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Slope stabilization in difficult conditions: the case study of a debris slide in NW Italian Alps

Abstract A case study of a debris slide (estimated volume of about 35,000 m^3) is described in this paper. This slide occurred in April 2009 in the North Western Italian Alps (Aosta valley) and damaged the SR25 road along the Valgrisenche valley. Ground investigations started with severe safety and logistic issues being posed. Given the need to open as soon as possible the road, the design of the landslide stabilization works was carried out using a "design as you go" approach. The stabilization measures were conceived to be flexible in order to allow for changes and integration during construction, in line with the progressive refinement of the geological–geotechnical slope model being developed. Back analysis by means of the limit equilibrium method (LEM) and the finite element method (FEM) was used. Groundwater level rise following heavy rainfall and spring snow melting was found to be the main cause of the debris slide. The stabilization works were designed by using both the LEM and FEM methods. The stability conditions of the engineered slope were assessed based on the available performance monitoring data.

Keywords Landslide · Back analysis · Limit equilibrium · Finite element method · Remedial measures · Valgrisenche

Introduction

In Alpine areas characterized by high energy relief, slope instability phenomena are particularly widespread and represent one of the main natural hazards threatening human activities and civil infrastructures (Barla et al. 2010). These phenomena are responsible each year for huge property damages with both direct and indirect costs. Moreover, in the last few years, the severity of the landslide problem continues to worsen due to changes in land use, increased urban development, deforestation, and changes of climatic patterns.

In the framework of landslide hazard mitigation, attention must be paid to the selection of appropriate and cost-effective landslide remedial measures. In particular, when slope instability processes affect directly or indirectly roads or railways, the economic consequences of severing a transport artery for even a short period of time can far outweigh the remedial costs. In this case, a rapid repair of landslide damages is needed. This generally leads planners to operate in emergency conditions and to face with the difficulties of obtaining a rapid and complete landslide characterization.

The intrinsic variability of landslides (i.e., size, type, material, water content, activity) and their dependency upon special local circumstances, including geological, geotechnical, tectonic, and groundwater conditions, implies the need to change and adapt progressively the stabilization measures to adopt, as the instability is better understood.

Through the study of a debris slide (estimated volume of about 35,000 m^3), which occurred in April 2009 in the North Western Italian Alps (Aosta valley) and damaged the SR25 road along the Valgrisenche valley, the complexities which can arise when carrying out landslide stabilization works in emergency conditions are described in this paper. The SR25 road is the only connection between the Valgrisenche valley and the Aosta valley. Restoring the road was therefore required as matter of urgency.

Ground investigations, monitoring, and slope stabilization works started concurrently with severe safety and logistic problems being posed due to the landslide movements. As many of the geological and geotechnical features of the slope were not well known in advance, the stabilization measures were designed to be flexible for changes or integrations to be made during construction by taking into account the progressive refinement of the geological–geotechnical slope model being developed.

The limit equilibrium method (LEM) and finite element method (FEM) were applied to investigate the slope failure mechanism and to test the effectiveness of different slope stabilization schemes. The final stabilization works were carried out with a step by step procedure and were designed in order to make the implementation of further refinement and adaptations possible.

Landslide description

During the 2009 spring season, following a period of prolonged rainfalls, a debris slide with an approximate volume of 35,000 m^3 occurred in the Valgrisenche valley, along the SR25 road (Fig. 1). The landslide was anticipated by a series of minor rock falls from the rocky cliffs above the road occurred during March and April. The failure extended from 1,220 to 1,310 m a.s.l. with a total length of 120 m and a maximum width of 80 m and was located in a thick layer of colluvial and morainic deposits which are irregularly present along the valley (Fig. 2a, b).

These Quaternary coarse grained deposits (boulders, blocks, pebbles, and gravels in silty/sandy matrix) lie unconformably and discontinuously on the bedrock (Ruitor Unit) which consists of pre-Permian (450–480 Ma, Guillot et al. 2002) garnet micaschists and paragneisses with abundant interbedded metabasites (Bucher et al. 2004). The crown zone was characterized by the absence of a clear scarp. The transition between the displaced mass and the stable slope was marked by a network of sub-parallel tension cracks.

The overall slope angle before the landslide occurrence was between 35° and 40°. Although the early evolution stages of the instability were not known, after the initial rock falls, the colluvial and talus deposits started to mobilize along the slope and gradually overstepped the counterscarp wall alongside the SR25 road (Fig. 2c). The road was progressively overrun by soil and debris, leading the local authorities to close it to traffic. The counterscarp wall alongside the road resisted to the slide, avoiding the propagation of the failure surface below the road level. The downslope propagation of the instability was further inhibited by the presence of an anchored wall which sustained the road (Fig. 2d).

During April 2009, the mean cumulated precipitation in the Aosta valley reached 212 mm, with a major rainfall event occurred between April 26th and April 28th, with 45 % of the 50 years mean

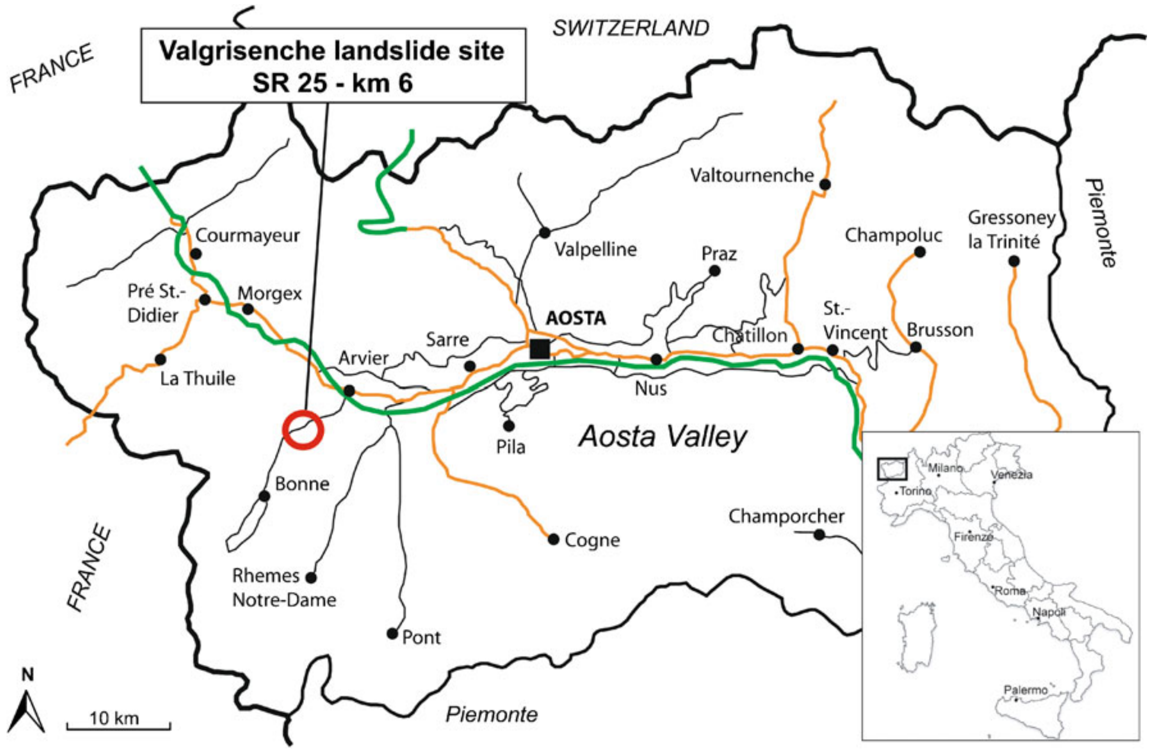

Fig. 1 Valgrisenche landslide site

Fig. 2 **a** General view of the landslide (May 2009) from the opposite valley side. **b** View of the landslide (May 2009) from the crown area. **c** Detail of the counterscarp wall partially overstepped by the landslide mass. **d** Details of the SR25 roadway and anchored wall

monthly precipitation being recorded (Regione Autonoma Valle d'Aosta 2009a). The saturated soil conditions were enhanced by the rapid spring snowmelt that took place during March. It is worth to note that the 2008–2009 winter season was characterized by very intense snowfalls across the whole Western Italian Alps and in particular along the Valgrisenche valley, where a cumulated snow height of 590 cm was recorded (Regione Autonoma Valle d'Aosta 2009b).

The stabilization works, required as a matter of urgency, started in May 2009, with the road kept closed to traffic up to the beginning of June. At first, a change of the slope geometry was carried out through the demolition of some critical unstable boulders and the creation of a system of ramps and berms along the slope. To allow the road to be partially opened to traffic, a provisional embankment acting as a barrier, wire-rope rock catch fences, and a rock catch ditch were built just above the retaining wall alongside the road. However, following a severe rainfall which occurred at the end of May 2009, a further rotational slope failure took place on the lower berm.

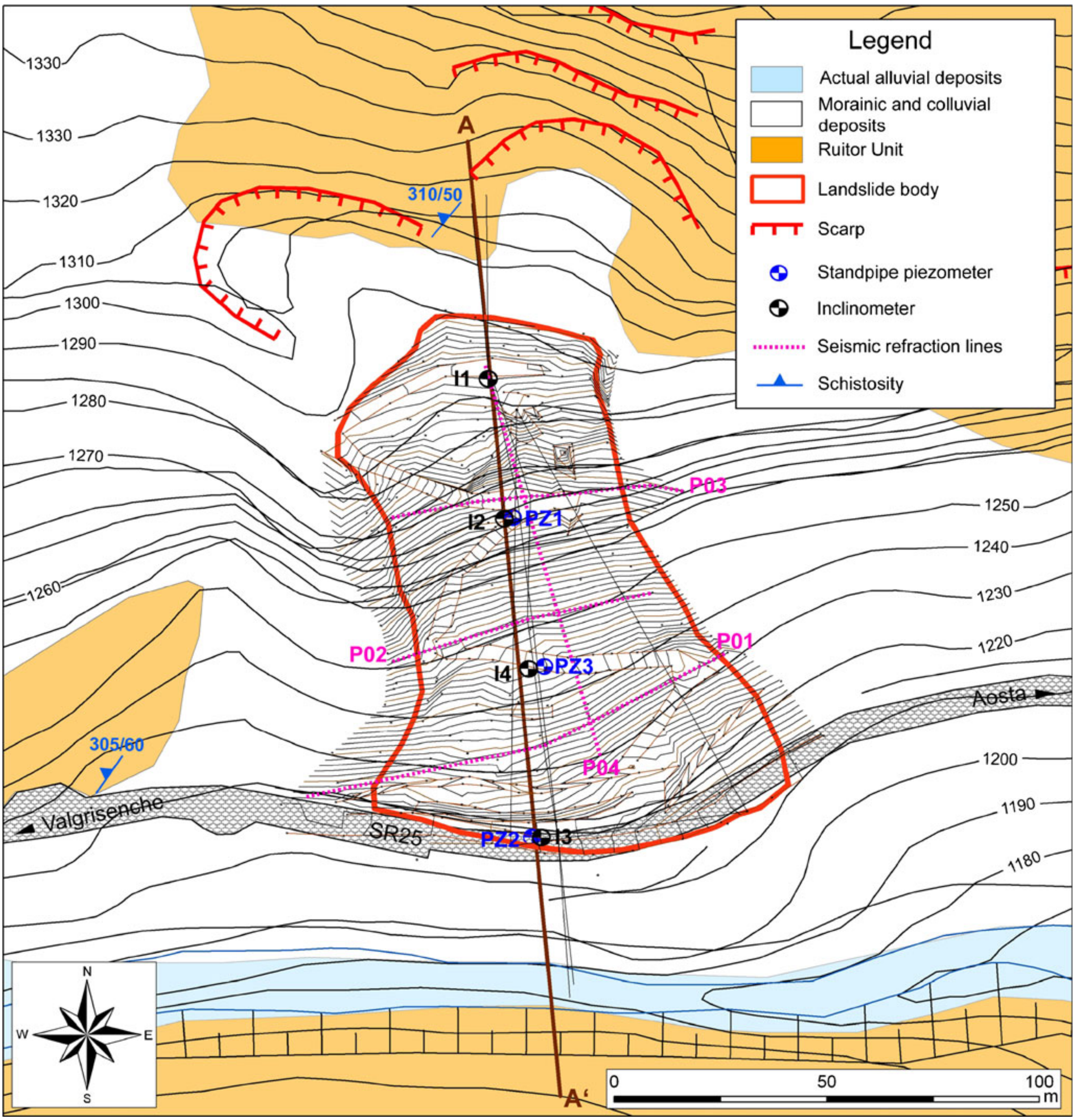

Fig. 3 Geological map showing the in situ investigations and monitoring system. Topographic base extracted from Regional Technical Map 1:10,000, integrated with a detailed landslide topographic survey

In situ investigations and monitoring data

In situ investigations were carried out in order to define the geological and geotechnical conditions along the slope. Monitoring of surface movements, displacements at depth, and groundwater levels was also performed.

Direct and indirect in situ investigations

As shown in Fig. 3, the direct investigations consisted of four boreholes, continuously cored to a maximum depth of 46 m and equipped with four inclinometer tubes (I1, I2, I3, and I4), together with three standpipe piezometers (PZ1, PZ2, and PZ3). Indirect investigations comprised a seismic refraction survey along four alignments (total length of 480 m).

The boreholes drilling showed the presence of a thick colluvium, consisting of a poorly sorted mixture of matrix-supported angular boulders, blocks, cobbles, and gravel in a fine-grained matrix material (silt and sand). This layer can be referred to undifferentiated Holocenic colluvial and morainic deposits which lie on the gneissic bedrock of the Ruitor Unit. The thickness increased from the landslide crown (16.5 m in the S1 borehole) to its middle sectors (30 m in the S2 borehole), decreasing again toward the landslide toe (23 m in the S4 borehole).

The geophysical investigations allowed the splitting of the colluvial and morainic layer into three units characterized by a progressive increase of the shear wave velocity versus depth as illustrated in Fig. 4. Moreover the seismic profiles highlighted that the contact between the bedrock and the deposits is characterized by a pronounced difference of the seismic wave velocities. On the eastern landslide flank, the contact between the two units rises toward the ground surface and the thickness of the colluvial and morainic deposits decreases.

Displacement monitoring data

Following the first signs of slope instability, during April 2009, slope movements were monitored on eight benchmarks by weekly GPS surveys (Fig. 5). Three benchmarks (GPS3, GPS4, and GPS6) were located in the basal slope sector, close to the counterscarp road walls, three (GPS1, GPS2, and GPS5) in the middle slope sector and two (GPS7 and GPS8) on the landslide crown zone. The monitoring data (Fig. 5) showed that the larger downslope movements occurred in the middle slope sector with a maximum cumulated displacement of 3.91 m being recorded on benchmarks GPS2 after 32 days. The slope movements progressively decreased toward both the eastern landslide flank, where the GPS6 benchmark recorded a cumulated displacement of 1.26 m in 32 days and to a larger extent toward the landslide crown where the GPS7 and GPS8 benchmarks showed a cumulated displacement of 0.25 and 0.23 m, respectively.

This is in good agreement with the development of a transition zone between the stable ground and the landslide mass along the crown zone without the generation of a clear scarp. The rate of displacement recorded during the 32 monitoring days showed acceleration and deceleration phases connected to rainfall periods. In particular the most important acceleration phase was recorded between April 24th and April 30th during an intense rainfall event (up to 100 mm in 2 days). All GPS benchmarks showed a clear displacement acceleration,

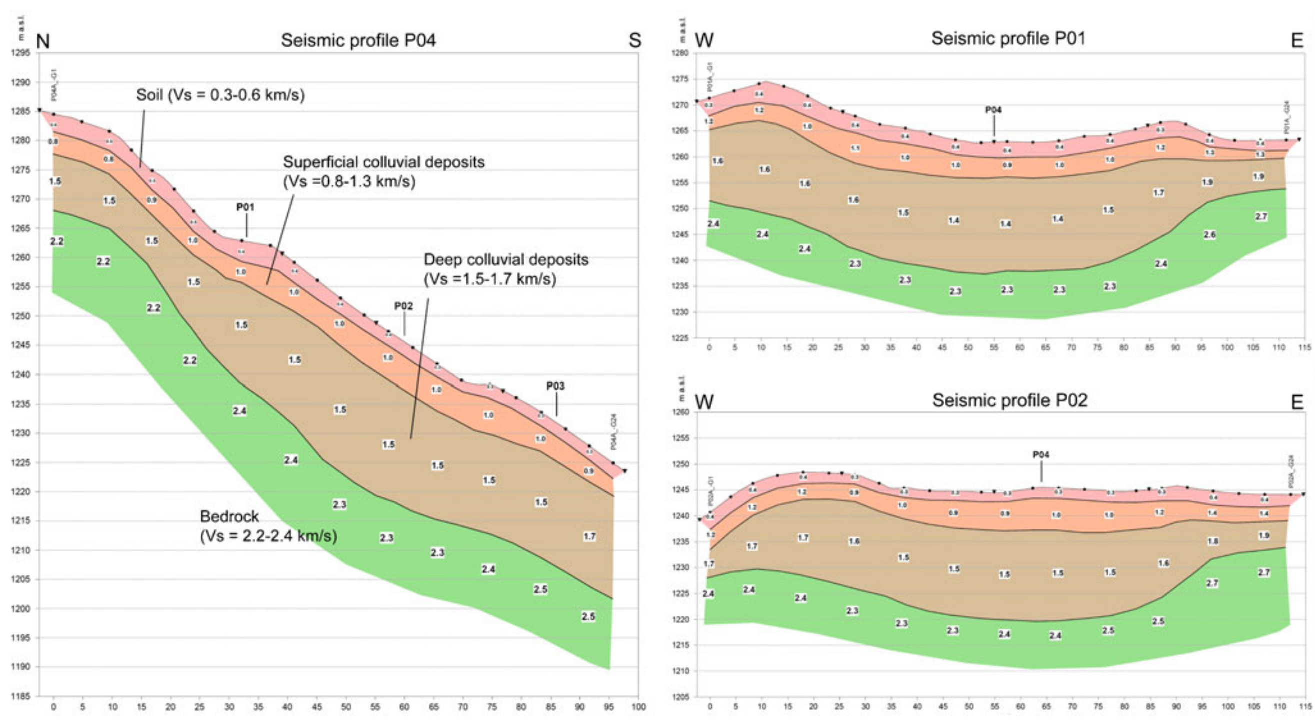

Fig. 4 Results of the seismic refraction survey on the landslide

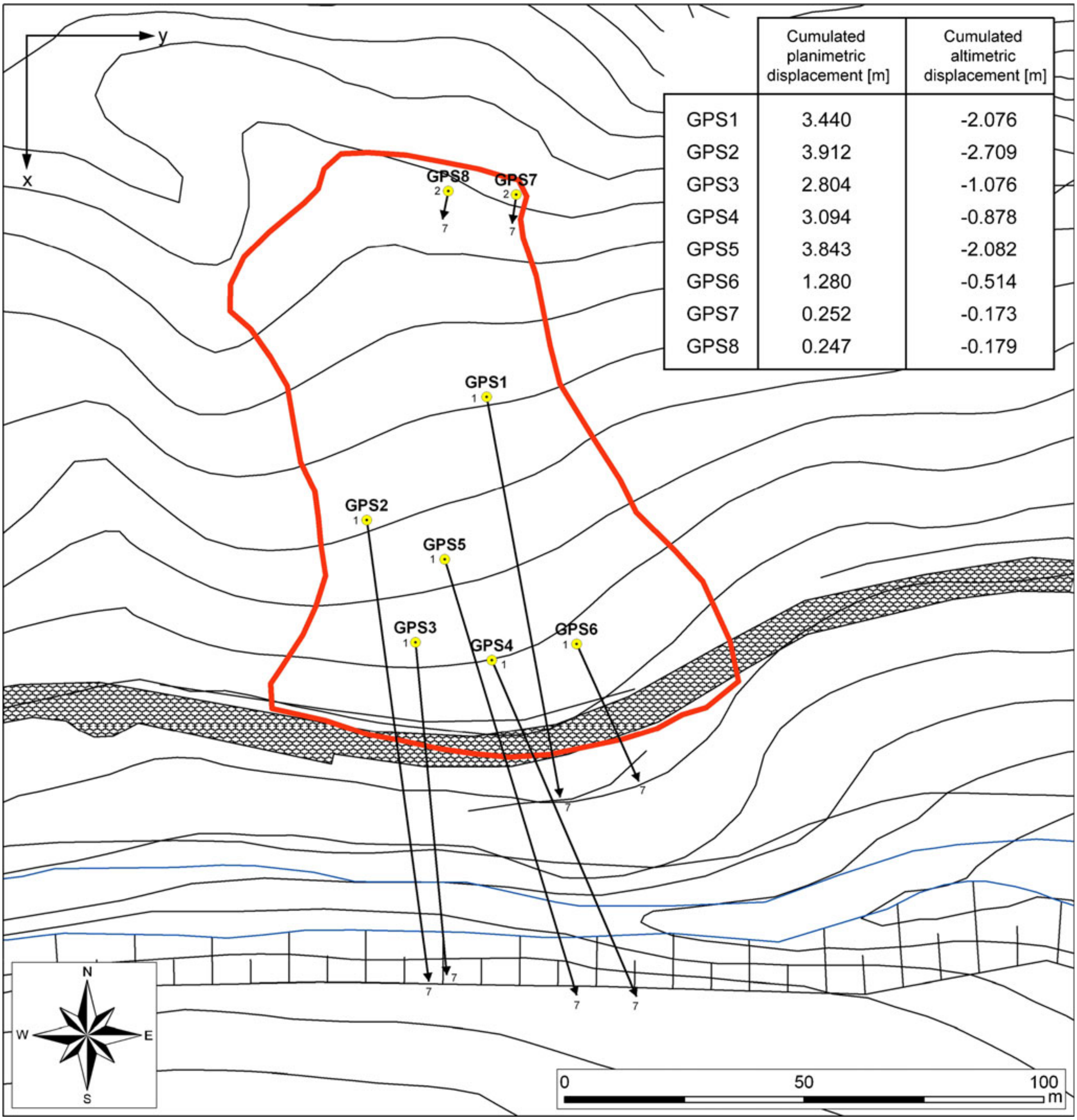

Fig. 5 Results of GPS monitoring. The vectors show the cumulated planimetric displacement measured on the benchmarks. The *numbers* indicate the date of GPS measurement (*1* 02/04/2009, *2* 06/04/2009, *7* 04/05/2009)

with a maximum velocity measured on the GPS2 and GPS5 benchmarks equal to 210 mm/day. From the beginning of May 2009 the GPS monitoring was interrupted.

With the boreholes drilling activities completed, as already mentioned above, four inclinometer tubes (I1, I2, I3, and I4) were installed respectively in the upper, medium and basal landslide sectors. Since the boreholes were completed in different times, spanning from the end of May (I1, I2, and I3) to July 2009 (I4), the reference measurements of each inclinometer vary accordingly. Three inclinometers highlighted a well-defined sliding surface at −16.0 (I1), −8.0 (I2), and −4.0 m (I4) below ground level as depicted in Fig. 6. The sliding surface is shown to progressively decrease in depth from the landslide crown to the landslide toe and it remains confined above the counterscarp retaining wall (the I3 inclinometer did not show any movement). This was confirmed by field observations and by the absence of damages in the anchored wall below the roadway.

The cumulated displacements (20–80 mm) recorded at the head of the three inclinometer holes I1, I2, and I4 from June to

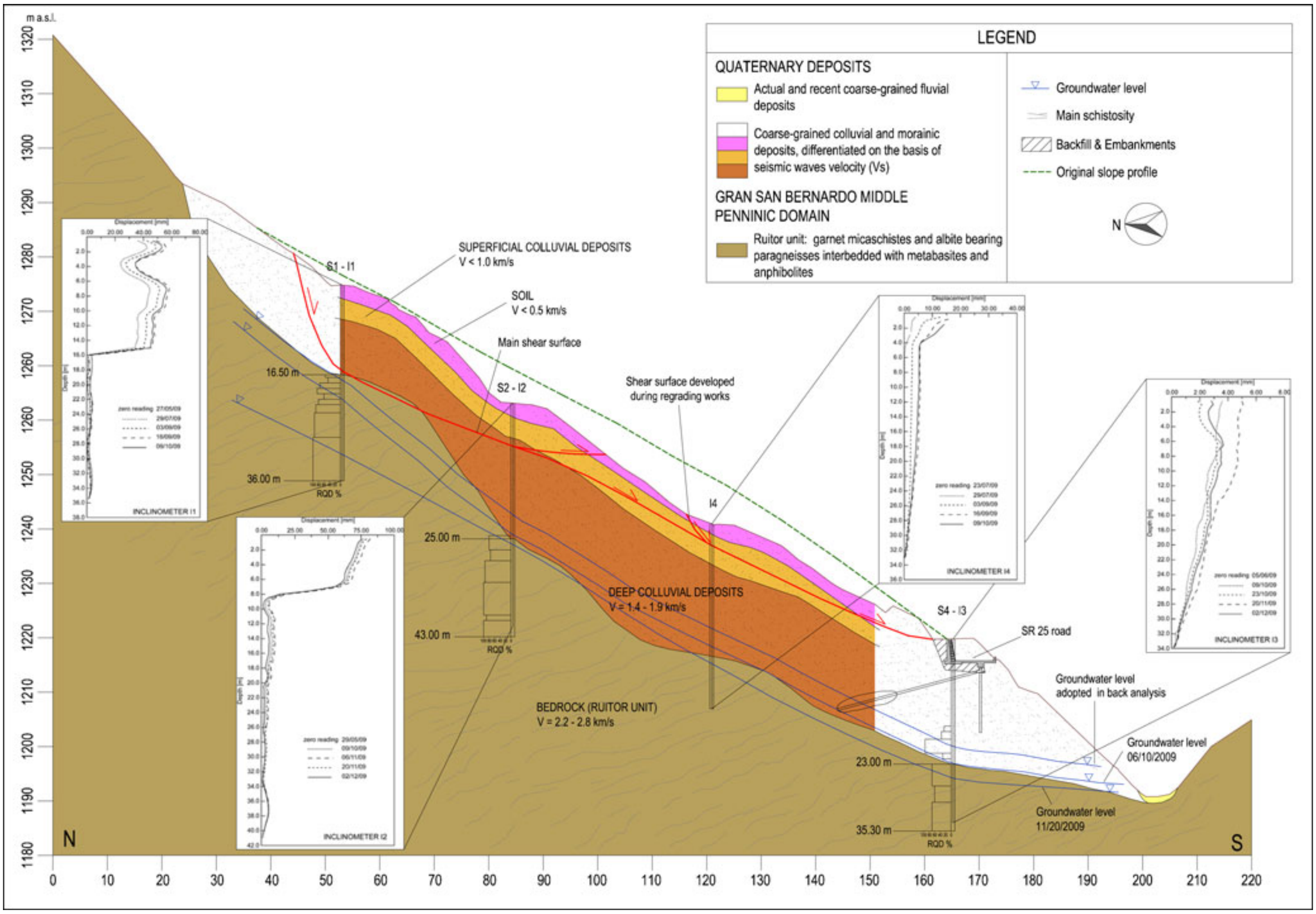

Fig. 6 Geological–geotechnical model of the landslide (cross section)

November 2009 were sensibly smaller than the surface displacements (up to 3.9 m) measured along the slope in April by the GPS monitoring. These differences can be explained with the changes in the slope geometry during May 2009 (before the installation of the inclinometer tubes), due to the removal of a large amount of unstable material, the construction of a provisional embankment along the road, and the progressive reduction of rainfalls. Hence, starting on June 2009, the slope experienced a progressive stabilization with a decrease of displacement both at the surface and at depth.

It has been inferred that the main groundwater flow is in the fractured rock mass of the Ruitor Unit. Following heavy rainfall (especially in the spring season), the groundwater level could undergo a sudden rise up of several meters, thus saturating the debris cover. This probably started in the middle slope portion, also through a direct seepage along the contact between the rock mass and the debris, thus triggering the instability.

Groundwater data

The groundwater levels were monitored from the end of May to December 2009 in the piezometer holes PZ1, PZ2, and PZ3. Additionally, data on the groundwater level have also been obtained with measurements in the inclinometer tubes I1, I2, I3, and I4 (Fig.7). The monitored groundwater levels in PZ3 were recognised not to be reliable and therefore were excluded from the subsequent analysis. After the first month of monitoring (June 2009), a general decrease of the groundwater level was recorded in all the boreholes.

The piezometer level tended to stabilize at a depth between −25 and −26 m, along the contact between the bedrock and the detritic cover. Only the groundwater level in I1 showed an extended lowering phase with minor raising peaks related to rainfall events (Fig. 7). The general groundwater lowering was shown to be related to the reduced amount of precipitation and the snow melting completion during the summer.

Back analyses

Back analyses were carried out with the main objective to determine the shear strength parameters mobilized along the sliding surfaces identified in the field. The advantages of the combined use of LEM and FEM lie in coupling the simplicity of LEM with the completeness related to slope geometry, ground non-linear behavior, in situ stresses that can be considered with FEM.

Limit equilibrium method (LEM)

The LEM method was applied by using the Slide computer code (Rocscience 2009a). A first analysis was performed in order to determine the shear strength parameters of the superficial colluvial layer considering the rotational slide occurred in the lower berm (Fig. 8). The geometry of the sliding surface

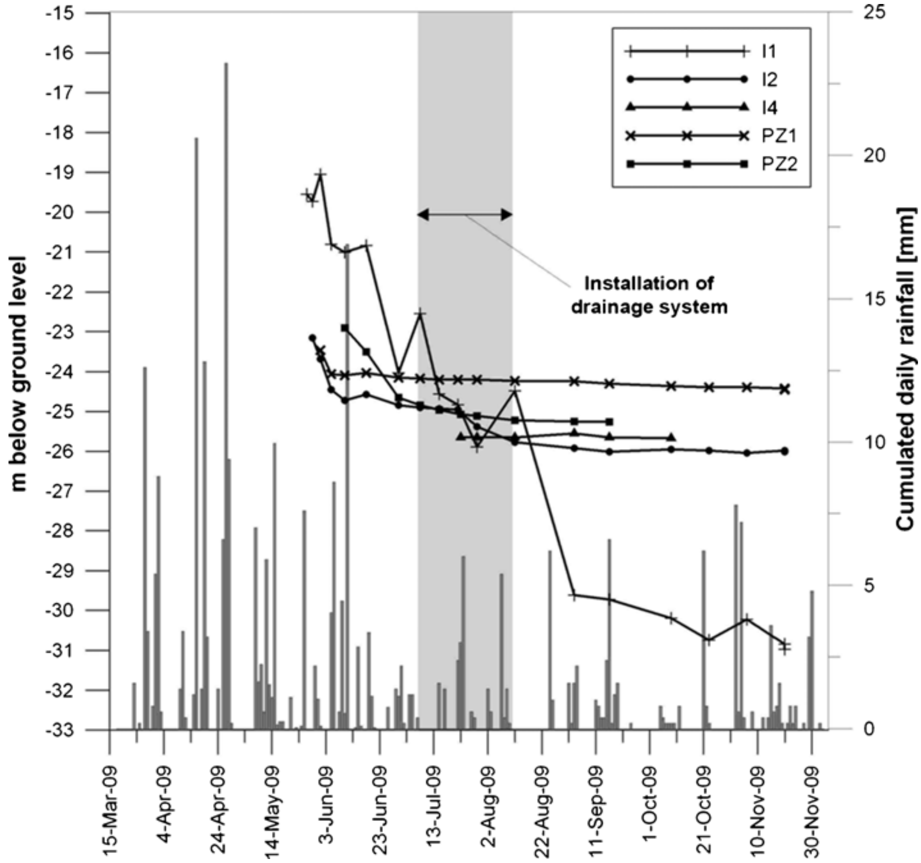

Fig. 7 Groundwater monitoring results and cumulated daily rainfall. *I1*, *I2*, and *I4* indicate water level measured in inclinometric tubes while *PZ1* and *PZ2* indicate water level measured in standpipe piezometers. The *gray box* indicates the time-span for the installation of the drainage system on the slope

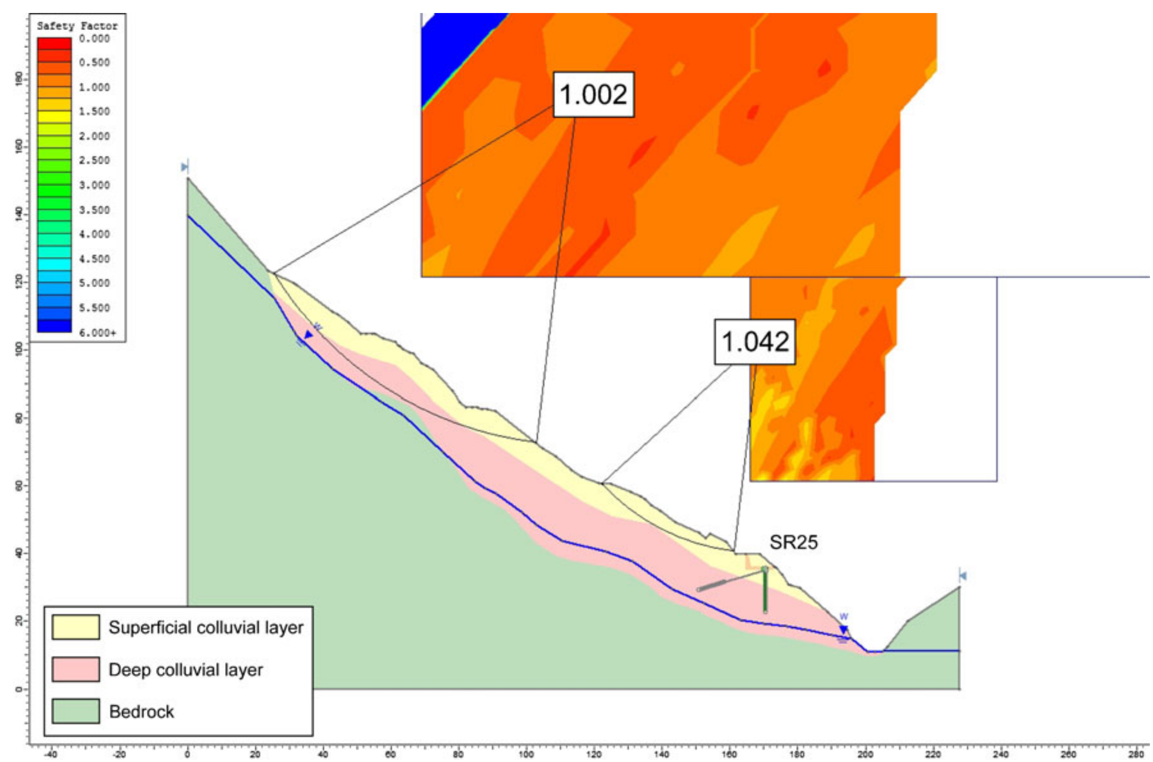

Fig. 8 LEM back analysis results. The *colored windows* in the image show the variation of the factor of safety for different potential slip surfaces

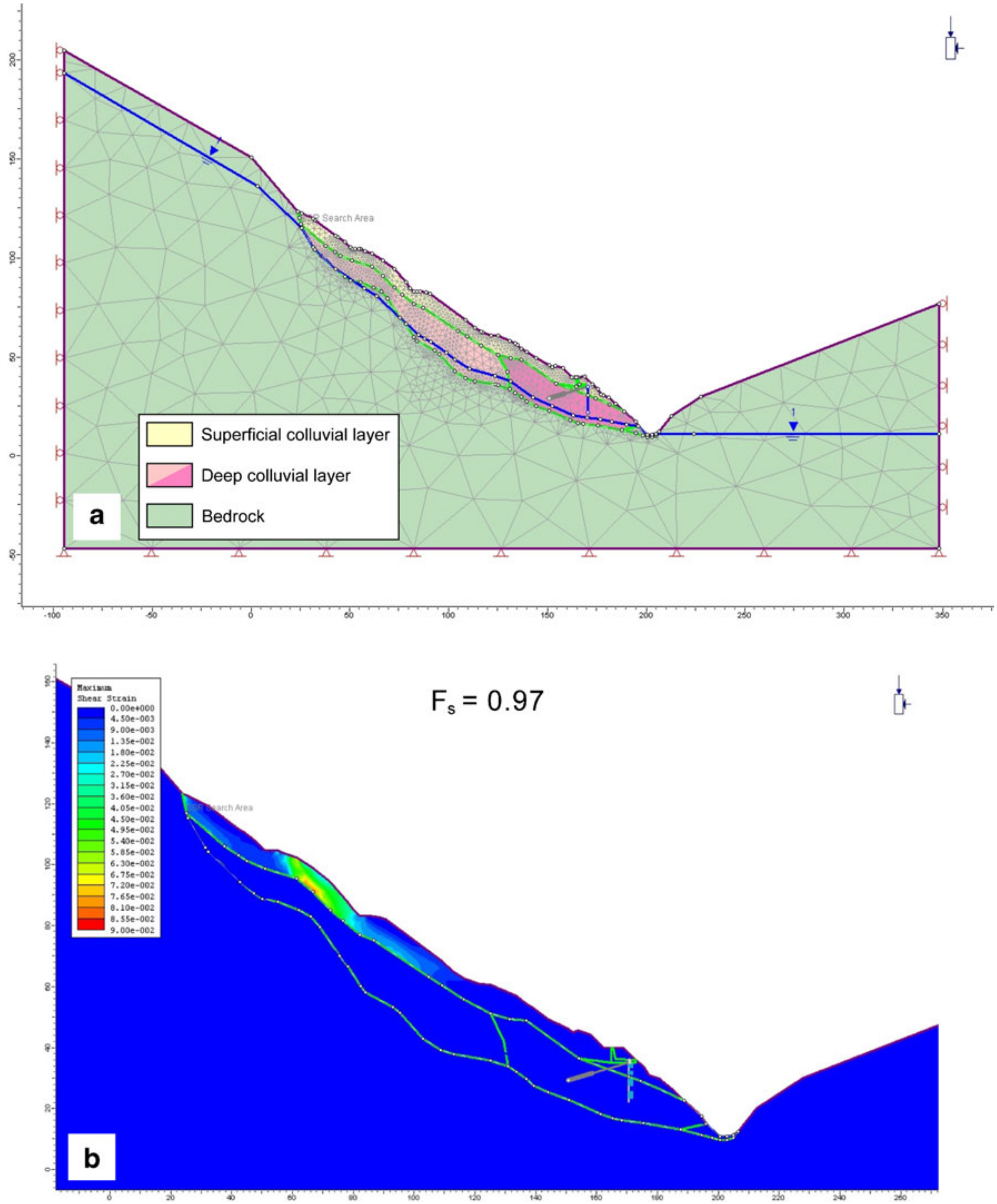

Fig. 9 FEM model (**a**) and back analysis results in terms of maximum shear strain distribution (**b**)

was chosen on the basis of field observations and the results of the inclinometer measurements.

The groundwater level was increased by 4 meters to take into account the effect of the rainfalls on the landslide body. Zero cohesion was assumed, which appears to be a suitable condition for the analysis of reactivated landslides and pre-existing sliding surfaces (Hutchinson 1969), while the friction angle along the same sliding surface was computed to be equal to 27°.

A further analysis was carried out on the rotational slide identified with the inclinometer measurements in the upper sector of the slope. This sliding surface involves the deeper portion of the colluvial layer (Fig. 8). By repeating the analysis, the friction angle was

Table 1 Deformability parameters adopted in the numerical analysis

Geotechnical units	*E* [MPa]	ν
Colluvial deposit (superficial)	500	0.3
Colluvial deposit (deep)	1,400	0.3
Bedrock	5,500	0.3

Table 2 Strength parameters of the materials adopted in the numerical analysis

Geotechnical units	c′ [MPa]	ϕ′ [°]
Superficial colluvial deposit	10	27
Deep colluvial deposit	50	31
Superficial colluvial deposit below road level	50	27
Deep colluvial deposit below road level	100	31
	UCS [MPa]	GSI
Bedrock (Ruitor Unit)	75	70

computed to be equal to 31°, using the previously determined parameters for the superficial colluvial deposits.

Finite element method (FEM)

A FEM model of the slope was created by using the Phase2 computer code (Rocscience 2009b), which allows the slope factor of safety to be computed based on the Shear Strength Reduction Method (SSRM) and the Mohr-Coulomb failure criterion (Zienkiewicz et al. 1975). The boundary conditions of the model and the FEM mesh adopted are shown in Fig. 9a. Gravity loading and plane strain conditions (with the in situ stress ratio k=0.43 being assumed) were applied.

The deformability parameters of the colluvial deposits and the bedrock were determined on the basis of the seismic investigation results (Table 1) by using the correlation equations due to Barton (1996). The Hoek–Brown and Mohr–Coulomb failure criteria were adopted for the bedrock and for the colluvial deposits, respectively. In both cases, an elastic perfectly plastic non associated constitutive law was assumed.

Preliminary analyses showed that the shear strength parameters determined with LEM generated the instability along the sliding surfaces previously identified. These analyses also showed the development of a yield zone at the slope toe below the road level which was not in line with the displacement pattern observed in the field. In order to overcome this problem, the same parameters were changed by increasing the cohesion c′ and keeping the friction angle ϕ′ constant as shown in Table 2. The results of the updated numerical analyses in terms of the maximum shear strain are illustrated in Fig. 9b. According to the FEM results, the factor of safety of the slope computed through the SSRM is equal to 0.97.

Validation of the landslide model

In order to validate the geological–geotechnical landslide model used in the previous analyses, a further LEM analysis of the slope was performed. With reference to the original slope profile obtained through a Digital Elevation Model (DEM) of the area with a 10×10 cell size, the shear strength parameters used for the bedrock and the colluvial deposits are listed in Table 2.

The results obtained (Fig. 10a) show that with low groundwater level (stabilized along the contact between the bedrock and the colluvial deposits), the most critical sliding surface (Fs=1.01) is extended from the road retaining wall to the middle-upper portion of the slope and is entirely inside the superficial colluvial deposits. Further surfaces with similar geometry and safety factor ranging between 1.01 and 1.20 developed in the same slope sectors.

The groundwater level was subsequently increased in order to simulate the most likely conditions after the spring snow melting and the prolonged rainfalls. As shown in Fig. 10b, a complete saturation of the deep colluvial deposits was assumed, while the groundwater level in the upper slope sector resulted from the observation of direct seepage from the rock mass towards the colluvium. The available piezometer data were of limited value since they recorded the groundwater levels only a month past the landslide occurrence.

The analysis shows the onset of the instability to take place along a circular surface (Fs<1) extending from the road retaining wall to the middle-upper portion of the slope. It is of interest to note that the most critical surface with a safety factor equal to 0.91 is similar to the main sliding surface of

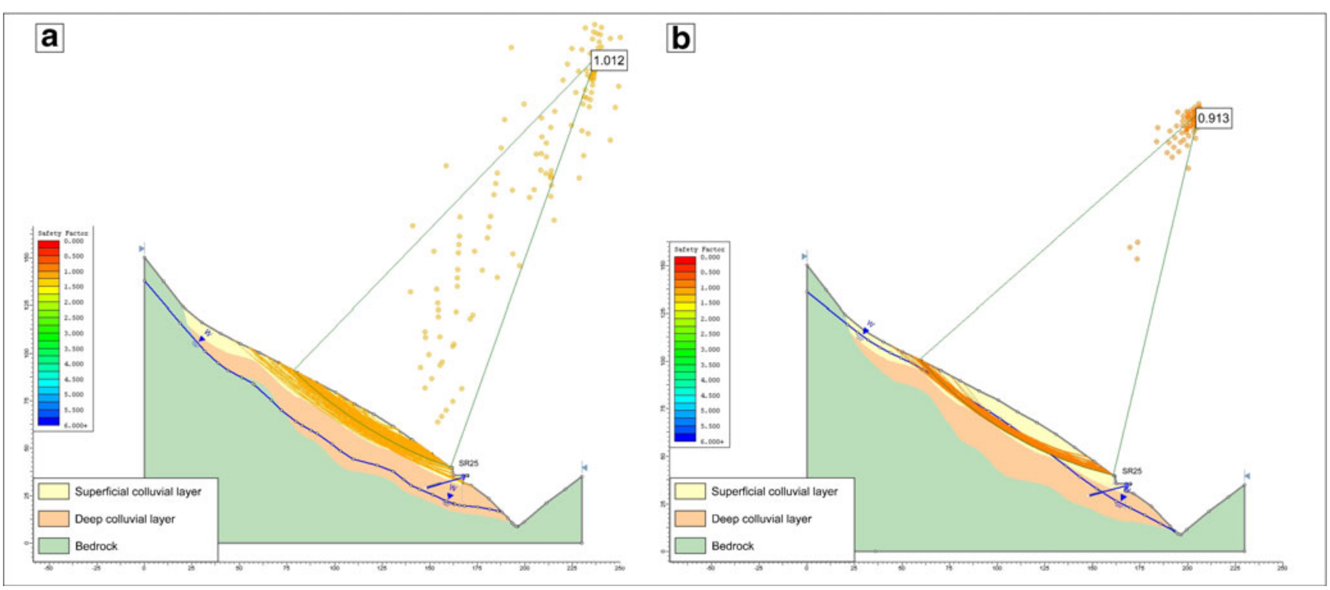

Fig. 10 Results of the LEM analysis considering the original slope geometry. a Low measured groundwater level, surfaces with Fs lower than 1.2. b Highest inferred groundwater level, surfaces with Fs lower than 1.0

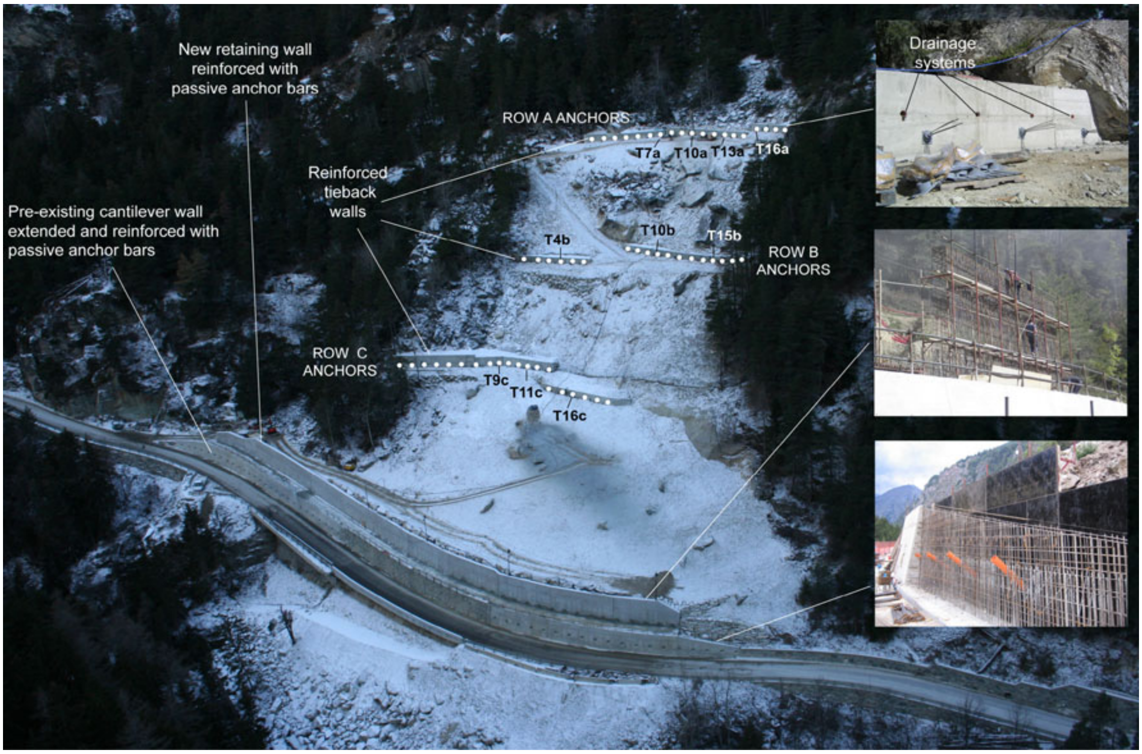

Fig. 11 Slope in November 2009, with the stabilization works completed

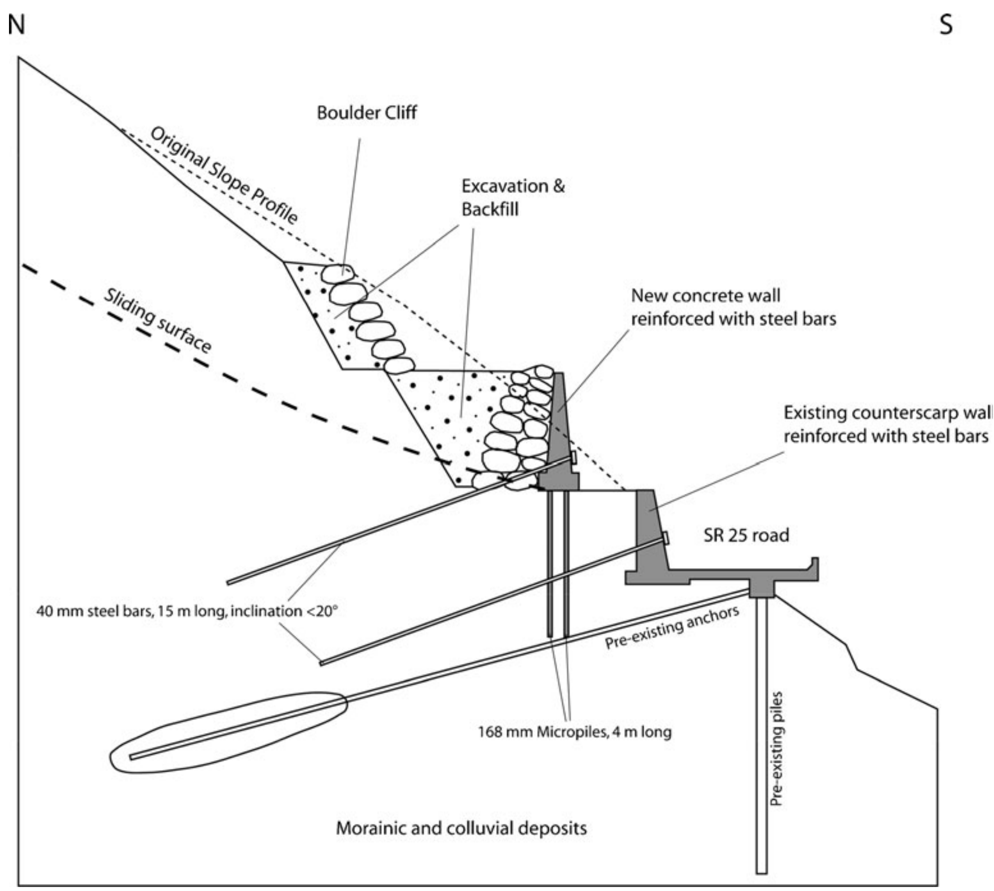

Fig. 12 Schematic cross section of the double wall solution adopted along the SR25 road

the landslide. The maximum thickness of the mobilized material (about 10 m) agrees with field observations proving the validity of the geological-geotechnical model developed.

Stabilization works and design analyses

The design and implementation of the stabilization works were significantly dependent on the priority posed by the local authorities to open as soon as possible the SR25 road with an acceptable level of safety. With these constraints being posed, the stabilization works were defined step by step, with a work in progress procedure and intentionally subjected to possible refinements and adaptations. In all cases the final objective was to make the area subjected to the landslide stable in the long term.

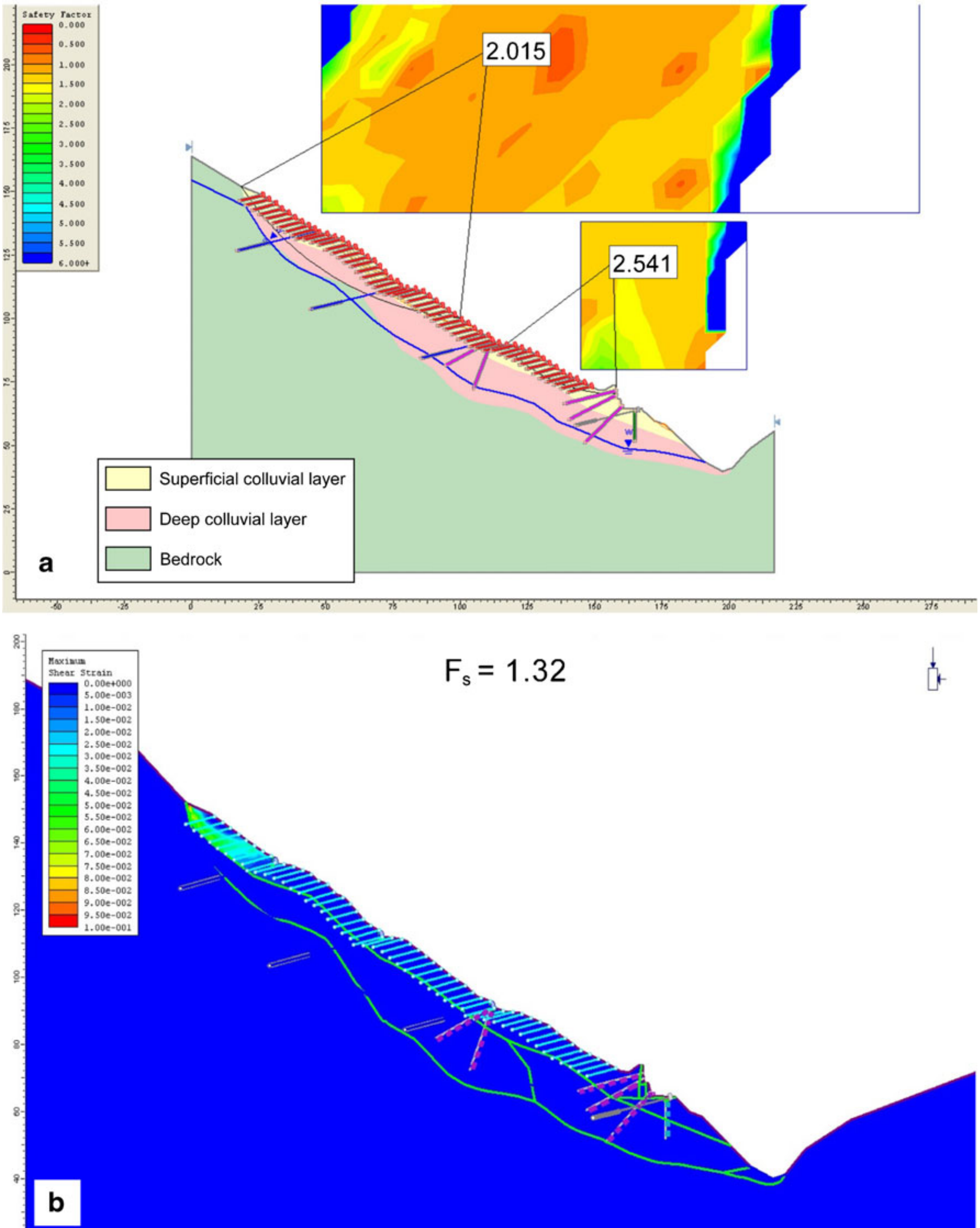

Fig. 13 Design analysis results: LEM (**a**) and FEM (**b**) stability analyses

The following main stabilization works were carried out (Fig. 11):

- Installation of a suitable drainage system consisting of sub-horizontal drainage pipes, with 2 m spacing, installed along the three berms created during the regrading works. The length of the pipes was chosen to reach the contact between the bedrock and the detritic cover (20–30 m);
- Construction of a double wall acting as a passive retaining structure along the road. The pre-existing counterscarp retaining wall was repaired, extended, and reinforced with a set of 40 mm diameter passive steel bars, 2 m spaced and 15 m long. A new micropile-founded retaining wall reinforced with 40 mm diameter, 15-m-long passive steel bars was built. The schematic cross-section of the double wall solution is shown in Fig. 12. Although simple in concept, the retaining wall was logistically complex and construction phases had to be sequenced with considerable care and attention to details;
- Installation of 56 four-strand anchors with double corrosion protection along three rows (A, B, and C in Fig. 11). The anchors were up to 35 m long, inclined 15° to 20° downwards, with a 2.5-m spacing. The anchors had a working capacity of 600 kN. The final lock-off loads for the anchors were 200 kN in rows A and C and 230 kN in Row B. Along each row, the anchor heads were tied to a reinforced concrete walls. The slope geometry and the presence of boulders forced to adopt discontinuous wall stretches in spite of a single continuous wall for each row.

The design of the stabilization works was carried out by making effective use of both the LEM and FEM methods as a follow-up of the back analysis studies described above. The slope geometry was modified according to the final profile. The factors of safety along the sliding surfaces were again computed and the overall stability conditions of the slope significantly improved. The factors of safety increased from Fs=1.00 and 1.04 to Fs=2.01 and 2.54, respectively, according to the LEM calculations for the two sliding surfaces considered. These stability analyses showed however the presence of some superficial sliding surfaces.

Therefore additional stabilization measures were considered:

- Installation of fully grouted, 20 mm diameter rock dowels, 9 m long, with a 3×3 m spacing pattern;
- Installation of an additional set of 40 mm diameter passive steel bars, 2 m spaced, 15 m long along the row C anchors;
- Installation of rock catch fences and rock fall net system (not considered in the stability analyses).

The improved stability conditions achieved with the LEM approach are shown in Fig. 13a and the corresponding FEM results are depicted in Fig. 13b. With the additional stabilization measures, FEM analyses indicate a factor of safety equal to 1.32, without the presence of the superficial sliding surfaces. In addition, it is interesting to note that the downslope displacements were reduced (one order of magnitude) and the loads acting on the retaining/reinforcement structures decreased significantly.

Performance monitoring following stabilization works

With the most important stabilization works completed as shown in Fig. 11, which illustrates the slope in November 2009, the traffic along the SR25 road could take place with no further restrictions. Ten of the ground anchors were equipped with load cells housed within the anchor heads. Load cells readings have been collected at irregular intervals after the completion of the stabilization works. The monitoring results are of interest in order to check the slope response. The results of monitoring, updated to March 2012, are shown in Fig. 14. It is noted that, following a load decrease, the anchors exhibit different patterns of behavior.

The anchors of the western end (T7A) exhibit a load steady value or a slight decrease while the central and eastern anchors display a substantial load increase of nearly 135 kN between November 2009 and March 2012. Also the central row B shows a load increase, which is more pronounced for the anchors of the eastern sector (T10B, T15B). The load pattern for the row C anchors is indeed different from the previous ones: after a slight relaxation, the anchors show steady load values (T16C) or a very slight load increase (T9C, T11C). It is of interest to note that the most significant load increase for all the ground anchors takes place during the spring (March–June).

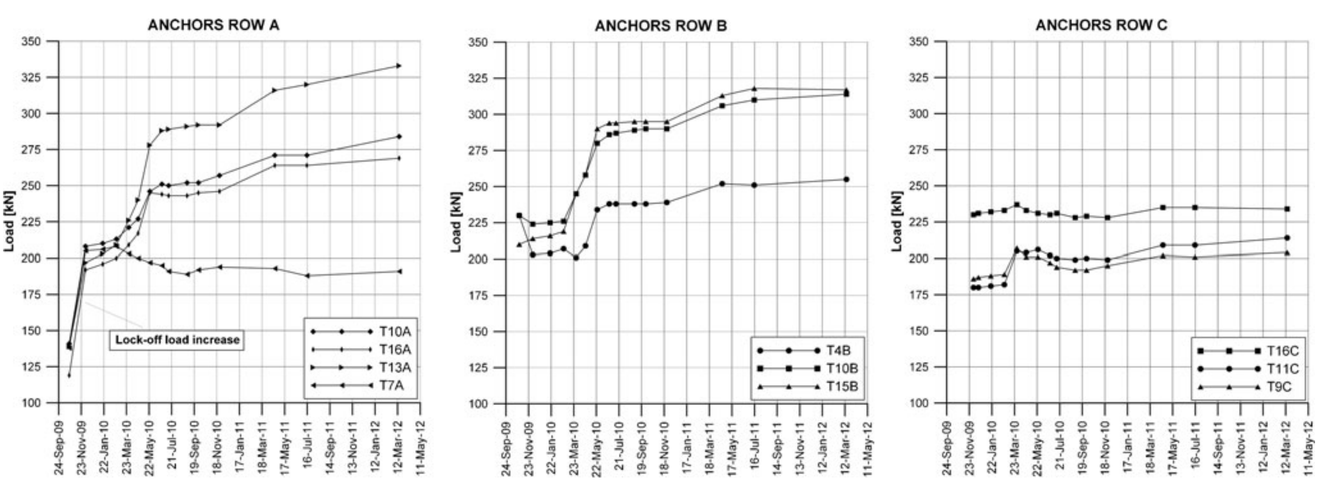

Fig. 14 Plots of load cell monitoring results

Groundwater monitoring could take place until the end of 2009, giving only limited information about the water regime within the slope following the stabilization works. Similarly no inclinometer data were available. It is noteworthy that following the installation of the sub-horizontal drains in the slope, a consistent piezometer lowering (6 m) was observed in the upper slope sector (I1 inclinometer tube, Fig. 7).

Concluding remarks

Based on the in situ investigations performed and the monitoring data that could be obtained concurrent with the stabilization works that needed to be engineered and implemented, the case study of a debris slide (estimated volume of about 35,000 m^3), occurred in April 2009 in the North Western Italian Alps (Aosta valley) along the Valgrisenche valley, has been described.

Simplified methods, such as the LEM and the FEM, have been used for the purpose of back analysis of the landslide behavior in order to understand its triggering mechanism and provide the ground shear strength parameters needed for design. The same methods were adopted for performing design analyses of the stabilization system selected in order to lead the slope movement to stability.

The interest of the work presented stems from the special emergency conditions that accompanied the investigations performed and the monitoring data being collected at the same time, as cost-effective stabilization measures were chosen in the framework of accepted design analysis methods which could allow one to demonstrate to the public authorities involved the reliability of the choices being made.

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