Mitigation measures of debris flow and landslide risk carried out in two mountain areas of North-Eastern Italy

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Citation: Genevois R, Tecca PR, Genevois C (2022) Mitigation measures of debris flow and landslide risk carried out in two mountain areas of North-Eastern Italy. Journal of Mountain Science 19(6). https://doi.org/10.1007/s11629-021-7212-6

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Abstract: The design of remediation works for the mitigation and prevention of the associated risk is needed where these geological hazards affect anthropized areas. Remedial measures for landslides commonly include slope reshaping, plumbing, drainage, retaining structures and internal slope reinforcement, while debris flow control works consist in open or closed control structures. The effectiveness of the remedial works implemented must be assessed by evaluating the reduction of the risk over time. The choice of the most appropriate and cost-effective intervention must consider the type of hazard and environmental issues, and selects, wherever possible, naturalistic engineering operations that are consequently implemented according to the environmental regulations or the design and specification standards imposed by the competent public administrations. The mitigation procedures consist of five basic steps: (a) acquisition of the knowledge of the hazard process; (b) risk assessment with identification of possible disaster scenarios; (c) planning and designing of specific remedial measures to reduce and/or eliminate the potential risk; (d) slope monitoring after application of remedial measures, (e) transfer of knowledge to the stakeholders. This paper presents two case studies describing the practice for the design of the mitigation measures adopted for debris flow and active landslide sites in North-Eastern Italy. The first case study is a debris flow site, for which, based on observation of past events and numerical simulations using the software FLOW-2D, the most suitable mitigation measures were found to be the construction of a debris basin, barriers and breakers. The second case study deals with an active landslide threatening a village. Based on the landslide kinematics and the results of numerical simulations performed with the code FLAC, hard engineering remedial works were planned to reduce the driving forces with benching and by increasing the available resisting forces using jet grout piles and deep drainage.

Keywords: Debris flows; Landslides; Geological hazards; Mitigation measures; FLO-2D; FLAC

1 Introduction

The scientific evaluation of hazards and risks induced by landslides and debris flows is a primary
concern in mountainous regions around the world; these mass movements often cause casualties and economic losses to housing, infrastructures, public services, roads, bridges, and the interruption of normal activities in the affected areas, such as agriculture, livestock farming, commerce and tourism (Schuster 1996; Froude and Petley 2018; Winter et al. 2019; Antronico et al. 2020; Emerson et al. 2020; EM-DAT 2020; Mirus et al. 2020; ADREM et al. 2021; Bowman 2022). Notably, Italy is among the countries leading in mass movements severity with an estimated annual cost ranging between $1 billion to $2 billion (Popescu and Sasahara 2009; Trezzini et al. 2013; Bowman 2022). Mitigation and prevention actions are taken based on the willingness of local stakeholders and supported by scientists and professionals (Winter and Bromhead 2012).

A very extensive literature has covered in recent years the topic of landslide and debris flow mitigation strategy (e.g. Abramson 2002; Spiker and Gori 2003; Prochaska et al. 2008; Lacasse and Nadim 2009; Cheng and Lau 2014; Hungr 2016; Cui et al. 2021). The choice of the most appropriate and cost-effective interventions for stabilising slopes and the subsequent design of remediation works are area specific and depend on various factors such as type of hazard, acceptable risk, desired stability (factor of safety), site’s environmental issues, aesthetic value, and prioritise naturalistic engineering operations which are implemented according to the directions provided by different public institutions.

Inspections must be carried out in the preliminary phase of design, planning the most appropriate field surveys for the problem to be addressed, performing geological and geotechnical investigations, and analysing and processing the results provided by surveys. On the basis of surveys and studies, technical professionals are able to identify the type of hazard and evaluate the best risk mitigation operations (Popescu 2001). Continuous or periodic real-time monitoring of the principal mass movement parameters, namely displacements, velocity and groundwater level plays an essential role in the design procedures (Tohari et al. 2011; Zaki et al. 2014). Hungr (2016) recommends a balanced approach combining factual observations, judgment and analysis. From the design information it is possible to determine the most appropriate remedial measures that would ensure acceptable safety levels for people and infrastructures while guaranteeing high standards of quality relative to the cost/benefit ratio.

The most used remedial works for landslide stabilisation include slope reshaping, plumbing, drainage, retaining structures and internal slope reinforcement (Royster 1979; Highland and Bobrowsky 2008; Arbanas et al. 2009; Serdarevic and Babic 2019), with a view to protection from erosion, mitigating the environmental impact through biotechnical slope stabilisation, and re-naturalization of land, so that locations and infrastructure may then be reintegrated (Pepe et al. 2020; Turconi et al. 2020). Different types of debris flow control structures are sometimes used in conjunction with one another (Hungr et al. 1987; VanDine et al. 1997; Prochaska et al. 2008; Cui et al. 2015; Genevois et al. 2018): open control structures (unconfined deposition areas; baffles; check dams; lateral/deflection/terminal walls, berms, or barriers) and closed control structures (debris racks, or some other form of debris-straining structures located in the channel; debris barriers and storage basins). Equations to aid the design of these mitigation structures have been outlined by various authors (e.g. Hungr et al. 1984; VanDine 1996; Cui et al. 2015). The remedial measures design, both for debris flows and landslides, must be flexible enough to adapt to any possible changes during or after the realization of the same works.

This article presents two case studies, focusing on practical experience in the design and realization of different remedial works within a debris flow site and a landslide area, both located in the Dolomites, North-Eastern Italy (Fig. 1), known worldwide for its naturalistic and touristic significance. The main aim of these studies is to describe the performed interventions as well as examining their efficiency and post-construction effectiveness.

The first case study considers the debris flows that seriously threaten the left side of the Boite River valley, at Fiammes, (46°34’09” N; 12°08’11” E) near Cortina d’Ampezzo (North-Eastern Italian Alps). The site is affected by a hill-slope debris flow which often changes its flow path from event to event, after the avulsions caused by damming and/or overflowing in the middle and lower path. Discharges of debris in the existing depositional basin often result in blockage of the road, damaging houses and possibly damming the Boite River. After the 1997 debris flow event, with an estimated volume of 25,000 m³, studies were performed on the implementation of the mitigation works design.
The second case study presents the Perarolo landslide (46°23'57" N; 12°21'22" E), a complex secondary process, involving a volume of about 180,000 m$^3$ at the lower portion of a larger old landslide, on the left side of the Boite River valley. The landslide is very active and subject to continuous movements, causing a very high-risk condition for the near Perarolo di Cadore village (North-Eastern Italian Alps), as a possible sudden slope failure could produce a temporary dam, blocking the Boite River valley and directly damaging the village (Teza et al. 2008).

2 Debris Flow Mitigation and Control Works: Fiames Site

2.1 Study area

The rock basin (Fig. 2) is composed of Upper Triassic to Lower Jurassic massive dolomite and limestones, producing coarse debris which consists mainly of gravel and coarser elements (> 10 cm), with boulders up to 3-4 m. Rock debris forms a thick talus on the slope, from the base of the rock cliffs to the valley bottom (1268 m a.s.l.), including heterogeneous scree, alluvium and old debris flow deposits. Three major channels and some minor ones originate from the same source area. The main morphometric parameters of Fiames site are reported in Table 1.

![Fig. 2 South-westerly view of the drainage basin in Fiames, displaying rock basin (red dashed line), source area, main channel, and depositional area. Image Google Earth 2017.](image)

![Fig. 1 Location of study sites in the Dolomites (North-Eastern Italy). 1. Fiames; 2. Perarolo di Cadore.](image)

Table 1 Main morphometric parameters of Fiames site

| Parameters                                 | Value   |
|--------------------------------------------|---------|
| Rock basin area (km²)                      | 0.19    |
| Basin maximum elevation (m a.s.l.)         | 2450    |
| Rock basin outlet elevation (m a.s.l.)     | 1786    |
| Source area mean slope (°)                 | 40      |
| Total main channel length (m)              | 1500    |
| Main channel depth (m)                     | 3-8     |
| Main channel width (m)                     | 10-22   |
| Mean main channel slope (°)                | 20      |
| Apex of deposition area elevation (m a.s.l.) | 1500 |
| Valley bottom elevation (m a.s.l.)         | 1268    |
| Deposition area mean slope (°)              | 10      |

Most recent flows, from 1992 to 2013, had volumes ranging from 8,000 m$^3$ to 25,000 m$^3$. After the 1997 event, to protect the National Road S.S. 51, the artisan production area located just below the main road, and to prevent the possible damming of the Boite Torrent, a retention basin was built (Fig. 3). The main components of this basin, the general layout of which is shown in Fig. 3, are a 220 m long and 4 m high confining dike made of the same debris flow material (dike A in Fig. 3), a shorter dike 40 m long (dike B in Fig. 3), a boulder drain and a large diameter culvert pipe, under the roadway and up to the Boite River, and a natural debris berm (upstream the dike A), (Genevois et al. 2018).

Due to topographical constraints the shape of the storage basin is very narrow (maximum bottom width
about 20 m) and due to high viscosity proved itself unsuitable to evenly distribute the flowing material, in the whole basin, as it tends to deposit mainly along the flow direction rather than to spread laterally as with low viscosity fluids. Although the 1997 debris flow had a total volume of 25,000 m³, and a debris flow basin should have a capacity equal to the Design Debris Event (DDE) (Prochaska et al. 2008), the storage basin was built with a capacity of only 15,000 m³, so it was not effective in containing subsequent flows.

2.2 Mitigation measures design

Passive measures such as zoning and planning with consequent property relocation or land-use limitations, were not considered possible due to the presence of the Boite River and the impossibility to move the National Road. When debris flow passive measures are not applicable (Huebl and Fiebiger 2005) the general purpose of active mitigation measures focuses on hazard reduction, mainly consisting in controlling the velocity and course of descent, and providing debris containment at a safe location at the base of the slope. The design for debris flow mitigation works is supported by the large scientific literature (e.g., VanDine 1996; Armanini 1997; Okubo et al. 1997; Huebl and Fiebiger 2005; Wendeler et al. 2006; Kaitna et al. 2007; Mizuyama 2008; Hungr et al. 1987). The following criteria were considered: to construct relatively low-cost structures according to the site topography and using local materials; to minimize the risk to users of existing infrastructures; to minimize the amount of coarse-grained sediment entering the storage basin from future debris flows. The most appropriate mitigation measures designed for the Fiames site are described below.

To dissipate the energy of debris flows, filter coarse solid components and deflect flows from the areas at risk (Genevois et al. 2018), mitigation structures were designed to be built on the lower fan of Fiames, upstream the National Road S.S. 51, taking into account the existence of the numerous possible flow channels. The main components of the mitigation work are (1) a terminal debris dike; (2) a storage basin upstream the berm to trap the debris within the deposition area at the base of the slope, and a road access for the basin cleaning after an event; (3) a debris barrier and baffles along the main flow path or depositional area to disperse the debris flow, to contain part of the debris and to sustain the full impact force, and to control the velocity. The general layout of these features is shown on Fig. 4 and the design procedures for each of these components are hereby presented.

2.2.1 Predicted debris flow volume

A predicted debris flow volume is a rational basis for basin design, since volume will dictate the capacity...
of the structure and is also a good indicator of the event hazard (Jakob 2005). In the following years a more accurate study of the actual geotechnical properties of the debris material has been carried out with numerical modelling of debris flows, to evaluate the pressure exerted by the flow on a mitigation structure.

The natural dry bulk density evaluated through on-site replacements tests ranges from 1,960 to 2,160 kg/m³ (Marchi and Tecca 1996); similar values were obtained by Iverson (1997).

Debris flow material has been sampled along the flow channels and in the deposition area, the main geotechnical properties of the fraction < 2 mm are reported in Table 2.

Table 2 Debris flow mixtures properties

| Parameters                        | Value       |
|----------------------------------|-------------|
| Cohesive strength (Pa)           | 0           |
| Effective internal friction angle (°) | 38-42       |
| Void ratio                       | 0.33–0.36   |
| Saturated density (kg/m³)        | 1960–2160   |
| $D_{60}/D_{30}$ ratio            | 3.9-14.3    |

The DDE was estimated after appropriate calibration of the model based on the September 1997 event, with the two-dimensional finite difference flow routing model FLO-2D (O’Brien 2006) for simulating water and non-Newtonian sediment flows. When routing hyper concentrated flows such as mud or debris flows, the momentum equation includes the viscous and yield stresses. For a complete discussion of the model attributes, see the FLO-2D User’s Manual (O’Brien 2006).

The predicted debris flow volume is 30,000 m³. A hazard map, based on a methodology developed by Garcia et al. (2003, 2004) was then created, with process intensities defined in terms of a combination of flow depth $h$ and the product of $h$ and velocity, in each grid element of the computation area, outlining the areas characterized by 3 hazard levels, from low to high.

2.2.2 Predicted impact force on structures

FLO-2D calculates the pressure induced by the impact of the debris flow with the barrier in dynamic conditions $P_i$, as a force per unit length (N/m); the user can then multiply the $P_i$ by the structure length within the grid element of the computation area to obtain a maximum impact force on the barrier (O’Brien 2006).

The modulus of the impacting force $F_i$ (N) is computed by the momentum equation:

$$F_i = \rho_m g v^2 A \sin \beta$$  \hspace{1cm} (1)

where $\rho_m$ is the mean density (kg/m³) of the impacting fluid, $g$ is the gravitational acceleration, $v$ is the velocity of this fluid (m/s), $A$ is the impact surface of the grid element of the barrier (flow depth $h \times$ structure unit length $l$) in m², and $\beta$ is the angle between the barrier and the flow direction in deg, in order to evaluate the effective force component normal to the barrier itself.

The impact pressure $P_i$ produced on a structure arranged perpendicular to the flow direction is calculated according to the hydrodynamic model (Cui et al. 2015) and in this specific case with the following expression:

$$P_i = 1.5 \rho_m v^2$$  \hspace{1cm} (2)

where $P_i$ is the impact pressure of the debris flow (N/m²), the other variables are described as in Eq. (1) and 1.5 is a multiplying factor of flow depth introduced by Hungr et al. (1984) to account the formation of a stagnant debris wedge in front of the barrier toe, assuming a debris flow density equal to 2,000 kg/m³ and a mean velocity value of 5 m/s, as shown by the magnitude of the velocity vectors computed by FLO-2D.

2.2.3 Terminal debris berm specifications

Debris barriers and storage basin are located across the debris flow path and designed to encourage deposition. The debris-straining structure, a debris barrier, must incorporate a weir or spillway into the structure to allow fine-grained sediment and water to escape, while the coarse-grained debris is contained within the storage basin located upslope of the barrier. The new embankment is located in continuation of the pre-existing one, optimising both the existing containment structure and the geometry of the depositional area. FLO-2D simulation results (Figs. 5a and 5b) suggest a dike made of the same granular material from the excavation of the basin would have a total length of 800 m, a minimum height of 4.5 m, and a ridge width of 2.5-3.0 m.

The possibility of creating only unconfined deposition areas (VanDine 1996) was excluded for operational and reliability reasons, in addition to the significant landscape impact that it would have, as well as the connected works that involve substantial modifications of the natural flow paths.

2.2.4 Storage basin specifications

To reduce the gradient and to increase storage
capacity the area upstream of the debris barrier can be excavated. Design considerations include: design magnitude or volume of a debris flow, size and gradation of the coarse-grained debris, potential runout distance, impact forces, and deposition angle.

Terminal berms, or barriers are constructed across the path of a debris flow to cause deposition being a physical obstruction to flow. Once a debris flow has been deposited upstream of a terminal structure, the coarse-grained debris must be removed from the area. Terminal walls, berms, or barriers are usually located as far as possible downstream from the apex of the fan to obtain a larger area for deposition and to minimize the impact forces and run-up on structures. The artificial deepening of the deposition area lowers the gradient, increases storage capacity, and decreases runout distances, impact forces, and run-up.

### 2.2.5 Baffles specifications

Impediments to flow, or baffles, are primarily used to slow down a debris flow enhancing its deposition, and are often placed in unconfined areas. In the main or depositional area, they are also used to separate the coarse-grained debris from the fine-grained debris and water of the debris flow, thus encouraging the coarse-grained portion to be deposited, causing the reduction of the solid concentration of the flow, with a reduction of its viscosity. The baffles can be constructed of earth berms, timber, or steel, and can be emplaced as single units, in lines or staggered. To be effective, the coarse-grained debris must be removed from behind the straining structure after every event. The slit aperture is designed at 1.5 to 2 times the maximum mean diameter of the boulders; the openings used for the straining structures associated with debris barriers and storage basins range between 0.5 and 1.00 m. The configuration and number of baffles and their spacing is adapted to the channel characteristics and mitigation requirements and must be designed with respect to the eventual debris clean-out.

To increase functionality and efficiency at the Fiames site 15 staggered baffles would be constructed upstream the terminal dike of the storage basin. The baffles, placed within the three main flow channels, would consist of rows of cylindrical steel elements, embedded in a reinforced concrete foundation, and designed to sustain the impact forces of individual boulders. The baffles (Fig. 4) would have row lengths and heights of about 20-25 m and 2.0 m respectively.

The openings have been designed also in relation to their position on the slope with respect to the main flow channel. Two baffle rows are placed along the main flow channel with openings 1.5 m wide, and a baffle row with openings 0.8-0.6 m wide is placed at the outlet of the main channel. The other baffle rows would have smaller openings. A road access to clean out to the structures would be constructed.

### 3 Landslide Mitigation and Control Works: Perarolo Site

#### 3.1 Study area

The Perarolo area is affected by a very active "secondary landslide", characterized by a mean displacement rate of about 20-25 cm/year and involves the lower part of an older largest rockslide affecting the entire southern slope of Mt. Zucco up to the Boite River valley. It is located between 630 m
and 530 m a.s.l., with a longitudinal length of about 150 m and a width of about 100 m. Due to reshaping by means of four terraces about 3 m high, the upper part of the landslide slope has a slight gradient. Downhill the gradient increases forming a sub-vertical scarp 75-80 m high, ending in the valley floor. Screes, accumulating at the base of the slope, are continuously removed by the Boite River periodic floods (Fig. 6).

Fig. 6 North-easterly view of the study area. Image Google Earth 2004. Landslide edge in red.

The slope is formed of Triassic dolomitic and anhydritic limestones, dolomites and marls, locally affected by karstic cavities and dissolution cracks. The Triassic formations are covered by a thickness of about 30 m of moraines, colluvium, and old landslides deposits. The geological structure of the bedrock is articulated by several vertical faults and a low angle overthrust that locally doubles the stratigraphic sequence (Squarzoni et al. 2005b; Teza et al. 2008). Through the observation of stratigraphic logs, the failure surface was located at a maximum depth of about 30-32 m, in correspondence of the transition between the weathered top of the underlying anhydrites and the anhydritic limestones. The soils of the landslide mass host a small aquifer, supported by deeper and relatively less permeable levels and partly by the upper altered part of anhydritic limestones. The water level strongly fluctuates over time in relation to meteoric events.

The landslide of Perarolo, which dates back to the 19th century, has been the topic of numerous studies and remediation works by different State and Regional Organizations, due to the risk associated with the possible damming of the Boite River and subsequent flooding of the Perarolo village. The actual hazard of the slope was further highlighted by the Italian State Railways who decided to definitively abandon the local railway line in 2002, after many geotechnical and geophysical investigations and the construction of long retaining walls, one of which anchored. Following small and continuous mass movements affecting the steepest part, the slope was monitored with inclinometers and ground-based and Global Positioning System techniques, that indicated in the last decade, maximum displacement speeds of about 1.0-1.5 cm/month, particularly in the western sector. In addition, numerous terrestrial laser scanner and interferometry radar measurements have been carried out (Squarzoni et al. 2005a; Teza et al. 2008; Genevois et al. 2012).

From early 2000s, the surface and deep displacements have almost linearly increased, showing a rotational failure mechanism and a quasi-constant trend of displacement velocities. Highest water levels were registered just above the deepest slip surface identified by the inclinometers. However, their fluctuations, in the order of about 2 m, could not be related to local rainfall but only to the regional seasonal rainfall regime.

3.2 Geological and geotechnical investigations

A large number of surveys have been conducted in the area since the early 1990s (Fig. 7).

New field investigations were carried out to investigate the lithological and hydraulic properties variations in the whole landslide body and the fracture intensity and weathering of the bedrock. These investigations were carried out by means of 20 boreholes, at depths between 20 m and 45 m, with SPT and Lugeon tests; 3 down-hole tests; 3 seismic profiles, and 3 electrical resistivity profiles (ERT). The collected data identified the following litho-stratigraphic succession, from the top to bottom: (1) coarse debris, in a sandy or silty matrix, loose to medium dense, with irregular clayey lenses, forming the body of the more recent landslide; (2) alternations of sands and silts, occasionally gravels, mostly medium dense, representing ancient recurrent shallow landslides bodies mixed with colluvium and glacial deposits; (3) coarse debris and blocks in a
sandy or sandy-silty matrix, dense to very dense, probably a more ancient landslide body; (4) anhydrites and anhydritic limestones deeply fractured and weathered, with dissolution cavities, corresponding to the top of the formation involved in the primary landslide triggered in the upstream dolomitic and anhydrite limestones cliffs; (5) thin-layered anhydrites and anhydritic limestones, with mostly small scale dissolution cavities; (6) compacted marls.

Physical and mechanical parameters of the landslide materials were determined through both laboratory and field tests. 20 remolded and undisturbed samples of soil and rock were collected at depths between 5 m and 30 m for geotechnical laboratory tests. The soils resulted mainly coarse, unsaturated, loose to very dense, frictional but with some cohesion value. Anhydritic limestones are characterized by very variable physical and mechanical values to be related to both the content in gypsum and/or anhydrite and to the presence of dissolution cavities (Hoxha et al. 2006; Wichert et al. 2018).

The investigations identified in the landslide body three distinct interfaces, characterized by significant variations of elastic parameters and physical properties, located at depths of about: (1) 10 - 12 m, between loose superficial and underlying medium dense to dense material; (2) 25 m (upstream) to 17 m (downstream), marking the transition between dense to very dense soils; (3) 18 – 40 m, at the bedrock top.

3.3 Mitigation measures design

When mitigation strategies such as zoning and land use planning are not applicable due to the presence of buildings and infrastructures in areas

![Fig. 7 Plan of the upper landslide area (orange dashed line) with position of field investigations: Borehole (red dot); Down-hole test (black dot); Seismic refraction profiles (blue line) and ERT (red line).](image-url)
prone to landslide hazards, the general purpose of landslide mitigation measures refers to several man-made hazard specific control activities on slopes with the aim of reducing the effect of landslides.

The design of landslide remedial works, currently supported by an extensive scientific literature, can be implemented to increase the shear strength of the unstable mass or to introduce active external forces (e.g. anchors, rock or ground nailing) or passive forces (e.g. structural wells, piles or reinforced ground), and to counteract the destabilising forces (e.g. Royster 1979; Highland and Bobrowsky 2008; Arbanas et al. 2009; Galli et al. 2017; Serdarevic and Babic 2019; Pepe et al. 2020; Panigrahi RK 2022).

The most appropriate mitigation measures for the Perarolo landslide were designed in 2005, based on the morphological, geotechnical, and kinematic characteristics of the site and taking into account the scientific literature available at the time (e.g., Hutchinson 1977; Popescu 1996; Holtz and Schuster 1996; Pinto et al. 2005). The remedial works, to be implemented in the upper part of the slope, mainly consisted in controlling the groundwater regime and in the use of a deep ground reinforcement system.

### 3.3.1 Numerical analysis

The numerical analysis of the Perarolo landslide (Cioli et al. 2012) has been performed using the two-dimensional explicit finite difference code FLAC 4.0 (Itasca Consulting Group Inc. 2000), which provides the displacement and velocity values and their distribution in the landslide mass, considered as a continuum medium. The geotechnical model has been obtained by simplifying current complexities (Laloui et al. 2009): existing soils have been grouped in a discrete number of geotechnical units, sufficiently representative of reality, and the observed variability of the parameters has been simulated considering statistical value ranges for each one. A hydrostatic condition, with the highest groundwater level observed over many years, was taken into consideration. The appropriate constitutive model of each unit was chosen relying on the lithological characteristics of the different materials and the stress state: a conventional Mohr-Coulomb model for soils and a simple elastic model for the bedrock. An anisotropic plasticity model, including weak planes in a Mohr-Coulomb solid (ubiquitous-joint model), has been assigned to the deeply fractured and weathered anhydrite bedrock to consider the higher fracturing degree and the orientation of the stratification. The geotechnical parameters (with mean and standard deviation values) used in the numerical analysis are displayed in Table 3.

The simulation results showed that maximum unbalanced forces equilibrium was not reached and that due to the progressive displacements increase the slope was not stable in presence of the considered groundwater level. Horizontal displacements show an unstable soil mass, limited to the upper part of the slope, with a maximum thickness of about 28-30 m (Fig. 8a).

However, the distribution of the horizontal velocity and displacements highlights three superimposed bodies with decreasing velocities from the top to the bottom, limited by surfaces located at depths of about 10-12 m, 20-25 m and 25-30 m (Fig. 8b), approximately coinciding with those identified by the investigations carried out. Unstable conditions were obtained even by lowering the groundwater level well below the minimum recorded values.

### 3.3.2 Control works design and specification

The analyses of the stability conditions highlighted a complex geological and geotechnical condition, where the most suitable stabilisation works...
appeared to be deep ground improvement, as other stabilisation works were either not feasible or did not provide appreciable results (Hutchinson 1977; Jones 1991; Abramson 2002; Popescu 1996).

The designed stabilisation scheme included both a groundwater drainage system and deep ground improvement (Popescu 1996; Cheng and Lau 2014). The drainage system consisted of a limited number of deep wells bored to the stable rock basement, to prevent the formation of temporary groundwater in the unstable mass. Initially, in 2005, the designed solution for the deep soil reinforcement consisted of five rows of jet-grout piles up to the anhydrite limestone top (about 30 m long and diameter 80 cm),

![Fig. 8 Horizontal displacement contours (a) and velocities (b). Marls (light grey); Anhydrite limestones (dark grey).](image)

![Fig. 9 Stabilization works in the upper landslide area (orange dashed line).](image)
spaced five meters apart, alternatively reinforced with micropiles driven at least 10 m inside the underlying anhydrite limestones. Due to operative difficulties in the steepest part of the slope, the three downstream rows of jet-grout piles have been replaced, in 2007, with triplets of 40 m long micropiles, two of which 10° downhill and one 40° uphill dipping.

Groundwater level and pore pressure were controlled in boreholes and piezometers in the whole area (Fig. 9); the data, collected for a period of approximately 200 days, did not display any volumetric alterations of the ground surface, highlighting a continuous increase of both deep and surface displacements.

The effectiveness of the designed stabilisation system was preliminarily evaluated by means of a numerical analysis, considering the maximum possible groundwater level, but not the existing soldier pile wall, built at the end of the 90s to protect the railway, whose efficiency was not guaranteed. Only the steepest part of the slope, where it was not possible to carry out the designed remedial measures, turned out to be still unstable for a maximum thickness of about 10-12 m (Fig. 10).

Advances in geotechnical engineering science are the result of the dissemination of historical case studies as they provide real data against which designers can test their predictions with the predicted event (Lambe 1973).

The article presents two case studies, both focusing on practical experience in the design of specific mitigation measures of a debris flow site and of a soil-rock landslide. Remedial works implementation was based on data collection from engineering geological and historical studies, numerical simulations of the landslide and debris flow before the construction of stabilisation works, choice of remedial works and stabilisation proposals and numerical simulations in presence of the proposed mitigation measures. However, both designed projects failed for different reasons and an attempt is made to identify the factors which may have contributed to these failures.

Following the criteria indicated by the scientific literature existing at the time (e.g., Huebl and Fiebiger 2005; Hungr et al. 1987), the design for debris flow mitigation works at Fiames consisted in a terminal debris berm, a storage basin with its access road in the deposition area and a debris barrier and baffles along the main flow paths. The mitigation works were designed considering the DDE and the impact forces on structures, giving specifications for the terminal debris berm, storage basin and baffles. All information and specifications resulted from collected historical data, morphological characteristics of the area and results of simulations carried out with the FLO-2D code, pre- and post-construction of the planned works. The control works project was approved by the Local Authorities, but only partially implemented, leaving out the construction of the prescribed number of baffles and neglecting the control works maintenance, namely the emptying of the retention basin and the cleaning of the baffles after each debris flow event.

Regarding the Perarolo landslide, the design of the stabilisation works relied on the results of field investigations and their elaborations. The landslide body is characterized by a complex structure due to a set of soils with different geotechnical characteristics and behaviour, displaying a marked spatial variability. The landslide stabilisation was addressed towards the improvement of the shear strength of the involved soil mass (Poulos 1995; Popescu 1996; Chen et al. 1997; Ashour 2004). The mitigation measures,
implemented in the upper part of the slope, consisted in the control of the groundwater regime by means of deep wells and the implementation of a soil improvement system, obtained by 5 rows of steel tube reinforced jet-grout piles and micropiles triplets, driven 10 m inside the underlying anhydrite limestones. After a preliminary field test, remedial works were realized between 2006 and 2007, but the monitoring of the topographic targets revealed that the landslide continued to move with horizontal displacement velocities from 1.1 to 1.5 cm/month (Fig. 11).

The unsuccessful performance of the designed remedial works was due to a more complex than expected behaviour of the sliding soil mass, which has influenced the geotechnical parameters and the response of the structural works. The geo-mechanical model used for the analyses (Barbour and Krahn 2004) is a necessary simplification that may not fully reflect the actual variability of soils characteristics, besides the unavoidable measurement error. Complex and strongly variable geological deposits are typically characterized by a strong vertical and lateral anisotropy of the shear strength and hydraulic characteristics. The numerical simulations carried out have considered the inherent variability of each soil by introducing the average value and the standard deviation for significant parameters, but they did not consider the anisotropy of soils, a feature not implemented in FLAC for the Mohr-Coulomb model, if not only for some specific surfaces. However, the lack of both physical and geo-mechanical correspondence between the actual structure of the landslide body and the considered numerical model, may not be the only determining cause of the remedial works failure.

The real efficiency of the implemented system of piles depends on the soil-pile interactions, in particular on the resistances mobilized by piles subjected to lateral loads, as in unstable slopes. The design of such piles is based on the assumptions that acting forces are mainly function of group and arching effects, that depend on the geometry of the piles system (piles spacing, length L and diameter D) and on the spatial distribution and geotechnical characteristics and behaviour of soils (Chen and Martin 2002; Liu et al. 2020; Hu et al. 2021). Since significant group effects occur if pile spacing is less than 6 D, the planned spacing (5 m) can be considered sufficient to develop a substantial arching effect.

A further relevant aspect is the pile length below the slip surface, that significantly affects the magnitude and distribution of pile deformation (Poulos 1995; Chen et al. 1997). The increase of the pile length above slip surface results in a decrease of both pile horizontal displacements and maximum absolute shear stresses, while bending moments exhibit an opposite trend. The monitoring of the pile heads also indicated significant vertical displacements, increasing the complexity of the response of piles to the stresses exerted by the sliding mass. The causes are not so obvious, but they could be related to the dissolution processes at the expense of the chalks present in the limestone bedrock (Alberto et al. 2008). Ultimately, the irregular distribution in the Perarolo sliding mass of both soils and stable bedrock led to a strongly irregular response of the piles to the stresses resulting from the sliding of the soil mass. When the ultimate flexural capacity of piles is reached, coinciding with the yield limit of inserted steel tubes, a rapid increase of soil mass displacements develops (Al-abboodi et al. 2020; Wang et al. 2020), as consequent landslide monitoring later showed.

5 Conclusion

Many mountain slopes pose a potential hazard by landslide or debris flow to human activities and structures located downslope. After detailed geotechnical studies of the threatened sites, suitable procedures should be chosen according to the different type of hazard. In particular, remedial measures are oriented to control the course and
reduce the velocity of a debris flow and to provide containment in a deposition basin, or to implement appropriate measures to stabilise landslides by shear stresses reduction and/or shear resistances increment.

The results achieved in both cases here presented deserve some consideration on the success or failure of designed projects.

In the Fiames debris flow site, the realization of the planned works was not completed and the lack of maintenance by Public Administration of cleaning basin and baffles upstream of the basin, which had the function of retaining the coarse debris, caused a rapid burial of the basin itself, inducing a risk of flooding on the National road below.

The failure of the remedial works of Perarolo landslide seems instead to be attributed to the geological and geotechnical complexity of the landslide body, typical of the so-called secondary landslides, despite the numerous investigation surveys carried out over the years. In this case it is more difficult to reduce the risk posed by the existing landslide, and the choice of mitigation measures should be rather addressed to urban planning strategies, environmental management and community preparedness or public outreach projects.

Concluding, in case of complex landslides it must be realized that it is not always possible to carry out fully effective stabilisation.

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Funding note: Open access funding provided by IRPI - PADOVA Area Di Ricerca Di Padova.

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