Risk Management of Rockfall/Rockslide Hazard Based on Remote Sensing Techniques: The Example of Arvel Quarry (Switzerland)

A. Pedrazzini Institut de géomatique et d’analyse du risque, University de Lausanne, Switzerland  
M. Jaboyedoff Institut de géomatique et d’analyse du risque, University of Lausanne, Switzerland  
T. Oppikofer Geological survey of Norway, Trondheim, Norway  
R. Chantry CSD Ingénieurs-conseils, Lausanne, Switzerland  
E. Stampfli CSD Ingénieurs-conseils, Lausanne, Switzerland

Abstract

The Arvel quarry is located at the end of the Rhône valley (SW Switzerland) and is one of the most important producers of ballast and other construction materials in Switzerland. On the 12th December 2008, a rockslide of about 20'000 m³ occurred in the upper part of the quarry inducing the partial interruption of mining operations. After this event, the stability of the entire quarry was analysed coupling classical field approaches, aerial and terrestrial laser scanner (ALS and TLS) analysis. A monitoring system was installed based on continuous Ground-Based Radar and periodic TLS measurements. After fourteen months of movement’s observation, early warning systems as well as a new slope design concept were proposed to reach tolerable risk level allowing the partial reopening of the mining activities.

1 Introduction

The Arvel quarry is located in SW part of Switzerland close to the eastern side of the Geneva Lake (Fig. 1). The rock forming the slope above Villeneuve has been extensively exploited as construction material since the Middle Ages. The site of the actual quarry has been industrially exploited since 1905. The main rock exploited in the quarry is a siliceous limestone that represents a good material to produce ballast for railway construction. The annual production is about 500,000 tonnes. The site’s configuration, especially the steep and rough topography, represents a real challenge in quarry design. The Arvel quarry has experienced several rockslides and rockfall events during the last century. The most important one occurred on March 13th 1922, when a rockslide of about 600,000 m³ destroyed all the installation located below the cliff. This rockslide also created an important deformation of the alluvial sediments on the Rhone plain (Choffat, 1929; Jaboyedoff 2003; Crosta et al., 2009). More recently, on December 12th 2008, a rockslide of about 20'000 m³ occurred on the upper part of the quarry. The presence of a potential unstable rock spur in the same area, presenting the same characteristics, induces a temporary closure of the quarry activity.

In this paper, we illustrate the application of remote sensing techniques, in particular Terrestrial Laser Scanner (TLS) for slope monitoring but also as an important base to produce a reliable geotechnical model. In the first part, we analyse the main structural and lithological characteristics of the study area in order to define homogenous structural domains and the related failure mechanisms. Then, we present a detailed characterization of the rockslide of December 2008 and the potential unstable spur that is still present at the top of the quarry. In the last part, we describe the preliminary results of the monitoring system that has been installed and the design of the early warning system. Based on the new available information, a risk management procedure has been proposed to permit the partial reopening of the quarry.
2 Geological setting

The Arvel quarry is located on the Préalpes Medianes Nappe. The rocks forming Les Monts d’Arvel belong to the normal limb of a kilometric-scale anticline (Tinière Anticline). The lower part of the slope is formed of an intercalation of weak marls and crinoid-rich limestone layers. The upper portion of the quarry representing from an economic point of view, the most interesting lithology, is composed by a regular intercalation of marls layers and thick (1-1.5m) fine-grained siliceous limestone layers. Bedding planes plunge 40°-50° into the slope. Two series of major tectonic faults (NNW-SSE and NE-SW) cross the entire area and show clear slickensided surfaces and a localized degradation of the rock mass quality. The hydrogeological system is complex involving both karstic and fracture permeability. Superficial drainage and seepage between rainfall periods are almost absent.

Figure 1. Overview and general setting of the Arvel quarry. A) Geographical location B) View from the bottom of the main quarry excavation. Numbers on the picture correspond to the 2008 rockslide scar (1a) and its related deposits (1b) and the main potential unstable area (2). C) Rock mass characteristics in the upper portion of the quarry, showing regular intercalation of marls and siliceous limestone. D) View of upper bench face showing the important rock bolting necessary to stabilize the cliff (black arrows).
3 Structural analysis and rock mass characteristics

3.1 Characterization of discontinuity sets

In this study, we combine a classical method of geological field survey, (manual compass measurement), with a semi-automated extraction of discontinuity sets by means of an aerial High Resolution DEM (HRDEM) and locally completed by TLS data. Structural analyses were performed using the COLTOP 3D software (Jaboyedoff et al., 2007, Pedrazzini et al., 2011). This software allows the representation of a DEM or 3D point clouds by a 3D shaded relief that displays the orientation of the slopes by means of a Schmidt – Lambert projection with one colour for a given dip and dip direction. It results in a coloured shaded relief map that combines slope and slope aspect in a unique representation coded by the Hue-Saturation-Intensity system (HSI). The main advantage of using the HRDEM and TLS based analyses is that the time spent in the field survey could be substantially reduced and investigations could be concentrated in the most representative/accessible areas. Recommendation and potential bias related to the remote sensing approaches can be found in Sturzenegger and Stead (2009).

COLTOP 3D analyses have been carried out on both 0.5 meters cell size DEM derived from helicopter-based LIDAR and TLS point clouds (Fig. 2a). Thirteen discontinuity sets have been identified within five different structural domains (Fig. 2b). The detected structural domains (SD) are mainly divided from the others by major faults crossing the entire area. A clear example is shown by the main gully passing in the middle of the exploited area which follows a regional fault and creates an important distinction between the structural domains on the two side of the quarry. The slightly differences in term of discontinuity orientations between the upper and the lower portion of the quarry could be related to the lithological variations. Punctual field observations were performed to characterize the geometrical (spacing, trace length and roughness) and geomechanical characteristics (infilling materials and joint compressive strength) of the detected discontinuity sets (Pedrazzini et al., 2010 for details).

3.2 Rock mass quality

The Geological Strength Index (Hoek and Brown 1997, Marinos et al. 2005) was applied to describe and quantify the rock mass quality for the different parts of the quarry. Estimated GSI values in the study area vary from 45 to 60, corresponding to a good to fair rock mass quality. However, rock mass condition in quarry is quite variable and their spatial distribution is correlated to the detected structural domains (Pedrazzini et al., 2010). Local decrease of the rock mass quality is generally observed within 2-5 meters to the main regional faults particularly in term of weathering conditions (Weathering Grade III). This could be related to water circulation and previous tectonics movements. Intact uniaxial compressive strength has been estimated using manual index tests (ISRM, 1978) and Schmidt hammer test. It varies from 50 to 75 MPa for marls layers and from 100 to 150 MPa for siliceous and spathic limestone layers. As suggested by Brideau et al. (2009), the observed GSI values correspond to heavily jointed rock masses where slope failures are mainly structurally-controlled. This observation justifies the stability analyses proposed in the next sections where the discontinuity sets characteristics are explicitly integrate.

4 Stability analysis at the quarry scale

During feasibility and operational stages, stability analyses at the quarry scale are a very important step to point out the areas where potential instabilities could occur and to have a first estimation of the potential mobilized volume (Read and Stacey 2009). Here, we associated advantages from TLS and ALS data resolution to classical limit equilibrium stability analysis method to produce a fast and reliable analysis of the entire quarry, in term of rockslide and rockfall susceptibility. This allows prioritization of areas to where slope design reassessment is needed.
4.1 Spatial kinematic analysis and susceptibility index

Kinematic analysis are widely used as slope stability test for simple structurally controlled failure modes such as planar sliding, wedge and toppling. It takes into consideration the orientation of the discontinuities, the slope orientation and the assumed effective friction angle along the discontinuity surfaces. The main limitation of classical stereographic techniques is related to the choice of the main and “relevant” slope orientation especially for complex topographic settings (Brideau et al., 2010).

The Matterocking software (Jaboyedoff et al. 2004), (available online at www.crealp.ch) compares planes or wedge intersection vector orientations with the topographic surfaces from a digital elevation model (DEM) to detect the areas where discontinuity sets daylight. This technique coupled with a GIS visualisation is very useful to analyse rugged slope topography. It allows a first screening of bench orientations susceptible to fail. The input parameters are orientation, the spacing and the trace length of discontinuity sets. Main limitation of this approach is the impossibility to introduce the natural variability of the discontinuity sets in the analysis. Matterocking analyses were performed for all discontinuity combinations identified in the different structural domains. An example of the produced map is presented in Figure 3a. In order to provide a susceptibility level based on the two sliding mechanisms, the number of potential planar slides, as well as the number of potential sliding wedges were summed and normalized, for each structural domains separately, to a number between 0 and 1 by dividing the cell value by the maximal value (Figure 3b). The produced susceptibility map has been compared with the most active areas detected and to historical and recent zones of instabilities of the quarry. The comparison shows a very good agreement with field analysis validating the proposed approach. In particular, the slope orientations parallel to the slope of the 1922 rockslide and those benches located in the SE portion of the quarry show an important susceptibility of failure.

Figure 2. Structural analysis of the main quarry area based on COLTOP 3D software analyses on TLS point cloud. A) COLTOP 3D colour view of the quarry showing the several orientation of the bench faces and the related discontinuity sets. B) Discontinuity sets orientation and potential failure mechanism susceptibility for each structural domain based on COLTOP3D and Matterocking analyses.
Figure 3. A) Example of kinematic analysis obtained for wedge sliding using Matterrocking software. B) Final failure susceptibility obtained by summing and normalizing the number of planar and wedge for each DEM cell, (0-0.15 = low susceptibility, 0.25-0.5 = moderate susceptibility, 0.5-1 = high susceptibility). The orientation and the discontinuity sets detected on the scar of the 1922 rockslide clearly pointed out as the highest susceptibility area indicating that potential instabilities could still occur in this area.
4.2 Limit equilibrium analysis

The susceptibility map reported in Figure 3 proposes qualitative information about the location of potential unstable areas. This kind of data is very useful especially at feasibility level. Stability problem concerning an active quarry like Arvel, also needs semi-quantitative information about the factor of safety and the expected unstable volumes for different bench orientations. For this reason, limit equilibrium code Swedge (Rocscience 2008) was used to investigate the volume and the stability of potential rock wedges formed on bench slopes. The combination analysis implemented in Swedge uses a user-defined list of discontinuities to calculate the factor of safety for valid wedge intersections for a given bench face. Table 1 reports the results obtained by the limit equilibrium analysis. It is interesting to note that SD 1 represents the most problematic zone as it is characterized by most important probability of failure and the largest potential unstable volumes. These results are in agreement with GIS-based susceptibility map and field observations, indicating a very useful complementarity of the two presented methods. SD 2 presents also an important probability of failure. However, in term of risk management, it does not represent a major issue, because the potential unstable volumes are relative small and easily manageable.

Table 1. Combination analysis computation obtained for different structural domain and main bench configurations.

<table>
<thead>
<tr>
<th>Structural domain</th>
<th>Assumed slope surface (orientation and high)</th>
<th>Valid wedges (% of possible combinations)</th>
<th>unstable wedges (% of valid wedges)</th>
<th>Max. wedge volume with FS&lt;1 (m$^3$)</th>
<th>Probability of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD 1</td>
<td>315/70 50 m</td>
<td>269440 (55)</td>
<td>138960 (51)</td>
<td>11'700</td>
<td>28%</td>
</tr>
<tr>
<td>SD 2</td>
<td>315/70 50 m</td>
<td>307730 (43)</td>
<td>110617 (35)</td>
<td>3'300</td>
<td>15%</td>
</tr>
<tr>
<td>SD 3</td>
<td>270/70 50 m</td>
<td>230997 (32)</td>
<td>40499 (18)</td>
<td>4'500</td>
<td>6%</td>
</tr>
<tr>
<td>SD 3</td>
<td>230/70 35 m</td>
<td>202913 (28)</td>
<td>25777 (12)</td>
<td>5'500</td>
<td>3%</td>
</tr>
<tr>
<td>SD 4</td>
<td>230/70 35 m</td>
<td>170049 (26)</td>
<td>42338 (25)</td>
<td>5'800</td>
<td>6%</td>
</tr>
<tr>
<td>SD 4</td>
<td>250/70 35 m</td>
<td>346413 (53)</td>
<td>50395 (15)</td>
<td>8'500</td>
<td>5%</td>
</tr>
<tr>
<td>SD 5</td>
<td>230/70 35 m</td>
<td>143048 (20)</td>
<td>9997 (7)</td>
<td>3'500</td>
<td>1.5%</td>
</tr>
<tr>
<td>SD 5</td>
<td>250/70 35 m</td>
<td>147990 (20)</td>
<td>16177 (11)</td>
<td>1700</td>
<td>2.3%</td>
</tr>
</tbody>
</table>

4.3 Rock fall hazard

Small rockfalls (< 1m$^3$) could have a great impact on the safety of the mine workers and it could induce important economic losses (Read and Stacey, 2009). Release zones are not often controlled by a simple failure mechanism but are mainly associated, in open pit environment, to poor blasting/scaling properties (Read and Stacey, 2009). However, relationship between structural setting and benches orientation could also have an important impact on small rockfall development. In particular, when the direction of maximum fracture density is perpendicular to the bench face, fragmentation destabilization and bench retreat increase. This situation can frequently increase the local rockfall susceptibility. In order to identify the areas with high rockfall susceptibility, we computed the discontinuity frequency in all directions of space and in each structural domain, by using the equation proposed by Hudson and Priest's (1983) and implemented for stereographic visualization by Jaboyedoff et al., (1995). Zones where maximal frequency direction is perpendicular to the slope face orientation were extracted and visualized in GIS environment (Fig. 4). This information, combined with other predisposing factors, (e.g. benches slope, proximity of large faults, rock mass and blasting quality) and structural-based susceptibility map allow a better identification of areas with important rockfall susceptibility.
5 Detailed characterization of major instabilities

5.1 Back-analysis of the 12th December 2008 event

During the night between the 11th and the 12th of December 2008 in the upper portion of the quarry, the large rockslide of 20'000 m$^3$ occurred and destroyed two borehole machines (Fig. 5a and b). This rockslide was investigated in detail to obtain precious information about the failure mechanism and the potential triggering factors. Kinematic analyses conducted on TLS point clouds by using COLTOP3D data, allowed the identification of a wedge failure involving wedges formed by the intersection between discontinuity sets J1$\wedge$J2 and J2$\wedge$J4. Field observations of the 2008’s rockslide scar indicate that wedge formed by J2$\wedge$J4 is less important in term of large slope stability due to the very low persistence of discontinuity set J4. The large wedge failure is mainly driven by the intersection of discontinuity sets J1$\wedge$J2. A limit equilibrium back-analysis of the 2008 rockslide was also carried out in order to confirm (form a mechanical point of view) the most critical wedge and to estimate the apparent friction angle and the apparent cohesion along the failed wedge. Combination analyses performed using SWEDGE® confirm the structural observations indicating that the only small potential unstable wedges (1-1’000 m$^3$) could be created by intersection of discontinuities J2$\wedge$J4. In the other hand, the aforementioned confirm that larger wedges (> 1’000 m$^3$) are controlled by intersection of discontinuity sets J1$\wedge$J2. Failure conditions (FS=1) are reached with an apparent friction angle of about 35°-38° and a residual cohesion of the infilling material of about 10-20 kPa. A preliminary 2D distinct element model (UDEC Itasca 2004) was also conducted to better characterize the failure mechanism (Pedrazzini et al., 2010). To simplify the model, only the bedding planes and the wedge intersection line have been introduced. The joint behaviour has been modelled using the strain softening model in order to account the effect of the silty-shale infilling of the discontinuities. The general failure mode can be described as a wedge-topple mechanism (Jaboyedoff et al., 2009). Results show that the bedding planes act as an active toppling surface only in the upper part of the profile. In lower part of the profile, the failure is controlled by sliding along the wedge intersection line (Fig. 6). Potential triggering factors of the 2008 rockslide are probably related to the important precipitations (snow and rainfalls) that had occurred the days before the rockslide event, coupled to the fast changes in temperature (freeze and thaw cycles) that had probably influenced the water pressure along fractures.
5.2 Comparison between remote sensing and in-situ analyses

In order to validate the structural analysis based on TLS point clouds and to define the continuity of structures in depth, three drill holes of about 60 m depth have been implemented close to the rockslide scar. Optical televiewer technique (Williams and Johnson, 2004) was applied to determine the orientation of the main structures. This technique provides a continuous and orientated 3D view of the drill hole allowing a precise detection and characterization of discontinuity planes. RQD (Deere, 1978) and visual rock mass quality investigation could also be carried out within the same borehole. Fig. 7 shows the results of the televiewer analyses compared to COLTOP3D analysis. Results indicate the good agreement between surface and drill-hole measurement and show a homogenous and constant structural setting also in depth. Low RQD values observed along the boreholes, seem to be correlated to persistent faults defining the external limits of the 2008 rockslide.
Figure 6:  A) UDEC back-analysis of the failure mechanism indicating a topple-sliding failure mode in the upper part of the cliff and sliding on wedge intersection line in the lower portion of the cliff (modified after Pedrazzini et al., 2010). B) Cross-section on the failure scar extracted from TLS confirming the modelling results and displaying the formation of a step-like failure surface. C) Detail of the rockslide scar displaying the formation of several small wedges and outlining the influence of trace length and spacing of two discontinuities J1 and J2 on the final shape.
Figure 7. Stereonet of discontinuities detected in structural domain I. A) Discontinuities extracted by COLTOP3D analysis on TLS point clouds B) Discontinuity sets extracted from the three boreholes based on optical televiewer technique. Colours correspond to discontinuity sets detected in three different boreholes located close to the 2008 rockslide scar.

5.3 Potential instabilities

A detailed database of the different instabilities detected both by field and by spatial kinematics analysis has been created for the entire quarry in order to set up a monitoring program. More than fifteen potential unstable areas are detected in the different structural domains. Potential unstable volumes vary between few cubic meters to more than 10,000 m$^3$. In particular, an important potential unstable wedge was identified 50 metres south of the 2008 rockslide scar. Geomechanical parameters deduced from back-analyses (section 5.1) were used to assess the present-day stability of potential wedge (Fig. 5b). Limit equilibrium analysis were carried out using Swedge (Rocscience®, 2008) software. Deterministic approach indicates a factor of safety of 1.2-1.3 in dry conditions. Probabilistic analysis was realized assuming a normal distribution for the geomechanical and geometrical parameters and using the Fisher distribution for the discontinuity sets. Results show that in dry conditions the probability equals 10%. The influence of water pressure was also tested by a sensitive analysis. It shows that after 30% of joint filling the safety factor decreases exponentially. Volume calculation based on TLS point clouds analysis (details on methods in Longchamp et al., 2010 and Pedrazzini et al., 2011) gives a potential unstable volume of about 21,000 m$^3$.

6 Monitoring system

As field and modelling results indicating a high rockfall and rockslide susceptibility a monitoring system was built on. Most monitoring efforts are focused on the structural domain I where most important instabilities have been detected (see Fig. 5). The monitoring system is based on continuous Ground-Based Radar coupled with repeated TLS acquisitions.

6.1 Terrestrial laser scanner

Between January 2009 and June 2009, six TLS acquisition have been carried out in the upper part of the quarry in order to detect the potential movements of the area. TLS datasets acquired during the same campaign were merged in a single file using a common coordinate system using PolyWorks® software (InnovMetrics 2009). Details on TLS data treatment for movements’ analysis could be found in Oppikofer et al. (2009) and in Abellan et al. (2009). The comparison of the different datasets indicates that the entire area is not affected by movements (at least larger than 1-2 cm). However, the comparison of TLS point cloud underlines an important rock fall activity. In the first months of 2009, more than 45 distinct rockfalls have been pointed out along the potential unstable area. The main active zone corresponds to the northern external limits of the instability (J2 discontinuity sets). The general rockfall activity drastically decreases after April (Fig. 8a). The attenuation of the rockfall
activity is probably related to the changing in weather conditions after March. In fact, a clear correlation between periods presenting freeze and thaw cycles and the number of rockfalls could be pointed out (Fig. 8b).

6.2  Ground based radar

In order to provide a continuous monitoring of potential instabilities in structural domain I as well as in the entire the quarry, a Ground Based Interferometric Synthetic Aperture Radar (GB-DInSAR) has been permanently installed at the bottom of the slope since 2009. Measurements were conducted by Ellegi Srl (Milano, Italy) using their GB-DInSAR system LiSA. Two subsequent SAR images can be combined to create an interferogram by extracting the phase difference between the two acquisitions (Tarchi et al., 2003). Displacements are then calculated between the two SAR images based on the phase differences and the wavelength of the signal. For LISA system the time span between two successive images is 3.5 minutes. The device accuracy, indicated by the manufacturer, is about 1 millimeter. Resolution is dependent on the acquisition distance and corresponds to 1.1 meters at a distance of 250 meters. More detail on this methodology can be found in Leva et al. (2003) and Tarchi et al. (2003).

6.2.1 Results after 14 months of monitoring

GB-DInSAR has been positioned at the bottom of the slope to obtain a complete vision of the main quarry. First measurements confirm that most important slope activity in term of displacement and rockfall is concentrated in structural domains I and III. In particular, several rockfalls with a volume varying between 3 and 30 m$^3$ have been identified based on millimetric pre-failure movements and a sudden pixel decorrelation. For larger rockfalls, pre-failure movements of about 3-4 mm were observed since one hour before the event. For the potential unstable wedge located in the structural domain I no movements was detected since December 2010, then small movements of about 3-4 mm appear in the lower portion of the wedge (Fig. 9). These movements were probably triggered by the important precipitation and temperature variations (freeze/thaw cycles) observed during this month. Similar climatic conditions were observed before the 2009 rockslide, suggesting an important correlation between the slope stability and the climatic factors, especially the coupled effect of rainfall and rapid temperature changes.

Figure 8. Evolution of the rockfall activity of the upper portion of the structural domain I. A) Number of rockfalls detected between the different TLS acquisitions. B) Correlation between the decrease of rockfall activity and the diminution of daily freeze and thaw cycles.
7 Risk management and early warning system implementation

In order to permit the re-opening of the entire quarry with a tolerable risk level, passive and active mitigation measures have been proposed. Active measures consist of the construction of a rockfall barrier and of a new slope design for structural domain I. Passive measure involve the creation of an early warning system and of a connected emergency response procedure.

7.1 Active mitigation measures

7.1.1 Protection of the lower installations

Trajectography modelling shows an important lack of protection structures for installation located at the bottom of the slope. Trajectography modelling (obtained for blocks of 50 m³, corresponding to the maximum block size of previous rockfalls) show that the actual barriers are not correctly designed to retain potential rockfall and rockslide falling from the upper part of the quarry (Fig. 10a). In particular, installations located in the northern portion are not protected with the present–day barrier configuration. The calculated travel-time for a rockfall between the source and the maximal run-out distance indicate a very short response time (16-19 s) that is not considered sufficient for workers safety (Fig. 10b). For this reasons, two additional geofabric barriers were designed to increase the safety of the crushing installation and of the material deposit. Pedestrian and vehicle accesses were also modified, in particular in the crushing installation area, to decrease the exposure to rockfall and to take advantage of the protection of the new and present-day barriers. Trajectography analysis indicates also that the debris cone created at the bottom of the slope after the 2008 rockslide event increased the potential rockfall propagation. In order to reduce the potential rockfalls run-out, re-profiling of the debris cone was proposed and replaced by a flat high energy absorbing area.
Figure 10. A) 3D traejectography modelling showing the potential exposure to rockfall of crushing installation and the ineffectiveness of existing measures, especially in the southern area. B) 2D rockfall modelling used to determine the time of propagation for a rockfall detaching from the source area and reaching the crushing installation.

7.1.2 Benches stabilization and new excavation concept

Geotechnical analyses conducted in structural domain I indicate clear dependence of the potential unstable mobilized volumes on the benches high and bench orientation (Fig. 11b). Factor of safety calculation follow the same trend (Fig.11a). In order to decrease the rockfall/rockslide susceptibility of the structural domain I a complete re-profiling of the upper benches is proposed. In particular the slope height will be drastically reduced to 20 m and the bench orientation will be modified northward from 320° to 350°. The new slope configuration should allow reaching an acceptable safety factor (> 1.4) and manageable potential unstable volumes (< 2000 m³).

Figure 11. Variation of the safety factor (A) and the mean potential unstable volumes (B) as function of benches orientation for structural domain I. The factor of safety is calculated, for dry conditions, using limit equilibrium method.
7.2 Early warning system and emergency response procedure

The continuous monitoring that has been set up for more than one year allows preliminary knowledge of the maximal variations on the normal trend recorded by Ground-based radar. Based on these observations, velocity thresholds have been proposed for the different potential instabilities. Back-analysis of the main rockslide event that occurred in the quarry indicates an important influence of climatic conditions on the development of large slope failures (Section 4). For these reasons, a meteorological station was implemented in the upper portion of the quarry.

Table 2. Fixed critical threshold values of the selected criterion and corresponding number of point for the emergency response procedure.

<table>
<thead>
<tr>
<th>Criteria</th>
<th>Threshold values I (Nb. of points)</th>
<th>Threshold values II (Nb. of points)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rockfalls/slope activity</td>
<td>Sporadic rockfalls from potential unstable areas and/or small volumes (&lt; 1 m³) (1 Pt)</td>
<td>Continuous rockfalls from potential unstable areas and/or large volumes (&lt; 1 m³) (2 Pt)</td>
</tr>
<tr>
<td>Precipitation/temperature variations</td>
<td>Rainfall (20 mm within 24h) or more than 1 day with freeze and thaw cycle (2 Pt)</td>
<td>Intense rainfall (35 mm within 24h) or more than 3 days with freeze and thaw cycle (3 Pt)</td>
</tr>
<tr>
<td>GB-InSAR displacements</td>
<td>Cumulated displacements in 24 h &gt; 2.5 mm and &lt; 3.5 mm (2 Pt)</td>
<td>Cumulated displacements in 24 h &gt; 3.5 mm (4 Pt)</td>
</tr>
</tbody>
</table>

Critical threshold values for temperatures variations and rainfall quantity was also proposed based on historical data. Displacements and climatic parameters have been combined to create a quantitative emergency response flowchart (Figure 12). For each criterion (rainfall, temperature variation, and observed displacements), a threshold value and a relative score have been assigned. Table 2 presents the different threshold values and the corresponding score. The sum of each parameter corresponds to a different alert level. For each alert level a risk reduction measure is associated. In particular, precise rules about the access and the activities that could be performed inside the exposed areas (potential rockfall/rockslide run-out areas, unstable benches) are clearly defined. For each alert level, roles and responsibilities of the different mine staff were also proposed.

<table>
<thead>
<tr>
<th>Nb. of points</th>
<th>Alert level</th>
<th>Direct action</th>
<th>Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>≥ 6</td>
<td>3</td>
<td>Very high</td>
<td>Site investigation by geotechnical engineer. Changing in alert level only after engineer’s advice.</td>
</tr>
<tr>
<td>≥ 3</td>
<td>2</td>
<td>High</td>
<td>Movements/weather situation checked by geotechnical engineer. Changes in alert level only after engineer’s advice.</td>
</tr>
<tr>
<td>≥ 1</td>
<td>1</td>
<td>Moderate</td>
<td>Re-evaluation of the situation every 24h by the production manager. Geotechnical engineers are informed.</td>
</tr>
<tr>
<td>≤ 1</td>
<td>0</td>
<td>Normal situation</td>
<td>Real-time evaluation by quarry workers and Production Manager. Company and state safety rules are applied.</td>
</tr>
</tbody>
</table>

Figure 12. Emergency response procedure applied for Arvel quarry. The four alert levels are associated to different access restrictions safety rules. For each level the responsibilities have been also clearly defined.
8 Conclusion

In this study, we propose an integrated approach covering the main steps of the slope design. The main challenge was to find a good compromise between a tolerable risk level and economic issues related to the quarry activity. Remote sensing techniques, in particular TLS and GB-DInSAR, have been applied both in the hazard assessment analysis and for an early warning system. By applying the proposed approach, in-situ analyses have been focused only on key locations. Detailed back-analyses of previous instabilities occurred in the quarry represent an important step that could help to identify the local predisposing and the triggering factors leading to the past failure but also to future instabilities such as the large unstable wedge detected at the top of the quarry. Calibration of critical displacements and rainfall values defining the alert levels still remains an important issue that will be only partially solved with long-time monitoring analysis. Continuous actualisation and refining of threshold values are needed to improve both safety and economic productivity of quarry operations.

9 Acknowledgements

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10 References


