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Landslide analysis of historical urban walls: the case study of Volterra.

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Abstract

This work arises from the evidences of recent collapses of historical urban walls. These events, mainly occurred after severe rainfalls, recalled the attention on the vulnerability of these infrastructures. Specific evaluation models which take into account the role of moisture inside the walls are not frequent in the literature. The problem is difficult to treat in many practical cases, due to the extension of the urban walls and the variability of their geometry and of their mechanical features.

The integrity of the urban walls is investigated throughout the analysis of a set of vertical sections. Risk scenarios of rainfall events are analysed. At this scope, four Limit States are proposed and investigated to determine both collapse and serviceability conditions. The combined risk analysis is obtained by treating the single-risk analysis. The results from the limit analysis are compared with those from the FEM models. The method is applied to the relevant case study of the Volterra historical urban walls with a retrospective analysis of the section which collapsed in January 2014 and the analysis of other relevant sections.

Keywords: historical urban walls; survey; vulnerability assessment; landslide vulnerability; hydraulic risk; Volterra's urban walls.

1 Introduction

Recent failures occurred in the last years recalled expertise attention on the collapse of historical city walls. In the last ten years a relevant number of defeats or collapse events of historical city walls took place in Tuscany Region (Italy) (Puppio et al., 2019), (Andreini et al., 2013).

The investigation of those collapses shows that the presence of moisture inside the walls and the surrounding soil due to several causes (rain, water losses etc.) as well as poor maintenance activities and inadequate restoration works played a relevant role. This forced to introduce the effect of rainfalls and moisture in the safety evaluation of the walls.

The current Italian Code provides indication for the natural actions induced by wind, earthquake, thermal effects but not specifically by the presence of moisture. This is not easy to implement because of the fact that humidity depends not only from extreme events as rainfalls, but also from permanent conditions such as type of soil, drainage systems and human interferences.

In Geotechnics two limit scenarios are traditionally considered: the drained and undrained conditions. The approach is related to the soil type and the rate of load application (permanent loads vs total loads). The additional effect of water is usually considered in terms of hydraulic thrust and internal pore pressure.

The decay of the soil mechanical properties due to rainfalls is often neglected in common professional cases. The actual method of modelling (Casapulla, 2008; Grillanda et al., 2019) and retrofitting of the existing masonry buildings (Mistretta et al., 2019; Sassu et al., 2017) is usually affected by complexity and a lot of uncertainties. Actually the rocking and cinematic approach (Casapulla C., Maione A., 2017; Casapulla, 2015; Casapulla et al., 2010) is able to provide adequate indication about the safety of the wall but the presence of the earth filling produces additional non linearity in the response. Nevertheless, taking into account the weakening effect of the water plays a crucial role as showed below. The method presents a general application which can also be applied to different cases and different sources of imbibition.

Urban walls are usually situated along the external perimeter of the historical centre of the old cities. They were constructed both with commercial and defence functions. They also covered the static function of retaining walls in case of slope. Those several aspects led to irregular shapes of the walls both in plan and elevation (Y.C. Chan, 1982). The need to develop a method for survey and analysis should also consider the influence of the water behind the walls. Numerical application is performed on Volterra urban walls based on a set of collapsed sections.

The paper is structured in five section. Section 2 highlights some recent collapses and illustrates a survey procedure to detect mechanical and geometrical parameters. In Section 3 a method to evaluate the effect of rainfall infiltrations with different literature contribution is proposed. In this Section the assessment of the hydraulic vulnerabilities is carried out considering the effect of weakening due to water imbibition. In Section 4 the results are discussed.

2 Collapses and survey strategy

Tuscany Region (Italy) was affected in the last ten years by several collapses of historical urban walls: seven examples on five different locations are summarized in Fig.1 and Tab.1.

In the investigated cases the presence of internal moisture and heavy rainfalls were recurring events during the failure. The repairing costs resulted considerably high, respect to preventive maintenance activities (an overall ratio of about 10/1), provided the other indirect costs, so the prevention based on vulnerability assessment would lead to a significant loss reduction for the community.

The proposed survey methodology consists of the following steps:

(1) identification of a set of relevant vertical sections of the city walls;
(2) on-site geometrical survey;
(3) on-site mechanical assessment;
(4) extension of the results to the entire perimeter of the city walls.

The description of the survey procedure is reported in (Puppio et al., 2019; Vagaggini, 2019), throughout the elaboration of on-site surveys with GIS data. The mentioned method is illustrated referring to the collapses of the Volterra's city walls. Nowadays it preserves a historical city center of Etruscan origin with a satisfactory state of conservation. The Etruscan portion of the walls is a part of these walls and the arch of access to the city center is a masterpiece of priceless beauty.

The discretization of the perimeter (Puppio et al., 2019) is shown in Fig.2. The Middle Age walls enclose the city centre and represent the core of this work. The surveyed sections on which the work is focused are shown in red and blue in Fig. 2 and Fig.3. (0 / 78 and 86 /107), representing the Middle Age portions of walls. The sections in blue in Fig. 2 are remains of the ancient Etruscan circuit of the walls.

On January 31st 2014, about 35 meters of the walls of Volterra collapsed in the area between "Porta dell'Arco" and "Piazza dei Fornelli". In this section the walls in front of "Via Lungo le Mura" are 9.5

meters high with the role of retaining structure, loaded by the upstream ground of about the entire height of the wall. The moisture probably emphasized the reduction of bearing capacity of the walls.

Evidences of the effects of humidity appeared on the external face of the wall (Fig. 4) before the failure. The rainfall that forced the collapse was preceded by water infiltrations with an evident lack of drainage in the walls. The collapse exposed the foundations of adjacent buildings, as well as Palazzo Stella (Fig.5). Non linear analyses taking into account an increasing level of hydraulic thrust were made in (Puppio and Giresini, 2019). In this work, the weakening effect induced by water is instead neglected; the effect of water is solely assumed as an additional thrust. The aging effect on blocks and mortar is also neglected, since the collapsed portions of walls were of good masonry quality, as visible in Fig.6.

The main geotechnical and masonry parameters were investigated after the collapse. Two vertical continuous drillings with Standard Penetration Tests (SPT) and three horizontal continuous drillings were performed. The geological description is reported in (Santerecchi, 2014), whilst the geotechnical parameters are recalled in Tab.2. To simplify the soil characterization, two main lithotypes were observed: the Villamagna layer and the lime layer (Fig.7).

The Villamagna formation is made by a medium thick stratum of sand, interposed by a very weak calcarenite layer; the lime layer just behind the wall is characterized by relevant inhomogeneity and poor mechanical characteristics.

The GIS database furnished similar indications for the soil foundations, so the information of Section 92-93 has been extended to the other vertical sections of the same typology. The collapsed section (92-93) also furnished indications about the masonry features of the wall (Puppio and Giresini, 2019). It is possible to identify two external layers made of sandstone irregular blocks, a porous rock typical of the site, while the internal core was made by a mix of soil and small sand rocks from the nearby. Internal and external parameters were not interconnected by diatons except for the top and the basement. Mechanical parameters of the Volterra walls were obtained by the literature of masonry with similar features (D.M. 17/01/2018, 2018; Deere, 1988).

Along the perimeter of about 2,6 km, four emblematic sections have been chosen (Fig.8) for geometric survey, to identify the most distinctive ones. These cross sections are emblematic due to the following reasons:

1) geometric and mechanical data related to them are reliable;

2) geometric and mechanical properties of the selected cross sections are similar to many others investigated along the walls perimeter;
3) the four cross-sections have all a soil backfill, so they are subjected to the highest horizontal thrusts;
4) in these four cross-sections the highest percentage of vegetation and drainage with uncertain effectiveness were found.

A partial survey was performed in other 25 sections to complete the geometrical assessment of the perimeter. Concerning the adjacent areas around the walls, several buildings with at most four floors were detected: the presence of buildings was then simulated by an extra load of 10 kN/m^2 for each floor.

The following conditions were observed during the survey:

(1) Upstream earth filling and inclined downstream face;
(2) Upstream earth filling and vertical downstream face;
(3) With no or partial filling and with a variable inclination of the downstream face;
(4) Upstream earth filling and vertical downstream face.

3 Analytical assessments of rainfall risk.

3.1 The model for soil weakened by rain.

Four specific Limit States (LS) have been defined: (1) Collapse (SLC), (2) Lifeguard (SLV), (3) Damage (SLD) and (4) Integrity (SLI). These are similar to the ones provided for the buildings in the European code in case of seismic action (Eurocode, 2004). In case of the retaining walls, those limit states are given by the following values of the relative displacement d_r. defined by:

$$d_r = d_t - d_b \tag{1}$$

where d_t is the displacement of the top and d_b that of the basement (Fig.9).

The identification of the LS is conventionally carried out throughout the comparison with corresponding displacements. Defining h_w as the height of the wall, it is possible to calculate the collapse displacement d_{SLCg} by the condition of equilibrium, whilst lifeguard displacement d_{SLV} is equal to the 90% of the collapse limit state one. Hence the retaining wall is essentially an isostatic structure d_{SLCg} depends on the first limit failure mechanism (equilibrium of the isolate structure, equilibrium of the complex soil-wall, sliding, soil limit capacity). This Ultimate LS can be calculated both with LEM or FEM methods. In particular the overturning is calculated with the limit equilibrium method. The identification of d_{SLC} and consequently of d_{SLV} is carried out, also depending on the actual boundary conditions of the section, thanks to limit and non-linear analysis.

The displacement d_{SLD} corresponds to the damage of adjacent buildings, related to the maximum displacements of the surrounding land. Finally, the displacement d_{SLI} can be defined, in aesthetic sense, by the activation of the first crack on the wall or on the adjacent road pavement or on the plaster of adjacent buildings. d_{SLI} have to be defined case by case according to the kind of masonry and the geometry of the wall and of the surrounding area. This is not simple to estimate especially because of the complex geometrical conditions and the variability of the kind of structure and infrastructure near the urban walls. For this reason, an "Expert Judgement" is proposed as a strategy to identify this LS. The values of the displacements for the four LS are summarized in Tab.3.

One of the triggering causes of collapse in historical walls is the presence of water. Ongoing variations in rainfall quantity and intensity often causes landslides. The presence of water, from rainfalls or from other sources, has a double effect: (1) reduction in the mechanical properties of soils, (2) increase of the hydraulic thrust.

The SLIP model given by Montrasio and Yoshida (Montrasio and Valentino, 2016) is used to predict the risk of landslides in an indefinite slope. This model, applied to recent case studies as in (Montrasio and Valentino, 2007; Schilirò et al., 2016; Valentino et al., 2014), showed a good predictability level. In the case of retaining walls, in order to estimate the effect of the decay of the mechanical parameters of soil resistance from imbibition, the results of Montrasio and Yoshida (Yoshida et al., 1991) can be implemented. This model is able to evaluate the reduction in shear strength, cohesion and friction angle of each soil layer due to the increase of water penetration.

The SLIP Model given by Montrasio is applied to the case of the indefinite slope and doesn't consider the reduction of the friction angle but only of the cohesion. The weakening effect of imbibition is given by rain penetration and only superficial layers can be affected. The results of Yoshida allow to also take into account the reduction of the friction angle. In addition, the discretization of the soil in horizontal strata consents to consider the effects of imbibition in depth making the model also suitable for the evaluation of the effects of the leaking of pipelines or other in depth sources of moisture. This differs with respect to the application made by Montrasio that evaluates the effects of indefinite slope on soil surface.

It is usual to evaluate the landside safety by examining a series of failure surfaces. The effect of water penetration can be considered by the equivalent saturation level: it depends on soil type, land use and vegetal covering and on the intensity of rainfalls.

An improvement of the model creating a set of horizontal layers that can be damaged by rain penetration is proposed. This modifies the overall strength and the consequent values of the defined LS.

Taken a portion of the soil of height H (Fig.10), the portion mH (m<1) is completely saturated and the remaining part H(1-m) is partially saturated.

The percentage of saturation m can be expressed as follows:

$$m = \frac{\beta^{*} h}{n(1-S_r)H} \tag{2}$$

where:

h height of rainfall;

H height of the soil interested by the rainfall;

β^* capacity of imbibition or percentage of filtered rain;

n soil porosity;

S_r saturation grade.

H depends on the presence of un-permeable soil layer. β^* can be estimated as 70% (Franceschini, 2012), (Losi, 2012).

The saturation grade is experimentally given by Montrasio et al. (Montrasio and Valentino, 2007) as:

$$S_r(h) = S_{r0} + \frac{\beta h}{nH} \tag{3}$$

where S_{r0} is the initial saturation grade, β the capacity of imbibition and h is the height of rain. S_{r0} depends on the initial moisture of the soil and consequently on the precipitation occurred in the days before the event for the considered case of study. For the Volterra event it was assumed, given the soil characteristics from in-situ tests, an initial saturation grade equal to 0,30.

The use of this model can also be applied to forecast what the intensity of the model that generate limit scenarios is. It is an interesting way to evaluate the vulnerability of historical walls.

From equation (2) one can determine the depth of saturated soil mH which corresponds to a certain height of rain h. It is well known that h is associated to rain duration and return period:

$$h = a\, d^{\kappa}\, t^{\eta} \tag{4}$$

where t is the rain duration (expressed in hours), t_r is the return period of the event (expressed in years) (Fig.11). Dimensionless parameters a, κ, η are regional coefficients estimated by a multiple linear regression of regional rainfall record: for Volterra's site they can be found in (AA. VV., 2006). For each rain duration and return period a different imbibition scenario is then provided.

In the meanwhile, the shear resistance of the saturated layer is expressed by the well-known Mohr-Coulomb law:

$$\tau = c' + \sigma \tan\phi' \qquad (5)$$

in which c' is the cohesion, ϕ' the friction angle and σ the compression stress.

The shear strength of the non-saturated soil is expressed in the model through a modified Mohr-Coulomb law:

$$\tau = c' + \sigma' \tan\phi' + c_\psi \qquad (6)$$

where c_ψ is the initial apparent cohesion, as by Fredlund and Rahardjo in (Fredlund and Rahardjo, 1993) adding the positive effect of partial saturated soil. It can be expressed by:

$$c'_\psi = AS_r(1-S_r)^\lambda \qquad (7)$$

where A and λ are dimensionless coefficients depending on the soil type. Both are identified by experimental tests by Montrasio and Valentino (Montrasio and Valentino, 2008), (Franceschini, 2012), (Montrasio et al., 2014), (Montrasio et al., 2009). For the sake of simplicity, the following value of c_ψ can be applied to the whole depth H:

$$c_\psi = c'_\psi (1-m)^\alpha \qquad (8)$$

where α is the homogenization coefficient, here assumed equal to 3,40.

The variation of the apparent cohesion c_ψ with saturation is shown in Fig.12: in (b) shows that c_w quickly decreases with S_r for medium sands and tends to zero when S_r is higher than 80%. Also the friction angle varies with saturation as reported in (Farooq et al., 2015). The decreasing of the friction angle is proposed by Yoshida et al. (Yoshida et al., 1991). The variation of the specific weight of the soil depends on the soil porosity. All these variations, homogenised for the entire layers of height H, are listed in Tab.4.

3.2. Analysis of the landslide soil with rain penetration.

Two types of static analyses are carried out both for the hydraulic and the seismic vulnerability assessment:

- Limit Analysis Method (LAM);

- Finite Element Method (FEM).

The safety of a retaining wall is usually performed throughout the limit analysis method (LAM). The analysis is here carried out using the SSAP2010 code (Borselli, 2018). LAM is a simplified method not suitable to find all the limit states, for this purpose a FEM nonlinear static analysis is implemented in the Straus 7 R.2.4. code (VV, n.d.). This takes into account the initial stress due to soil consolidation and the internal pressure due to water and soil imbibition.

To the contrary, this approach is not applicable to the imbibition event. As a matter of fact, soil imbibition can be related to external causes (such as leaking in water pipelines) and can maintain its consequences over time, depending on rain duration, air moisture and soil type.

The following steps have been followed to perform the numerical analysis:

1) discretization of the soil surrounding the wall in horizontal layers of 100 cm;
2) definition of five different mechanical parameters for each layer during time, to simulate the progressive effect of soil imbibition;
3) determination of the collapse load for the different limit states.

These steps are elaborated with an increasing height of rain which corresponds to different imbibition scenarios.

The decay of the mechanical properties of the soil due to imbibition in horizontal layers is shown in Tab.4. In Fig.13 the level of imbibition of each stratum is considered.

The SSAP2010 code for Limit Analysis implements the methods of Spencer (1967), Morgestern & Price (1965), Chen-Morgestern (1983), Janbu (1967), Sarma I e II (1973). They can be applied on different shapes and geometries of soil and structure. The output provides the surface failure characterized by the minimum safety factor.

The FEM model by Straus 7 (without and with progressive imbibition) was determined involving a soil area of about 4 h_w in plan and of about 2 h_w in elevation, with respect to the height of the wall (h_w). A series of 100 cm soil layers permits to introduce the progressive presence of water penetration (Fig.14). The model was created by 2-Dim plate elements with a max mesh size of 75 cm or 40 cm for the ground and masonry respectively. Specifically, the masonry was an isotropic plane element, while the ground was a soil element. A soil in situ stress is assigned to the soil as an initial condition to take into account the presence of the adjacent buildings.

The plate elements are Quad-8 type, with 8 nodes (4 at the ends of the elements and other 4 at the middle of the sides). Fig.14-16 represent the mesh of section 92-93. The different colours describe the property plates; the upstream load simulates the buildings while the red lines inside each plate represent the soil in situ stress applied to the soil.

The model is constrained laterally to fix horizontal movements along x, and at the base of the slopes to fix the displacements along x and along y. The mechanical parameters are in Tab. 5 according to the Mohr-Coulomb criterion.

As the height of rain increases with the duration, so does the saturation of the soil. This is reproduced through six versions of the mesh which have an increasing number of soil layers (100 cm thick) involved into the progressive decrease of the mechanical properties. These are summarized in Tab.6. Although the initial properties of the soil are good, saturation has a remarkable effect in the reduction both of the cohesion then of the friction angle. In fact, Tab. 7 shows a reduction of cohesion of -98% and of the friction angle up to -39% varying the Saturation grade from 30% to 79% form for the material considered.

3.3. Results

In this section the results of the analysis carried out for the different sections are shown. Starting from the collapsed section 92-93 the analyses are extended to the most relevant ones.

The corresponding results of FEM analysis, in terms of displacements and stresses for the increasing load steps (code Straus 7 R.2.4.), are listed in Tab.7 and Fig.16.

The pushover analysis and principal stresses domain in case of rain duration of one hour and a return period of 175 years is shown in Fig.17. This section, that is the one interested by the collapse of January 2014, reveals the reduced safety factor also in dry condition. In particular the application of the initial soil stresses in the FEM model is not possible for rain durations major than 1 h because of the non-convergence of the models. So, a sensitivity analysis is carried out considering the effective rain duration in the month before the collapse with the LAM model (Fig. 15).

Fig. 18 shows that the Safety Factor (SF) gets lower than one in an interval near the 29th of January reaching a minimum value of 0.73 on the 12th of February if the collapse hadn't happened before. It is possible to see that also a simple instrument such is the LAM method, can show a great sensibility in predicting the vulnerability. In addition, it is interesting to highlight that the minimum SF is reached a few days (12th of February) after the maximum height of rainfall is registered (31st January), as an effect of imbibition in depth and retention capacity of the soil. This work is extended to the most relevant sections presented in Fig.8 characterized by different dimensions and boundary conditions.

In the following:

- Section 32-33 (Fig.19);
- Section 48-49 (Fig.20);
- Section 71-72 (Fig.21).

With the increase of rain duration, the most relevant failure mechanism for all the analysed sections becomes the maximum displacement (d_{SLD}). Through the analysis of Fig.17-19 it was possible to determine the rain duration that corresponds to this:

- Section 32-33: 1hour 50min;
- Section 48-49: 2hours 50min;
- Section 71-72: 2hours 20min;

Finally, considering a rainfall of 12 hours, the results for all the limit states are summarized in Tab.7 with the determination of the corresponding return periods. It is then evident that the capacity of the Volterra walls in section 92-93 was insufficient to sustain the rainfalls occurred during the extraordinary climate events of January 2014. Tab.7 summarizes the results in terms of return period and intensity measure for the four limit states analysed.

4 Discussion

All the sections analyzed in this paper are studied both with the FEM and the LAM method. FEM allows a step by step evaluation of the stress and strain in the masonry and soil and provides a good estimation of which the first mechanism of collapse to occur is.

The comparison between LAM and FEM Models shows that both are in good agreement in order to provide indications about the collapse. In the analyzed case the presence of the filling and the significant slope of the wall determine a global soil-wall failure. The presence of different levels of imbibition involves different failure surfaces with the consequences of differentiate damages to the adjacent buildings.

The site survey and the failure evidences have shown that degradation elements such as humidity, vegetation, presence of buildings behind the wall can represent significant exposure elements. Indeed, not only rainstorms can be considered as degradation phenomena but also persistent humidity and filtrations caused by leak of pipelines are dangerous: in this sense a monitoring system of moisture, consisting of a set of piezometers inside the soil, can be useful to prevent degradation.

The retrospective analysis made for the collapsed section shows that in the case of Volterra the application of the imbibition model, as modified in this work, provides interesting information about the collapse happened and the possible cause. In particular, the reduction of the safety factor highlights the great vulnerability of the collapsed section and the failure surfaces obtained by SSAP and Straus 7 are close to the real one.

As the graphs of Fig.19-21 show the results are heavily dependent on the grade of imbibition. Different saturation levels affect not only the Intensity Measure that generates a certain LS but also the kind of failure that takes place first. The maximum damage displacement d_{SLD} is the first to happen in case of an high imbibition level. Instead for a reduced grade of imbibition other mechanisms are reached first. This suggests that the monitoring of displacements can provide important information to prevent damage or failure. Intensity of the rain and rain duration are both crucial variables.

This investigation is relevant in particular because of the “brittle failure” highlighted in real collapse cases. The attaining of the first collapse mechanism (sliding, bearing capacity of the soil) usually leads to the immediate failure of the retaining structure (dashed line on the curves). The collapse also happens without warning and in an unexpected way. The sudden nature of this kind of collapse is one of the most complex elements in the safety evaluation of ancient cities.

Possible countermeasures from engineering viewpoint could be the following, in ascending order of impact to the construction:

a) increasing the drainage system to ensure a proper washout of rain and other sources of water penetration, by new perforating holes distributed along the walls;
b) introducing passive steel tendons throughout the walls, to enforce the stabilization against rocking movements;
c) introducing active steel wires tendons throughout the walls, anchored to visible steel plates;
d) enforcing the basement of the wall through a reinforcement of the foundation or a set of micropiles to avoid any slipping.

The improvement of the drainage systems, both with deep and on surface works (as rain runoff and wastewater collectors and sub horizontal drains) is always recommended as shown here. In addition, the adoption of early warning systems for monitoring the moisture by piezometers and the relative displacements by inclinometers is a key strategy. The description of the mechanical parameters of the walls in terms of Bayesian approach (Croce et al, 2021) could also help the entire consolidation strategy.

This kind of analysis, in which vertical loads are incremented, shows some pushover curves that differ from the common ones. In detail, the first part of the curves exhibits displacements that are opposite with respect to those that lead to the collapse.

This is because of an initial consolidation effect induced by the increase of vertical pressures. The effect is due to the geometry, in particular to the presence of the upstream filling of earth. Indeed, for the first increments the displacements are directed towards upstream and from a certain point on the sign of the displacement changes until the failure happens.

These swelling effects, related to the specific kind of analysis carried out, do not produce significant variations on the reaching of the ultimate step and on the evaluation of the limit states.

5 Conclusion

This work is focused on the vulnerability analysis of the historical urban walls considering the weakening effect induced by moisture.

The typical landslides analysis considers the effect of water only as an additional thrust and not as a reduction of resistance in the soil or masonry. First of all, a model of imbibition is chosen in order to evaluate the effect of moisture. Thanks to the combination of the models developed by Montrasio and Yoshida, it is possible to consider the weakening effect induced by water for all the soil types. In addition, a discretization of the soil in horizontal strata, both upstream and downstream of the walls, allows to take into account more reliably the effect of imbibition in depth.

This model is applied in retrospective to the real case of collapse of the Volterra's urban walls. The properties of the materials are determined by accurate in situ surveys and the effect of this model in the landslide vulnerability is considered with five imbibition scenarios and two kind of analysis.

The Volterra case study shows a good sensibility of the results to imbibition and this can be probably assumed as a significant contributing factor to the collapse of the section 92-93 in January 2014.

The analysis, extended to other significant sections, allow to highlight the relevant effects of imbibition on the vulnerability of urban walls. In particular, for the considered sections the return periods of the rain induced by the four limit states are calculated.

The use of simplified models with the application of imbibition shows that in this case the LAM method can provide useful indication in order to prevent collapse. This method can be applied with a low information level and a reduced computational burden. For this reason, it is an interesting manner to evaluate the most critical sections.

The analysis also highlights the relevant role of the limitation of the displacements as premonitory signs of the collapse. The continuous check of the displacements to the head of the urban walls allows to have at disposal an efficient early warning system as actually performed in Volterra.

Data Availability

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

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N°	DATE	LOCATION	COLLAPSED PORTION	OBSERVED CAUSE	REPAIR COSTS
1	Sept. 2011	Pistoia	50 m	Poor masonry quality and low maintenance	1.500.000 €
2	Nov. 2012	Magliano in Toscana	30 m	Poor masonry quality and intense rainfall	980.000 €
3	Oct. 2013	Cana di Roccalbegna	13 m	Intense rainfall and low maintenance	170.000 €
4	Jan. 2014	Volterra	35 m	Intense rainfall and poor drainage system	1.500.000 €
5	Mar. 2014	Volterra	20 m	Intense rainfall and low maintenance	500.000 €
6	Dec. 2014	Magliano in Toscana	15 m	Poor masonry quality and low maintenance	300.000 €
7	Apr. 2018	San Gimignano	20 m	Intense rainfall	500.000 €

Tab. 1 – Extension, estimated cause of collapse and repairing costs (Puppio et al., 2019).

Lithotype	**ϕ [°]**	**c' [kPa]**	**γ [kN/m³]**	**γ_{sat} [kN/m³]**
Sand – Villamagna formation	37	0	20	22
Lime soil	15	20	17	18

Tab. 2 – Geotechnical parameters of the soil.

d_{SLCg}	d_{SLV}	d_{SLD}	d_{SLI}
Incipient collapse	0.9 x d_{SLCg}	h_w / 100	Expert Judgement

Tab. 3 - Limit states.

			T_r = 175 years				Units
t	**0**	**1**	**3**	**6**	**12**	**24**	**[hours]**
h	0	82	109	121	158	190	[mm]
m	0	0,31	0,41	0,49	0,59	0,7	-
S_r	0,3	0,51	0,58	0,64	0,71	0,79	-
c_w	10,4	3	1,8	1,1	0,5	0,2	[kPa]
ϕ	37	27,5	26	24,5	24	22,5	°
Υ	20	20,65	20,86	21	21,21	21,44	[kN/m^3]

Tab. 4 – Saturation coefficients Vs duration t of rain (hours).

Materials		γ (kg/m3)	*E* (kPa)	*Φ* (°)	*c* (kPa)
1.	Filling masonry	1900	1200000	44	410
2.	Masonry	2100	1200000	44	410
3.	Sand (Villamagna Formation)	2000	23076	37	10,4
4.	Lime soil	1700	16240	15	20

Tab. 5 - Material properties according to the Mohr-Coulomb criterion.

Layer	***Sr* [%]**	**γ [kg/m^3]**	***c* [kN/m^2]**	**φ [°]**
(t = 0)	30	20	10,4	37
(t = 1 hour)	51	20,65	3	27,5
(t = 3 hours)	58	20,86	1,8	26
(t = 6 hours)	64	21	1,1	24,5
(t = 12 hours)	71	21,21	0,5	24
(t = 24 hours)	79	21,44	0,2	22,5

Tab. 6 - Mechanical parameters of the soil with Tr=175 Years.

Section	**Limit State**	***Tr* [years]**	***h_C* [m]**	***Pns* [%]**
92-93	SLI	10	0,085	0%
	SLD	75	0,159	37%
	SLV	175	0,175	65%
	SLC	658	0,205	89%
32-33	SLI	10	0,085	0%
	SLD	466	0,196	85%
	SLV	658	0,212	89%
	SLC	740	0,217	90%
48-49	SLI	10	0,085	0%
	SLD	76	0,159	37%
	SLV	404	0,190	83%
	SLC	430	0,193	84%
71-72	SLI	10	0,085	0%
	SLD	401	0,190	83%
	SLV	593	0,207	88%
	SLC	658	0,212	89%

Tab. 7 – Results for the analyzed section from FEM Model. The analysis is referred to a rainfall duration of 12 hours.

Figure 1 Click here to access/download;Figure;Fig.1.pdf

Fig. 1 – Failure cases analysed in Tuscany (adapted from Puppio et al., 2019).

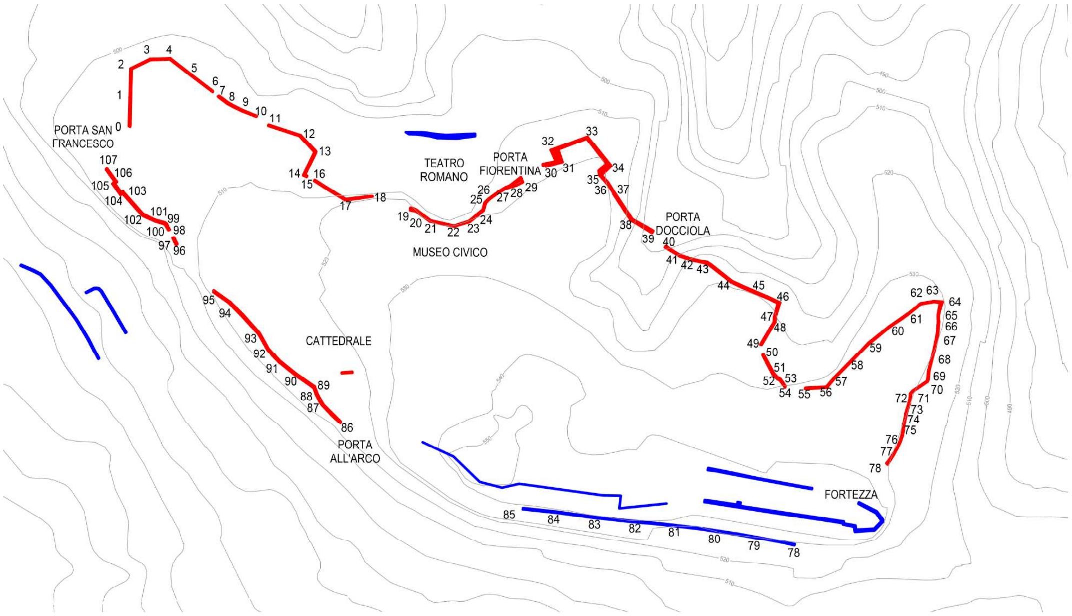


Fig. 2 - Perimeter of the urban walls of Volterra from GIS survey (adapted from Puppio et al., 2019). In red the Medieval part and in bleu the Etruscan part.

Figure 3 Click here to access/download;Figure;Fig.3.pdf

Fig. 3 - Perimeter of the surveyed urban walls of Volterra overlayered on an aerial view (adapted from Puppio et al., 2019).

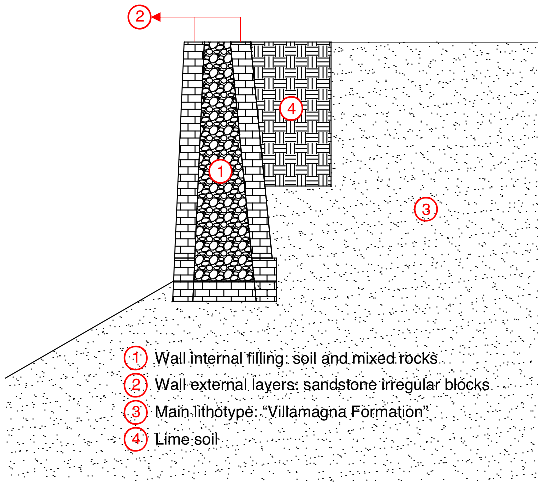
Fig. 4 - Presence of moisture before the collapse of January 2014 of the Urban walls of Volterra.

(a) *(b)*

Fig. 5 – Views of the collapsed area: (a) natural gas pipeline under the road, (b) foundation of Palazzo Stella.

Figure 6 [Click here to access/download;Figure;Fig.6.pdf]

(a)



(b)

Fig. 6 – (a) Picture of the wall after the collapse - (b) main soil lithotypes and wall layers.

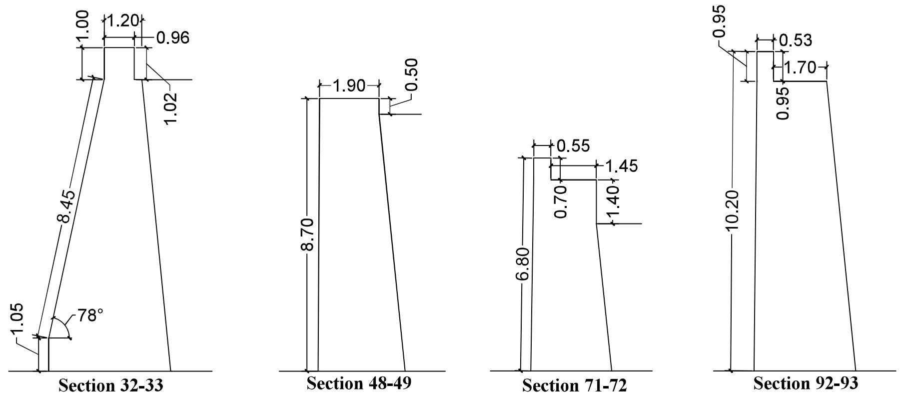
(a)

(b)

Fig. 7 - Drilling with SPT test: (a) location - (b) coring samples (adapted from Santerecchi, 2014).

Figure 8

Click here to access/download;Figure;Fig.8.pdf



(1) (2) (3) (4)

Fig. 8 - Main vertical sections analysed. Units in meters.

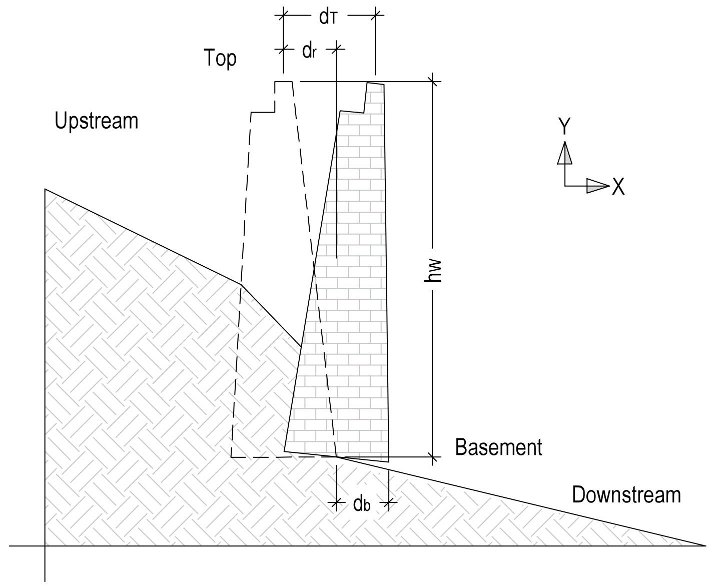


Fig. 9 – Relative displacements.

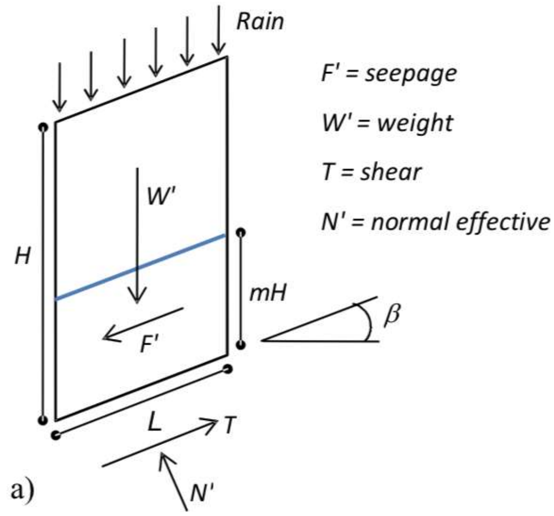


Fig. 10 – View of imbibition level for the Slip model.

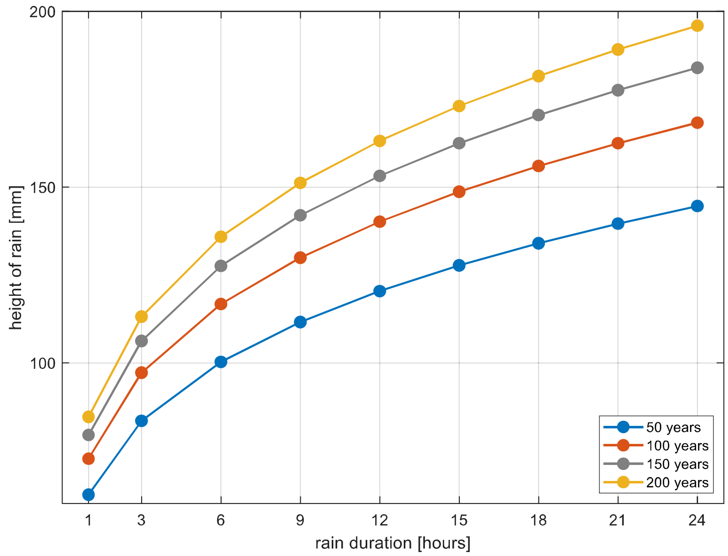
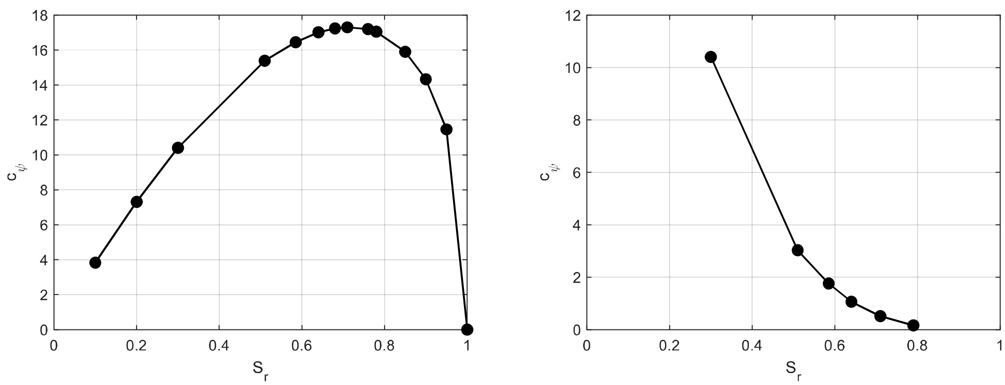


Fig. 11 – The return period of the Volterra site in terms of rainfall intensity (adapted from AA. VV., 2006).



(a) *(b)*

Fig. 12 – Apparent cohesion c_ψ Vs saturation grade S_r, for the state mH (a) and for the entire deep H (b).

Figure 13 Click here to access/download;Figure;Fig.13.pdf

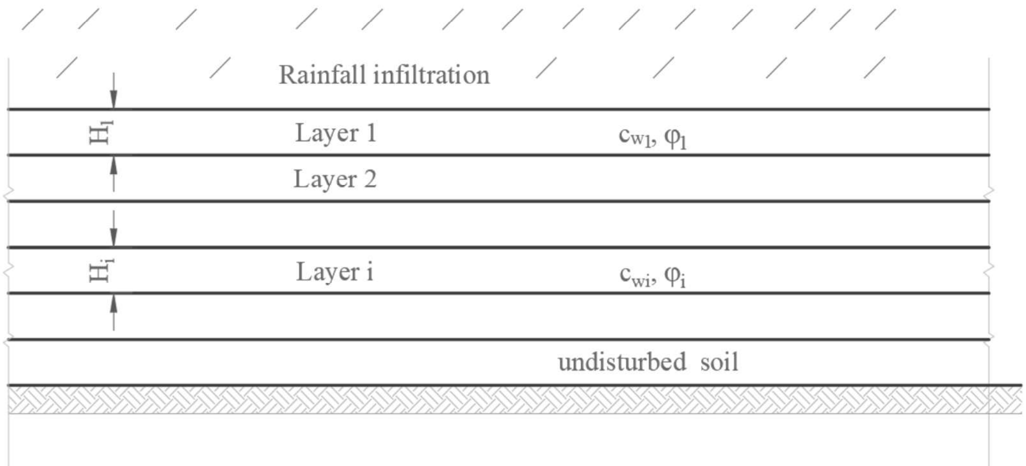


Fig. 13 – Vertical section of the soil with discretization in horizontal layers.

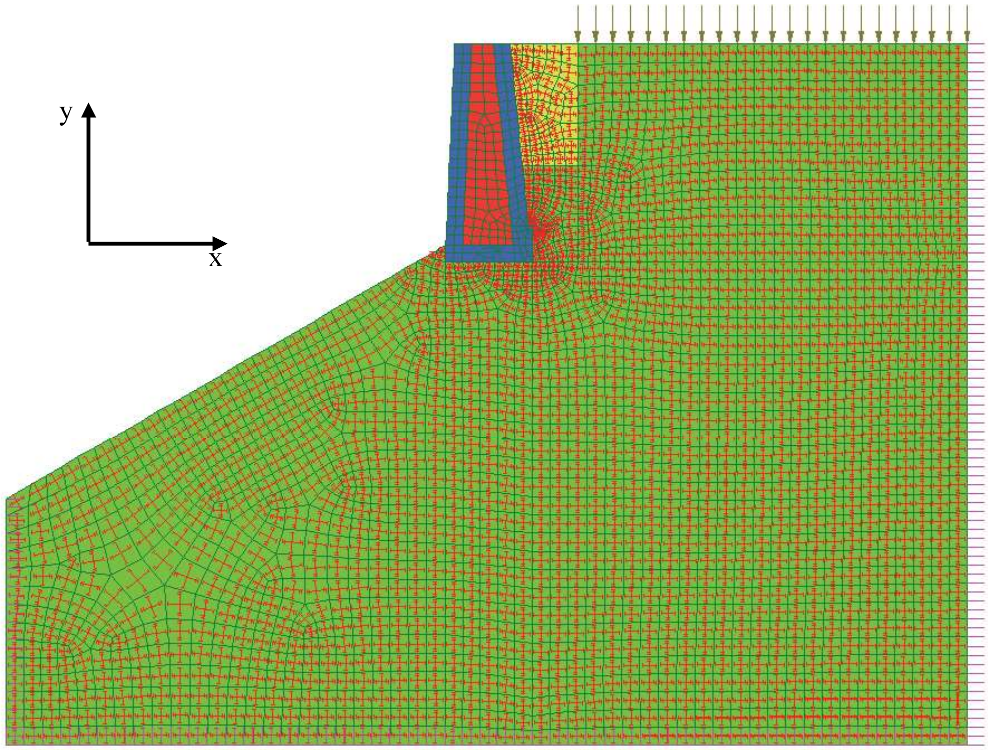
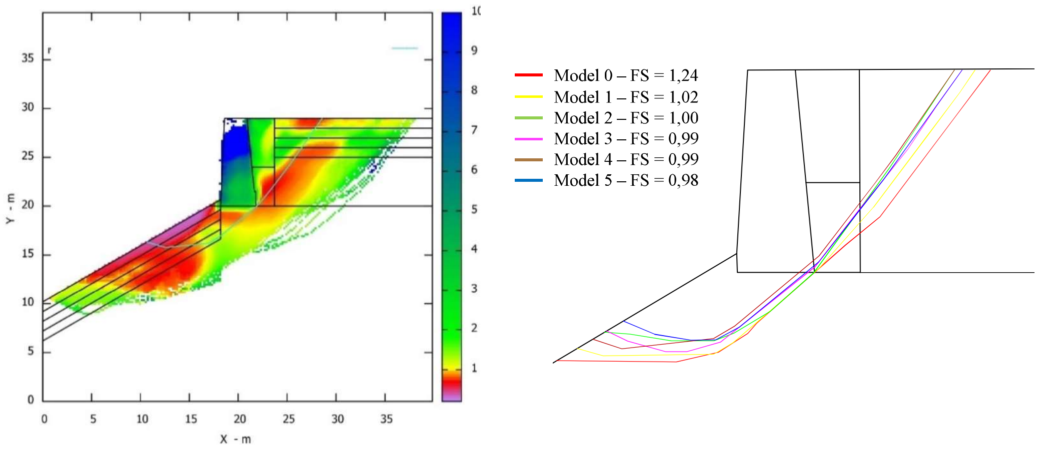
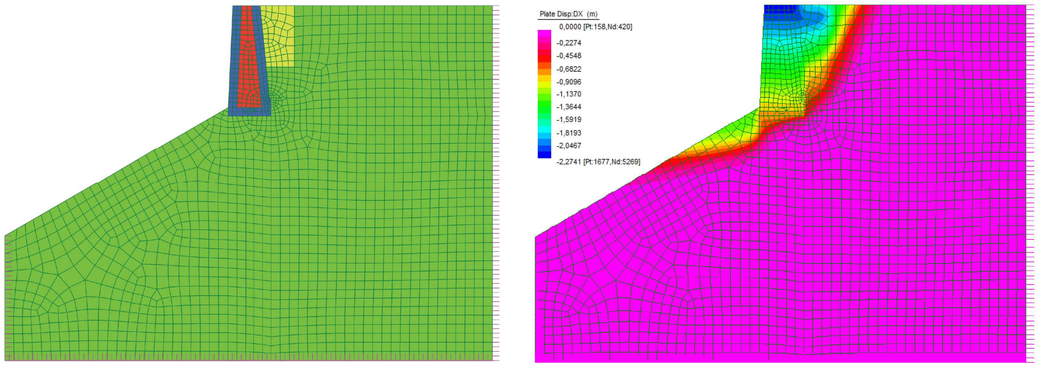


Fig.14 – Model of the section 92-93 (Straus7). The mesh is more accurate in which stress concentrations are expected.



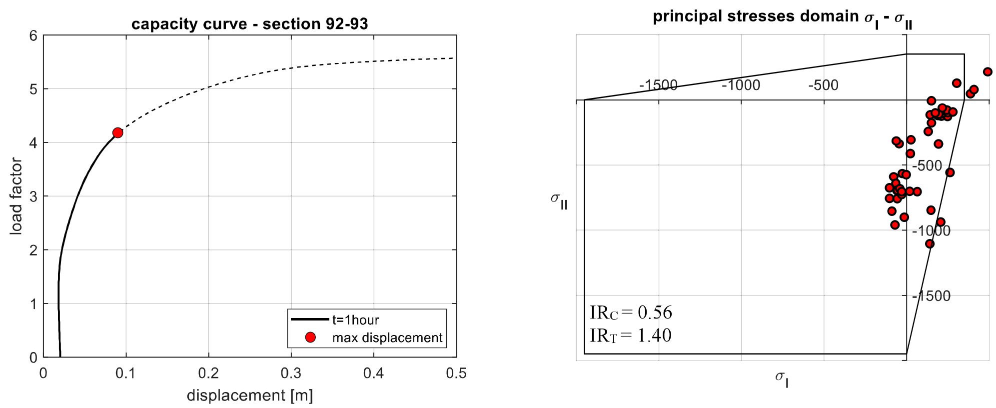
(a) (b)

Fig. 15 – LAM of section (92-93) code SSAP2010 (Borselli, 2018)*:); safety factors (a) and slip surfaces (b).*



(a) *(b)*

Fig. 16 – FEM Analysis of section (92-93); geometry and mesh (a); horizontal displacements (b).



(a) *(b)*

Fig. 17 – Pushover (a) and principal stresses domain (b) for a rain duration of 1 hour.

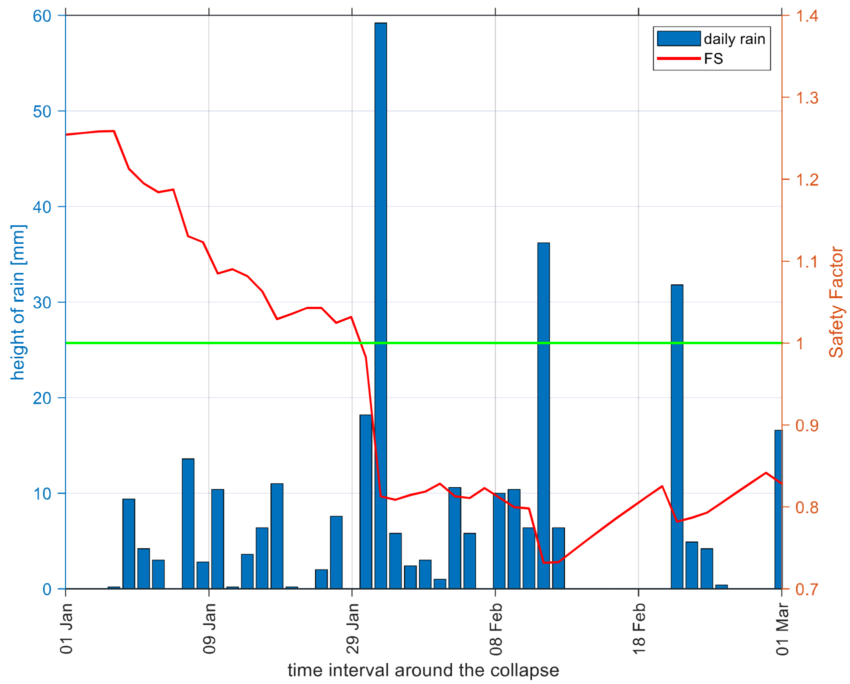
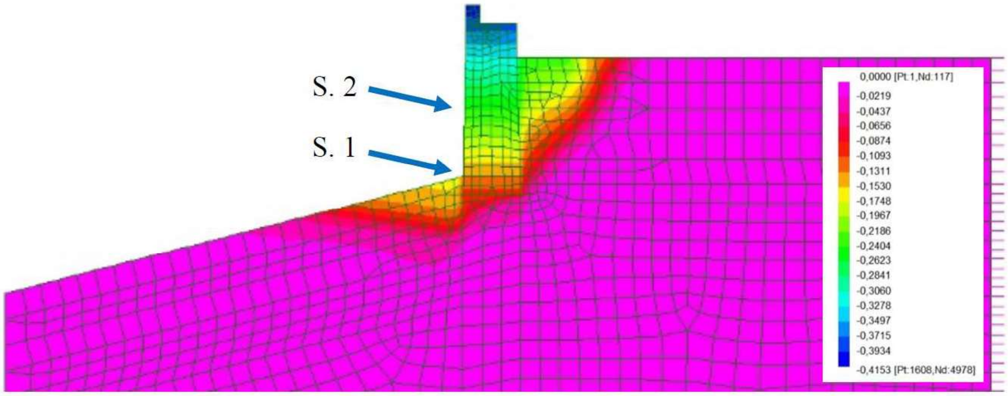
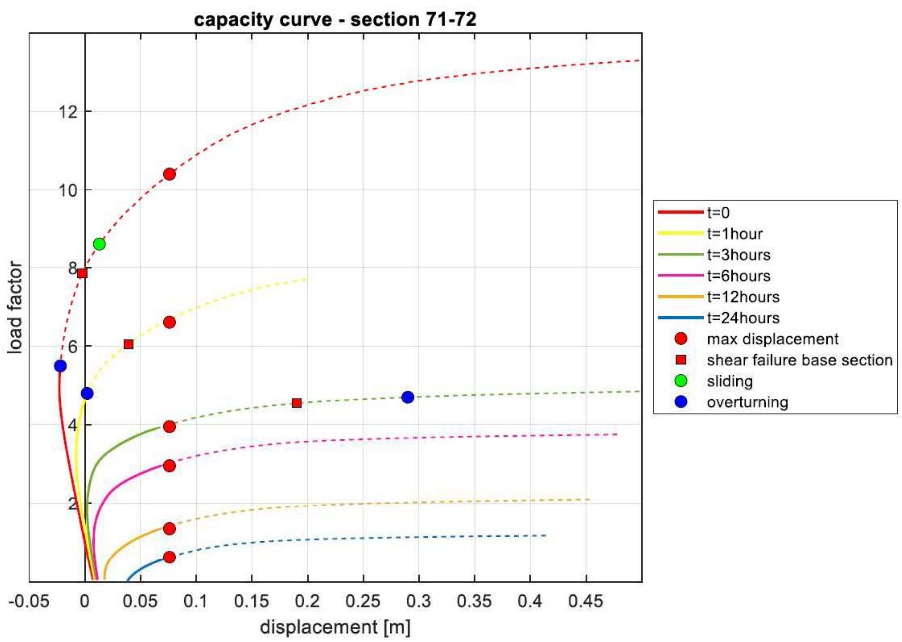


Fig. 18 – Sensitivity analysis of the effect of imbibition with the real rainfalls registered in the days around the collapse (30th January 2014).



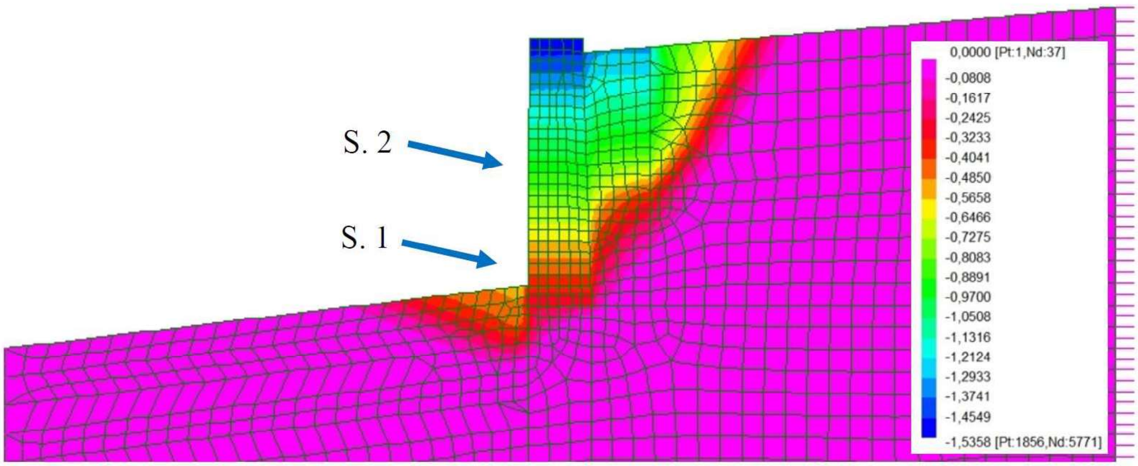
(a)



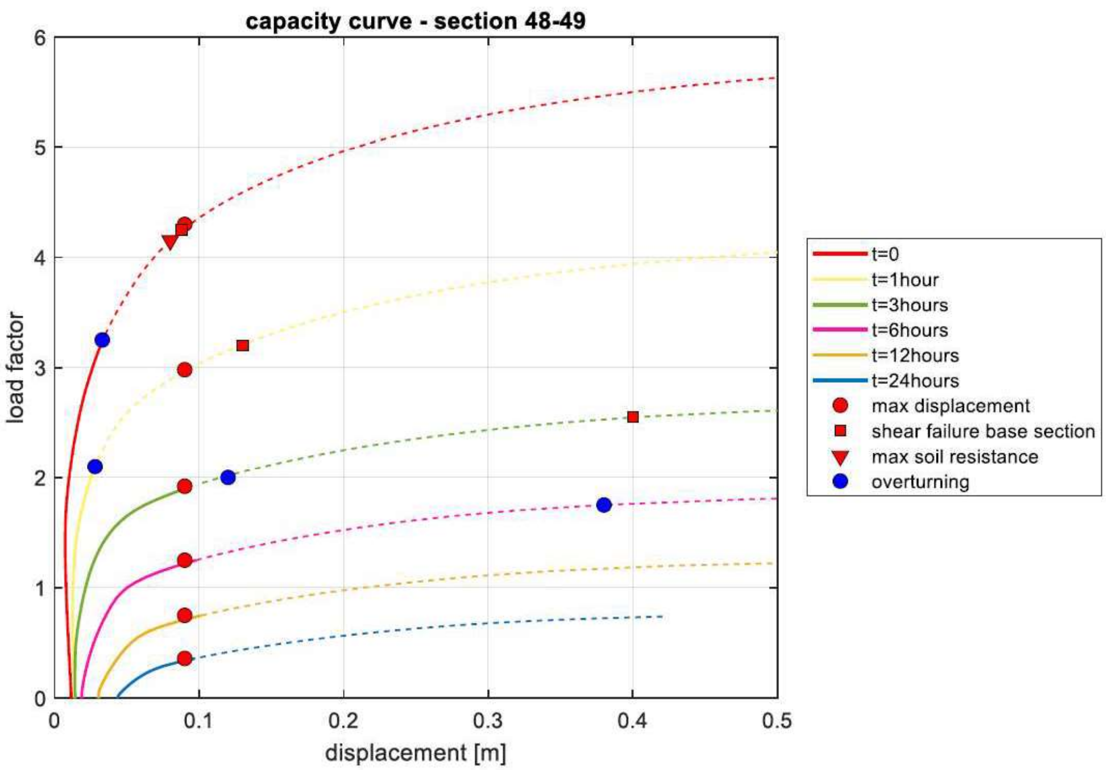
(b)

Fig. 19 – Section 32-33, (a) contour of the horizontal displacement [m] of model 5 and (b) results of the pushover curves for all the models.

Figure 20 Click here to access/download;Figure;Fig 20.pdf

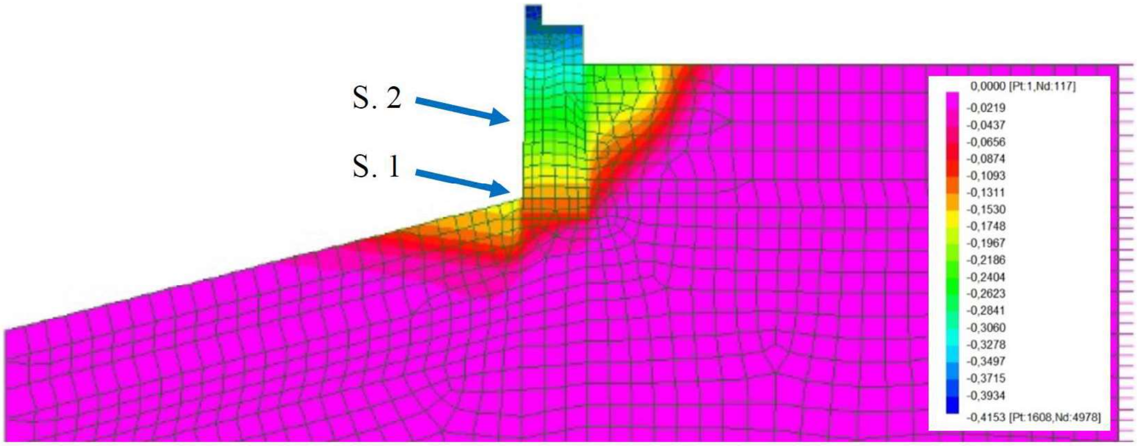


(a)

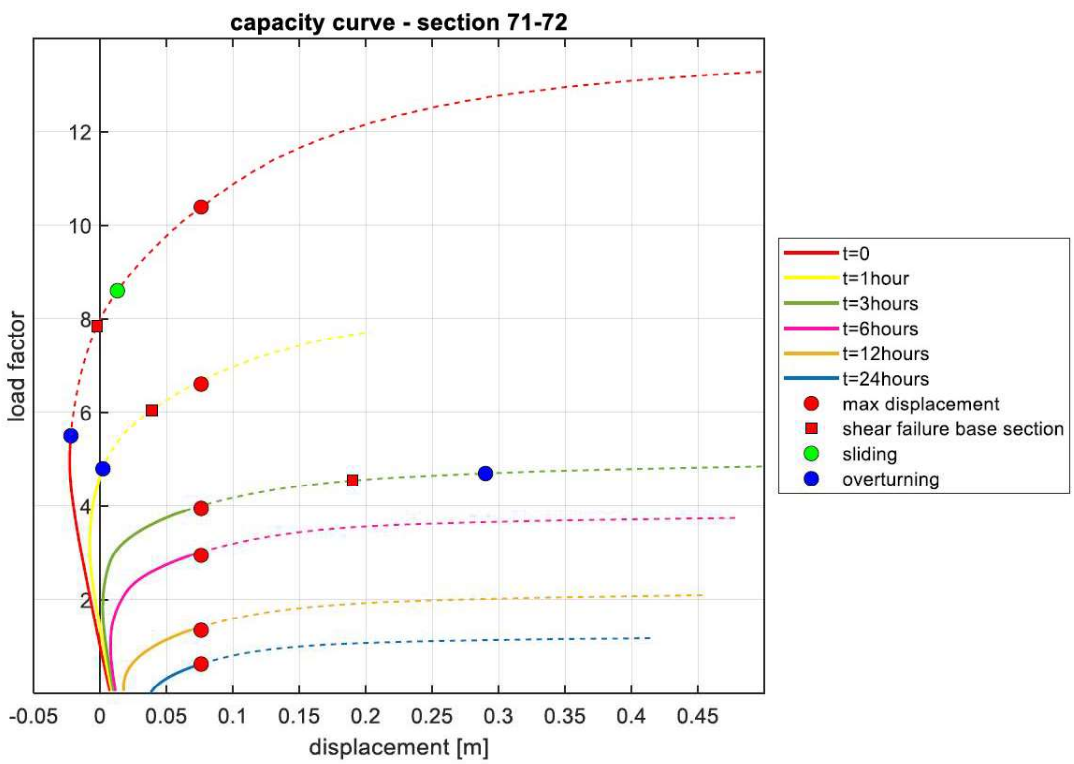


(b)

Fig. 20 – Section 48-49, (a) contour of the horizontal displacement [m] of model 5 and (b) results of the pushover curves for all the models.



(a)



(b)

Fig. 21 – Section 71-72, (a) contour of the horizontal displacement [m] of model 5 and (b) results of the pushover curves for all the models.