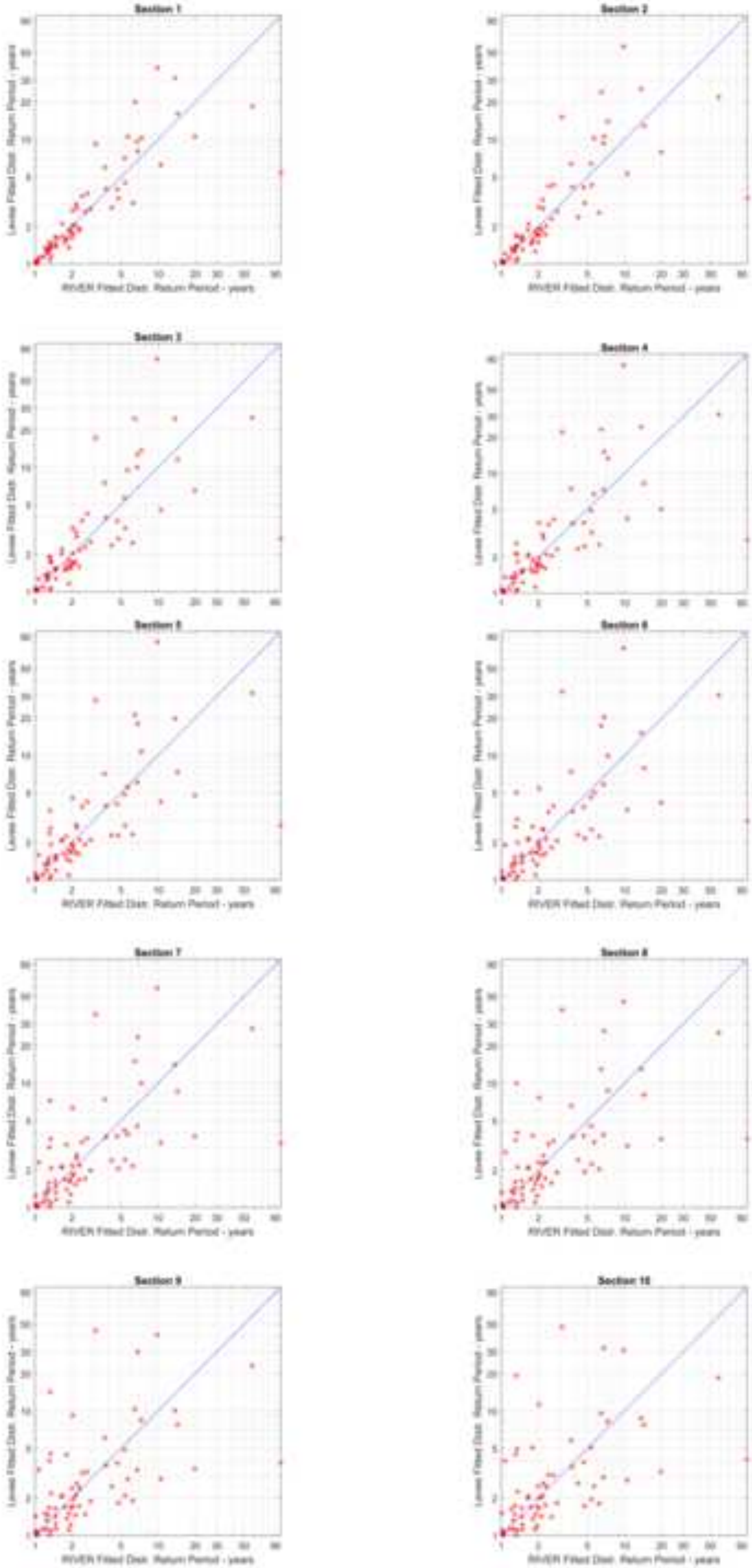

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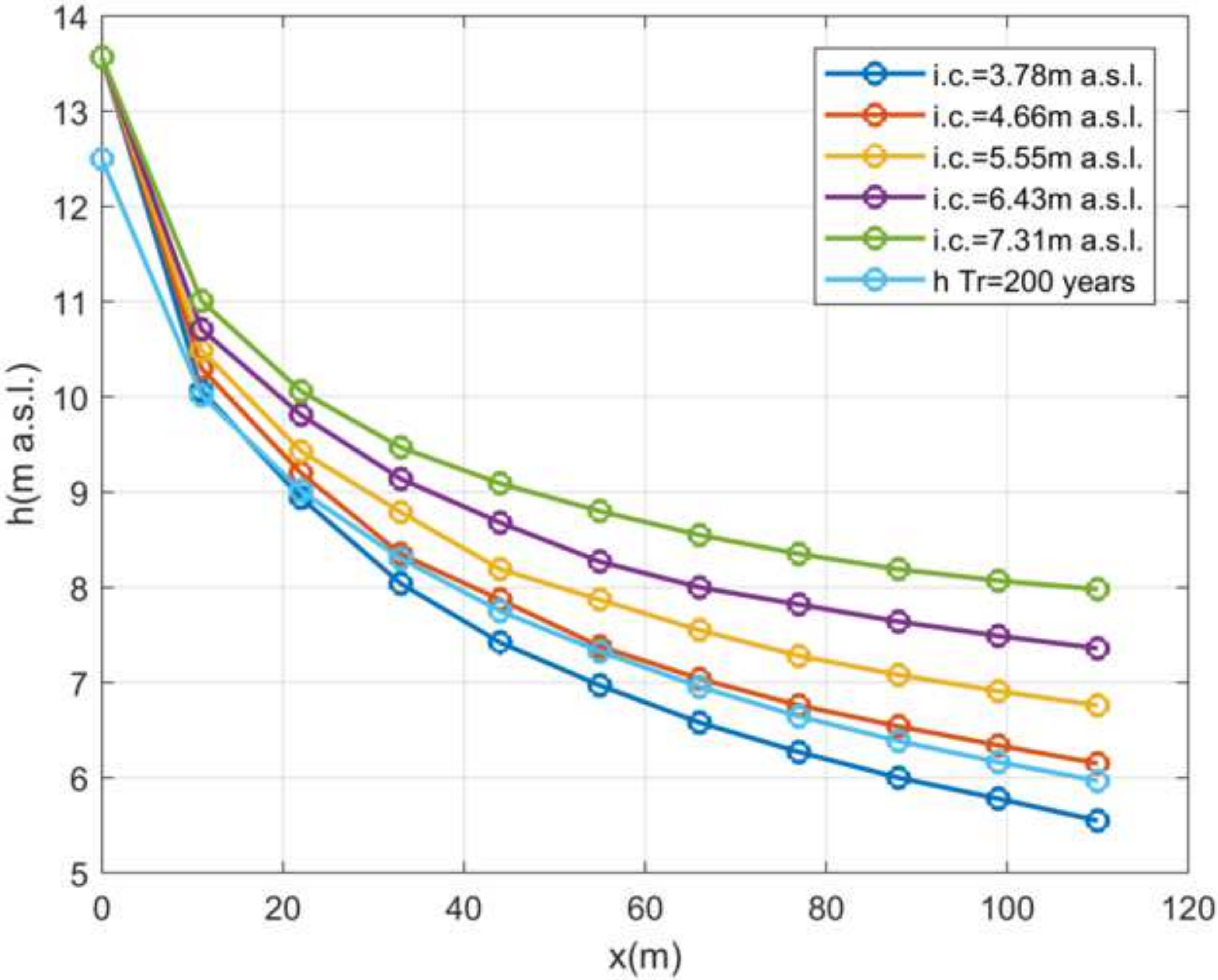

i.c.=3.78m a.s.l.
i.c.=4.66m a.s.l.
i.c.=5.55m a.s.l.
i.c.=6.43m a.s.l.
i.c.=7.31m a.s.l.
h Tr=200 years
h(m a.s.l.)
x(m)
14
13
12
11
10
9
8
7
6
5
0
20
40
60
80
100
120

Figure 7

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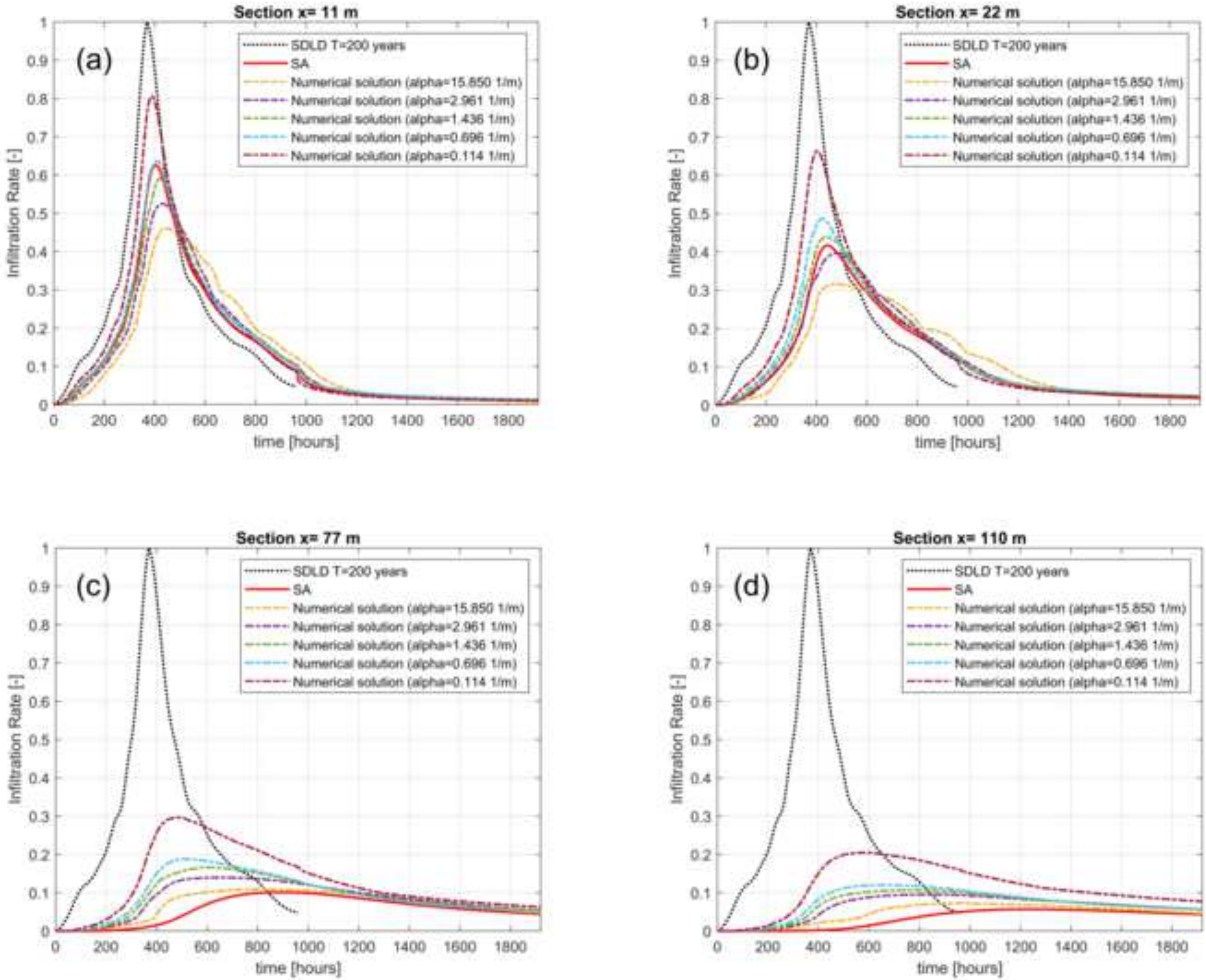

Figure Captions

Figure 1. Stage duration curve at the Pontelagoscuro section of the Po River

Figure 2. Pontelagoscuro section: the Synthetic Design-Hydrographs for different return periods (a) and the Synthetic Design Level Diagrams derived from the SDH for a given return period (b).

Figure 3. The used mesh in the [*xz*] plane

Figure 4. The soil water retention curve (a) and the hydraulic conductivity versus suction curve (b) for the considered soil.

Figure 5. Return period (years) of the maximum yearly piezometric levels as a function of the flood return period at different distances from the riverside.

Figure 6. Maximum water levels reached in the levee using SDH_{l200} (labeled for T_r=200 years), for different initial conditions of the piezometric levels, compared with the T_r=200 year piezometric levels.

Figure 7. *IR* results obtained from numerical modeling compared with those obtained from the semi-analytical solution, for different values of the α parameter and different distances from the river. (SDLD, derived from SDH for a return period equal to 200 years, as the river boundary condition).

Figure 8. Comparison of the piezometric levels obtained for different values of the α parameter and different distances from the river. Results of the numerical simulations and semi-analytical model.

Table 1 with caption

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Table 1. Distance –*x*- of the observation sections in the levee from the riverside.

Location of the observation sections in the levee			
Section number	*x* [m]	Section number	*x* [m]
0	0	6	66
1	11	7	77
2	22	8	88
3	33	9	99
4	44	10	110
5	55		

Table 2. Values of the unsaturated zone parameters.

Unsaturated zone parameter values	α [1/meter]	n [-]	m [-]
Wetting Boundary 90 % confidence condition (WB90)	15.850	1.3005	0.2311
Average Wetting condition (AW)	2.961	1.3005	0.2311
Average Wetting-Drying (AW-D) condition	1.436	1.3005	0.2311
Average Drying(AD) condition	0.696	1.3005	0.2311
Drying Boundary 90 % confidence (DB90) condition	0.114	1.3005	0.2311

Table 3. Maximum piezometric levels (m a.s.l.) reached at different distance from the river using different approaches to model the unsaturated zone.

	S.A.	α=15.850 1/m (WB90)	α=2.961 1/m (AW)	α= 1.436 1/m (AW-D)	α=0.696 1/m (AD)	α=0.114 1/m (DB90%)	
***x*(m)**	**piezometric levels - m a.s.l.**						**Range of level variations (m)**
0	13.57	13.57	13.57	13.57	13.57	13.57	----
11	11.23	10.19	10.6	11.01	11.3	12.36	2.17
22	9.92	9.29	9.8	10.06	10.36	11.46	2.17
33	9.13	8.88	9.21	9.47	9.8	10.77	1.89
44	8.64	8.43	8.92	9.09	9.3	10.23	1.80
55	8.32	8.25	8.61	8.8	8.96	9.79	1.54
66	8.1	8.11	8.35	8.55	8.7	9.44	1.34
77	7.94	7.99	8.18	8.35	8.49	9.17	1.13
88	7.83	7.91	8.07	8.19	8.31	8.94	1.11
99	7.74	7.84	7.98	8.07	8.17	8.74	1.00
110	7.67	7.76	7.91	7.98	8.06	8.59	0.92