Earth flow in a complex geological environment: the example of Pont Bourquin, Les Diablerets (Western Switzerland)

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ABSTRACT: On July 5th 2007, a heavy rainfall event triggered a rapid earth flow of around 11'000 m$^3$, cutting the road joining les Diablerets to Gstaad. This event was the result of an important reactivation of a landslide active area which had started in 2004. The earth flow involved the upper 5 meters of a “deep” landslide of about 20 m thick. The complex geological setting involves shales, flysch and highly permeable rocks. The dissolution of the cornieule and gypsum located at the toe of the landslide is probably the most important cause of slope destabilization. In this paper, the evolutions of slope stability through time and run-out distance are analyzed. Results are in agreement with observations. This example shows that erosion processes or groundwater changes in such material are responsible for the shallow landslides, even with minor rainfall events.

1 INTRODUCTION

On July 5, 2007 a rapid earth flow was triggered by a heavy rainfall. The landslide cut the road of an important alpine pass between the two famous ski resorts of les Diablerets and Gstaad (Western Switzerland) at the place called “Pont Bourquin”. Fortunately, the event did not cause any causality.

Figure 1. Location of the Pont Bourquin landslide near les Diablerets, surrounded by a large landslide area. Data from Swisstopo (DV335.2; MNT-MO 2008 SIT).

This earth flow occurred in an area that was identified as an active zone since 2004. In September 2006 it was estimated that 3000 to 6000 m$^3$ of a composite landslide (Cruden and Varnes, 1996) were susceptible to be mobilised in a mud-flow event that would involve the shallow portion of the 40'000 deeper landslide. The mudflow volume is now estimated to be 11'000 m$^3$ by comparing digital elevation models (DEM). The failure surfaces and volumes were estimated using sloping local base level (SLBL) technique and morphometric considerations. Stability back analysis was performed in order to assess the rock mass conditions for the 2007 event. The analysis confirms that slope was at the limit of stability in saturated conditions. Furthermore, a run-out analysis using BING (Imran et al., 2001) was performed. The results indicate that it is necessary to consider a modification of material properties during downslope flow to explain the run-out distance. This landslide appears very similar to the La Valette landslide (Colas and Locat, 1993; van Asch, et al., 2007) and the observed “slide-flow” at Super-Sauze by Malet et al. (2000). Slope reactivation of this area, close to a tourist resort, is expected to be active and cause further damages to the road. The present example shows that the reactivation of an ancient landslide initiated by erosion probably 15 years ago and not stabilized quickly can lead to risky situations.

2 METHODS

2.1 Field investigations

First field investigations were performed in September 2006 and standard mapping information was collected. In June 2007 and Mai 2008, intense field surveys were performed (Choffet et al., 2008). The
geological map was improved and the contours of the landslide were mapped in detail.

2.2 LiDAR and GPS survey

The pre-landslide topographic surface was obtained from the 1 m airborne LIDAR (Light Detection and Ranging) DEM provided by canton de Vaud. The post event DEM was obtained using a LIDAR ILRIS-3D laser scanning system from Optech Inc. with a wavelength of 1500 nm.

Six scans were used to acquire the topography. For technical reasons, the topography of the top of the landslide was obtained using a DGPS device (TOPCON® Hype –Pro).

The point distance for the Terrestrial Laser Scanner (TLS) DEM is approximately of 7 cm. 386 GPS points were added to complete the top of the DEM. The limits of the landslide were also delineated using 74 DGPS points.

2.3 Failure surface using SLBL

The SLBL is a method to estimate failure surfaces. It uses the limits of the landslide as invariant points. The SLBL can be determined either manually or by using an iterative routine that replaces the elevation of any non-invariant point of a digital elevation model (DEM) by the mean value of the altitude of its neighbors, plus a tolerance up to the moment with no changes in elevation. This leads to a curved surface of second order. In 2D, SLBL can be found numerically by an iterative procedure assuming equidistant altitude data of DEM. All points with an altitude greater than the mean value of their two neighbors are replaced by the mean value or by this value minus a tolerance (Jaboyedoff et al., 2004). This method was applied for the shallow landslide and also for the deeper one, using a 1 m grid DEM.

2.4 Simple geotechnical investigations

We performed vane test (Roctest®) in the field in order to get a rough estimation of the undrained unconfined cohesion (c_un), for the superficial material.

In order to get a characterization of the rock mass conditions, the Geological Strength Index (GSI) (Hoek and Marinos, 1997) was estimated for each lithology inside and outside the unstable area.

In case of lack of geotechnical data the GSI results could be used directly to calculate the factor of safety based on the generalized Hoek and Brown criterion (Hoek and Brown, 1988).

2.5 Modeling

The back analysis of the slope stability was performed using Slide® (Rocscience) software. The safety factor was estimated using different slice methods (Bishop, Spencer, Janbu corrected and Morgenstern-Price). The groundwater condition was computed using the finite element analysis implemented in Slide®. A sensitivity analysis of the Ru values was also carried out, in order to confirm this computation. Ru represents a parameter which models the pore water pressure as a fraction of the vertical earth pressure for each slide along the critical slip surface. The deeper landslide was modeled using the GSI values obtained outside the unstable mass. For the earth flow source area, the stability was estimated using the GSI values obtained inside the unstable mass. These values are lower compared to the first ones, due to rock mass damage caused by continuous flexural toppling (Figure 2B)

The back analysis of the runout distance is based on the estimation of the mobilized yield strength at the time of failure. Assuming that the final topography reflects the strength of the material involved in the debris flow, a parametric analysis was carried out using Hampton’s (1972) equation:

\[ \tau_c = H_c \gamma \sin \beta \]  

Where \( \tau_c \) is the yield shear strength of the material, \( H_c \) the critical thickness, \( \gamma \) the unit weight and \( \beta \) the slope angle. For the run-out modeling we used BING 1D- flow dynamics model software (Imran et al. 2001), with a bilinear rheological model.

3 GEOLOGICAL AND GEOMORPHIC SETTINGS

3.1 Geology

The geology is rather complex; four geological units are encountered within the 300 meters of the landslide (Badoux and Gabus, 1990). The different geological units are delimited by important tectonic thrusts. Starting from the lower part of the slope the following can be observed:

1. Weathered gypsum actively dissolved;
2. Over 50 m of cornicule. This formation is represented by a vacuolar dolomite with high permeability. These two lithologies belong to the Bex Nappe.
3. 150 m of Flysch composed of thin bedded turbidites including shale siltstone and few conglomerate and belonging to the Plaine Morte unit.
4. The next 70 m are thin bedded clayey schists of the Arveyes Nappe;
5. At the top of the slope cornicule from the Meilleret Nappe are outcropping;
6. (4) and (5) are overlaid by moraines; a ground moraine and a terminal moraine on the Northwestern part. It is possible that all these lithologies contain folds and others internal thrust. The contacts between units are dipping down to North by 20 to 30°. The bedding of the (3) and (4) lithologies follow the same trend, which allows flexural toppling over at least the first 3 meters.

3.2 The landslide activity

In the study area, numerous landslides are still active (Schoeneich, 1996; DUTI, 1985). The Pont Bourquin landslide is located in the western part of a former scar (Fig. 1). West of the Pont Bourquin area, the Parchet landslide is active since year 2000 and moves 5-10 cm.year\(^{-1}\) (Schoeneich, 1996). The surrounding zone is certainly active since the last glacial retreat. The Pont Bourquin slope is affected by two landslides which affect mainly the less competent lithologies in the central part of the slope (shale and Flysch) and morainal deposits in the upper part of the slope. The deeper landslide is characterized by an important subsidence at the top, delineating clearly the scar and implying an important rotational component in the upper part. The toe of the landslide is probably located above the cornièule (2).

3.3 Erosion activity

Erosion activity started at least for more than 10 years (Fig. 2). The orthophotos taken in 1995, 1997 and 2004 indicate a constant degradation of slope conditions. In 2006, significant displacements (80 cm) were observed above the erosion area (Jaboyedoff, 2006), indicating that not only the shallow part was moving but also influenced the deeper instability. The 2007 event was triggered after 3 days of heavy rainfall. The rainfall data from the meteorological station of Diablerets village, recorded an accumulated rainfall of 95 mm with a mean rainfall intensity of 2.5 mm.h\(^{-1}\) the previous 3 days. This value corresponds to a 5 years return period. Even with minor rainfall events, and considering the superficial erosion evolution between 1995 and 2004 (Figure), a major destabilization event is expected in following years.

4 RESULTS

4.1 Geomorphic interpretation

The analysis of the morphology and the SLBL method indicate that the mass movement has two differentiated parts: a deep failure with an average thickness of about 15-17 m and a shallow failure with a thickness reaching up to 5 m.
The total area of the deep landslide is around 8200 m$^2$. Its width varies from 12 to 60 m. The length is 244 m up to the road. The shallow landslide area is about 5000 m$^2$. The total volume of the shallow landslide based on the DEM before the event and the SLBL indicate that 11'000 m$^3$ were mobilized along the slope using only the area that shows significant flows on the topography (Figure 6). Only a few thousand reached the toe and subsequently the road. The estimated volume of the deep landslide is around 40'000 m$^3$ using the same method, but using as invariant points the external contour of the landslide.

4.2 Geotechnical parameters

The vane tests performed on the top of the landslide vary between 12 to 130 KPa in the superficial formations involved in the earth flow (moraine and schist). The parameters used for the Hoek and Brown failure criterion (1988) are given in Tables 1 and 2. The GSI measurements were performed outside the shallow unstable area to simulate deep landslide and within the landslide for the earth flow initiation zone.

Table 1. Parameters estimated by field investigations outside the main landslide area (deep landslide modeling).

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Cornieule</th>
<th>Schists</th>
<th>Flysch</th>
<th>Gypsum</th>
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<tr>
<td>Intact UCS (KPa)</td>
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<td>20'000</td>
<td>30'000</td>
<td>15'000</td>
</tr>
<tr>
<td>GSI</td>
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<td>25-35</td>
<td>25-35</td>
<td>35-45</td>
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<td>Intact Rock Constant $m_i$</td>
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<td>6</td>
<td>10</td>
<td>10</td>
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<tr>
<td>Disturbance Factor</td>
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<tr>
<td>Unit Weight (kN/m$^3$)</td>
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<td>23</td>
<td>25</td>
<td>23</td>
</tr>
</tbody>
</table>

Table 2: Parameters estimated by field investigation on the main landslide (shallow landslide modeling).

<table>
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<th>Flysch</th>
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</thead>
<tbody>
<tr>
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<td>25'000</td>
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<tr>
<td>GSI</td>
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<tr>
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4.3 Stability back-analysis

Using the parameters obtained in the field we computed the factors of safety for both landslides in fully saturated conditions. Using the undisturbed rock GSI values we obtain a quite deep failure surface with a safety factor varying between 1.3-1.4, depending on the calculation method. This surface corresponds to the deep landslide. This landslide was not directly involved in the 2007 event; nevertheless the FS is relatively low, indicating a potential unstable situation.

Figure 5. Cross-section along the landslide. Note that in the lower part near the road area the profile of DEM 2008 is extrapolated from pictures just after the 2007 event.

Figure 6. Results of the geomorphic analysis. Colors represent the thickness of the mobilized mass during the 2007 event. Black contours represent the modeled deep failure surface.

Sensitivity analyses for each lithology have been carried out varying Ru values from 0.1 to 0.5. Variations of Ru values in Flysch induce important changes in FS. The FS calculated using the ground water condition based on the finite element method corresponds to a Ru value of 0.2 to 0.3. This represents a realistic range of values compared to the water conditions observed in field. In the second model using the GSI values for the disturbed rock mass we obtained a shallow failure surface with a safety factor close to 1. The failure surface and the calculated volume correspond to the values obtained by the SLBL method. Figure shows that the presence of highly permeable and soluble layers such as
cornieule, strongly influences the general slope stability at the toe of the slope. Additionally, during important rainfall we could expect local pore water overpressure due to complex geology. The real influence of the cornieule layer is difficult to model using limit equilibrium model because of time dependent chemical weakening. In addition, an analysis has been carried out varying the anisotropy of the permeability coefficient for the case of the deep slide, obtaining a factor of safety around 1 when vertical infiltration dominates. For the case of shallow slides, a weak layer could exist (flexural buckling) that could promote the development of landslides.

4.4 Runout back-analysis

The parametric results using Eq. [1] are shown in Figure 6. It is interesting to note that field measurements in the shallow zone, provided values of the undrained shear strength ranging from 12 to 130 KPa. Considering that the debris mass had already started to drain these values should be considered as a maximum. As indicated in Figure 6, using Eq. [1] for a slope corresponding to the starting zone, i.e. about 25° would require a yield strength of about 40 KPa for an average thickness of 5m which is of the same order of magnitude than the in situ strength measured using the vane test.

Figure 7. Parametric analysis relating the critical yield strength as a function of the critical thickness for various slope of the debris (using a unit weight of 18 KN/m³) based on the cross-section in Figure.

The analysis of Pont Bourquin earth flow has been carried assuming that the average unit weight of the debris of 18 KN.m⁻³ and an average strength of 3 KPa, which would correspond to the observations of the material accumulated on the road (2m). This suggests that a significant part of the debris had an increase in the water content and the yield strength was reduced by a factor of 10. Using BING software (Imran et al. 2001) with a set of parameters that enable the flow to stop on the road, that the velocity reached a maximum of 12 m.s⁻¹ at the onset of the flow but was moving at velocities less than 5 m/s when approaching the road.

As it has been already shown in La Valette landslide (Colas and Locat, 1993; Malet et al., 2005; van Asch et al., 2007) the landslide in shale-like material exhibits a complex behavior. The process starts as a translational slide and transforms progressively to a flow with increasing water content.

Figure 8. (A) Back analysis of the deep landslide. (B) Back analysis of the shallow landslide stability. See Figure for lithologies.

Figure 9. Velocity of the frontal element as a function of distance from its initial position in the slope. The horizontal distance of 300m corresponds to the road location.
The results show that the stability of the mobilized material is close to equilibrium and once it starts to move, the yield strength decreases, increasing the mobility. The preparation factor to weaken the shallow rock can be attributed to flexural toppling.

Figure 10. Profile of the deposit at the end showing an average thickness of 2m over a length of about 200m.

This mechanism deforms the rock mass and creates shear bands in the shale layer, that can be transformed to a paste-like material. This weakening is also increased in the first 10 cm by freezing and thaw cycles that fragment the rock up to a paste consistency. Additionally, the till deposits are involved in the earth flow. These factors influence landslides when intense precipitation occurs. As mentioned before, due to the presence of cornielue in this folded complex geology high pore pressure can be suspected inside the slope. It is also clear that gypsum and cornielue at the toe are the source of the deep movements that started the weakening process. Active erosion at the top is certainly one of the main destabilization factors for earth flow processes. The lacks of surface vegetation cover increases surface erosion and can cause bad-lands if the area is not reforested.

Figure 11. Surface conditions of shale fragmented by mechanical crushing related to freezing and thaw cycles.

5 CONCLUSION

This case study represents an example of the influence of fine grained lithology in the formation of complex mass movements, which start as a slide and evolve into an earth flow, capable to travel down slope at high velocities. The predisposing factors for this phenomenon seem to be related the reduction of the rock mass strength induced by flexural toppling movements, freeze and thaw cycles and continuous surface erosion.

The main triggering factor is an intense rainfall of short return period, inducing a rapid increase of the water pore pressure and influencing the rheology of the material. The complexity and the variability of the geological setting introduce other important variables that very difficult to asses without an intensive geotechnical investigation.

REFERENCES


