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STABILIZATION OF A LARGE PALEO-LANDSLIDE REACTIVATED BECAUSE OF THE WORKS TO INSTALL A NEW SKI LIFT IN FORMIGAL SKIING RESORT

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ABSTRACT

The paper presents the study and stabilization of a paleo-landslide reactivated because of the excavation works carried out at its foot to construct a new lift in Formigal skiing resort. Inclinometers were installed and surface movements were controlled by GPS techniques in order to analyse the sliding surface and to define the stabilization measures to undertake. The data obtained from the instrumentation was used to perform a back-analysis that allowed to determine the kinematics of the movement as well as to define the appropriate stabilization measures. Presently, the evolution of the movement is controlled by GPS.

GEOLOGICAL CHARACTERISTICS

Geology of the area

The slope affected by the landslide is located in the so-called “Axial Zone” of the Central Pyrenees, in north-eastern Spain. The geology of the area is characterized by the presence of Paleozoic materials, mainly Silurian, Devonian and Carboniferous, affected by intense folding corresponding to the Hercynian orogenic processes, and reactivated later on by the Alpine orogeny. The materials present a low degree of metamorphism and a high intensity of fracturing.

Colluvial soil deposits, originated from erosion, weathering and transport of Paleozoic materials, are found on top of the bedrock. These colluvial materials are characterized by the presence of brown to dark grey clays, sometimes highly plastic, with shale, slate, sandstone and limestone pebbles.
The presence of different paleo-slides is clearly visible at the hillsides on both banks of Gállego river. The slope studied here is affected by one of the most significant of these paleo-landslides.

A deep valley was dug by the Gállego river after the uplift caused by the last phase of the Alpine orogeny. Due to the high alterability of the shales, a thick layer of weathered material was formed on the slopes, consisting of clays with limestone and slates pebbles.

The paleo-slide that can be observed presently at the site was originated by the eroding action of the Gállego river at the foot of the hillside and was probable triggered by seismic shaking.
As the paleo-slide invaded the river bed, a new channel was dug by the river through the slid mass, without reaching again the elevation of the underlying bedrock.

By means of the indirect methods used by *García Ruiz et al.* (2003), a period of about 20,000 years BP of significant ground movements, coinciding with the melting process of the Pyrenees glaciers, was determined.

Geomorphology of the paleo-slide

The paleo-slide is 1,500 m long, the difference of elevation between crest and toe being of 400 m. It is bounded laterally by two small creeks oriented parallel to the line of maximum slope angle and separated a distance of 200 to 500 m.

At the crest zone, slates dipping 30º-40º towards the Gállego river outcrop. Below, the slope angle of the hillside is between 8º and 12º.

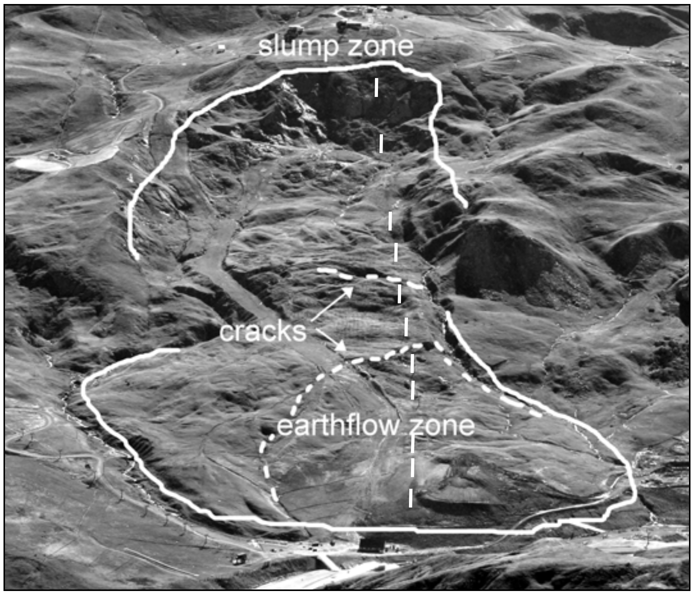

Figure 1. Panoramic view of the paleo-slide

The surface of the slope is completely convex in the area occupied by the slid mass, and creeping phenomena are frequent there. At the toe of the slide, the Gállego river is in a process of excavating the slid mass tongue. This is the cause of the existence of small slides near the river course.

In addition, some ponds, with water all the year around, can be found at mid-height of the slope, due to the local run-off and to the imperviousness of the slid material.

Figure 1 shows a panoramic view of the paleo-slide, as well as the location of the main cracks and the pillars of the ski lift.

Figure 2. Cracks at the crest of the slide.

KINEMATIC OF THE SLIDE

General characteristics

As shown in figure 1, the slide was reactivated at an area of accumulation of paleo-slide materials. This new slide has a length of 550 m, an average width of 250 m and a depth around 35-40 m. The slide was detected in October 2004 and the cause of the reactivation was associated to the excavation works carried out at the foot of the hillside to construct a new ski lift.

The most important cracks appeared at mid-height of the hillside, between pillars 7 and 8 of the ski lift. These cracks, with widths up to 2 m (see figure 2), form a big circle that cuts all the width of the paleo-slide. Other secondary cracks were observed at a higher elevation, around pillars 9 and 10, as a consequence of the lack of support originated by the slide below.

At the toe, the slide reaches the river. No surface cracks were visible at that area. However the movement of a pillar of a bridge over the river, the one located at the paleo-slide bank (see figure 3), shows clearly the occurrence of movements in depth at that zone.

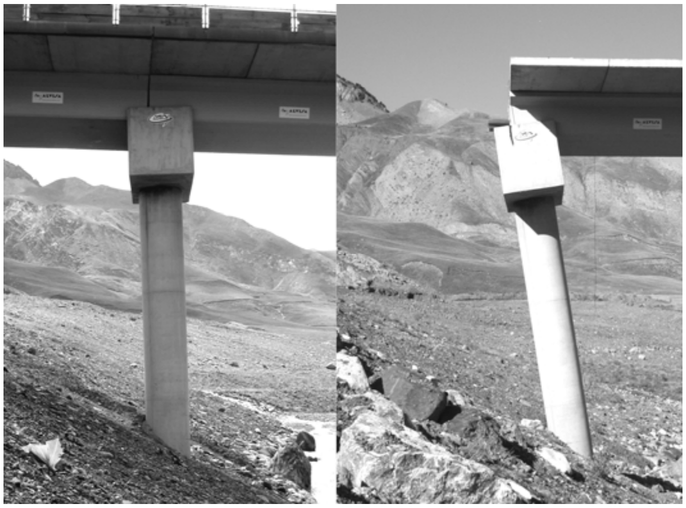

Figure 3. Evolution of the movement of a pillar located close to the toe of the slide

Visibly, the head of the slide exhibited a significant settlement, whereas the toe area moved slightly upwards, indicating a deep rotational-translational kind of mechanism for the slide.

Control of movements

A total of 93 surface control points by GPS were installed in order to analyse the kinematics of the slide. The points were located in the area presenting significant movements, as well as in zones presumably stable. In addition, 11 inclinometers were installed to detect the situation of the failure surface in depth.

The cross-section corresponding to the alignment of the ski lift pillars, located at the center of the slide and along a line of maximum slope, was chosen for analysing the slope movement. The points located at this cross-section at the pillars near the crest of the slide exhibited downward vertical displacements, whereas those situated at the toe of the slope showed a slight upward vertical displacement (figure 4). A similar pattern of surface movements was observed over the entire slide area. Movements were measured from October 2004 to January 2007 and they are still being controlled.

As to horizontal movements, figure 5 shows the horizontal projection of the displacement vectors determined for the different control points. The vectors are oriented towards the lower zone of the slope. Displacement rates of up to 7 m/year were registered.

The readings of the inclinometers indicated significant rates of movement and showed clearly the position in depth of the failure surface. In general this surface was detected at a depth of about 15 m, near the head of the slide, and of 40 m at the center. It should be noticed that by the river, the failure surface was detected at a depth of 30 m, due to the fact that the river bed is still located in paleo-slide materials, far above from the bedrock elevation.

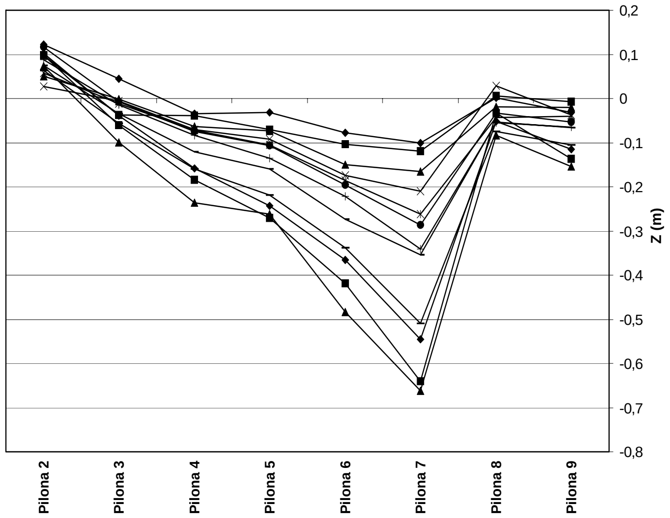

Figure 4. Vertical displacements measured of ground surface at the ski lift pillars

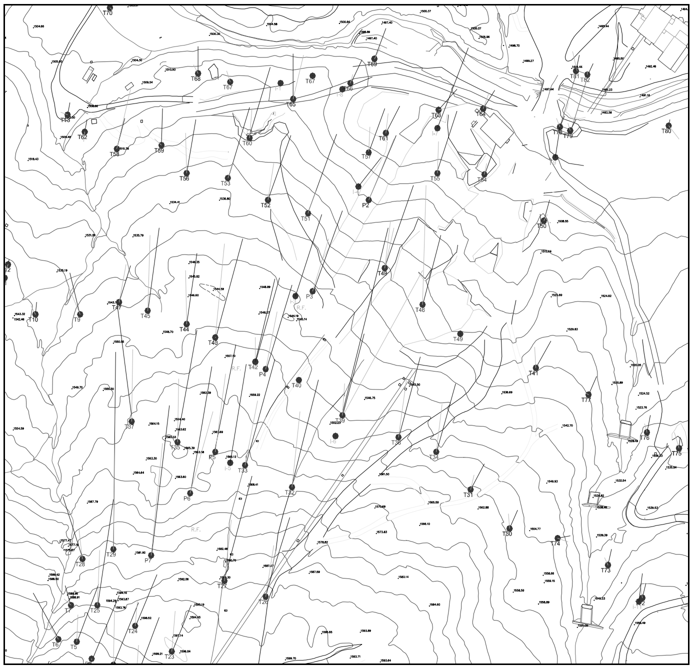

Figure 5. Displacement vectors measured at the surface marks by topographical control.

Obtaining the failure surfaces

By drawing the perpendiculars to the displacement vectors corresponding to the ski lift pillars, an array of straight lines was obtained converging very accurately on the same point, showing that the movement of the sliding mass could be assumed to be circular.

So, by drawing circles passing through the failure points determined by the inclinometers, four different possible failure surfaces were obtained.

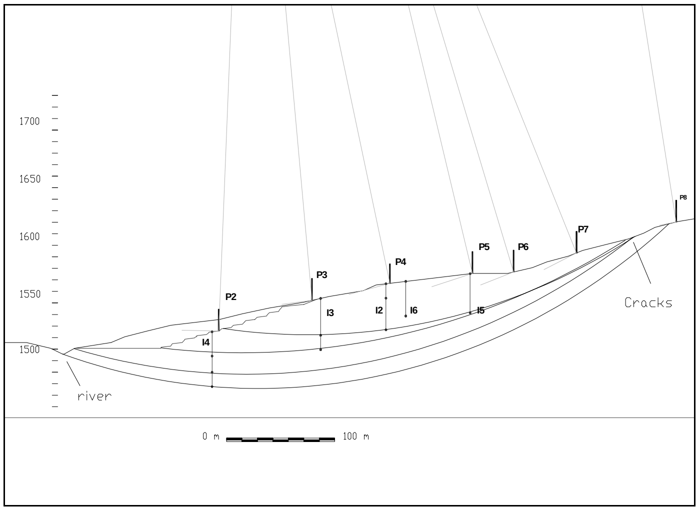

Figure 6. Possible failure circles determined from displacement vectors and inclinometer readings

Back- analysis

A stability analysis was performed considering the four possible failure surfaces, determined as mentioned above, in order to determine strength parameters of the materials affected by the landslide..

Considering the movements that were going on, a global factor of safety equal to unity was assumed. Therefore, the strength parameters of the materials were estimated on the base of a factor of safety of 1, the actual geometry of the slope and the four possible failure mechanisms that had been obtained.

Figure 7 shows one of the failure surfaces analysed in the study.

In order to calculate the strength parameters of the natural ground, a fixed value was considered for one of the parameters (cohesion or angle of friction), and then the other was obtained by imposing a factor of safety of the order of unity. According to this, four couples of strength parameters were considered for each failure surface geometry in the study. Values of 5, 10, 15 and 30 kN/m^2 were adopted for cohesion, the corresponding values for the friction angle being those shown in table 1.

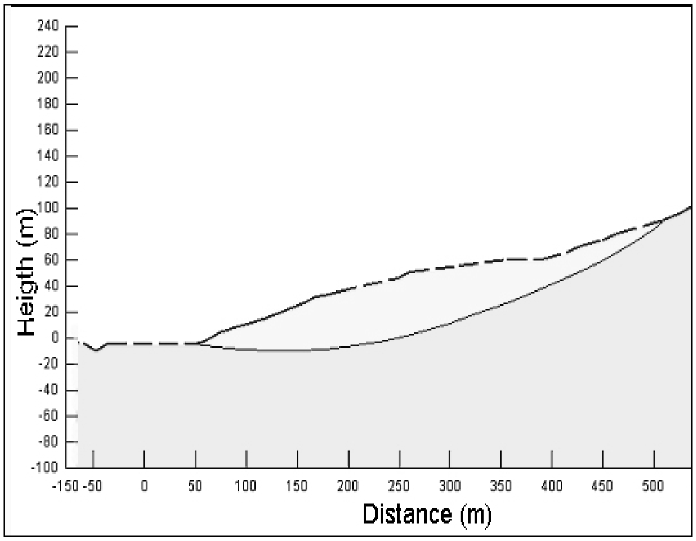

Figure 7. Example of one of the failure surfaces analysed.

In addition, as shown in table 1, local factors of safety were calculated which indicate the minimum factor of safety that take place in localized failures.

Table 1. Ground strength parameters obtained in back-analyses

Surface 1			
	T/m^2	phi(°)	SF local
Cohesion	0.5	20.5	0.798
Cohesion	1.0	19.5	0.828
Cohesion	1.5	18.5	0.852
Cohesion	3.0	15.5	0.911

STABILIZATION MEASURES

Design of stabilization measures

As mentioned before, the origin of the landslide reactivation process was attributed to the excavation work realized at the toe to build a new ski lift.

Stabilization measures had to be adopted following two main lines of action:

- Construction of a stabilizing fill in the previously excavated area, to provide a surcharge higher than the weight of the soils that existed before the excavation works.
- Drainage of the ground surface to get a depressed water level position in the area of the stabilizing fill.

It is important to point out that any stabilization measure to be adopted had to take into account the environmental legal restrictions in force in the area. One of them prohibited drainage operations at the upper part of the landslide, to protect some important ponds for protected species.

To determine the stabilization measures, different situations and geometries were considered in the calculations:

- Pre-existing situation, before the excavation works;
- Situation after execution of drainage and fill;
- Situation after construction of fill, but admitting failure of the drainage works.

The strength parameters obtained in back-analysis were used in these analyses, performing additional calculations for each of the possible failure surfaces.

The geometry adopted for the second case, (situation after execution of drainage and construction of fill) is shown in figure 8. In this case, the water level was considered to be located at the position of the natural ground under the fill.

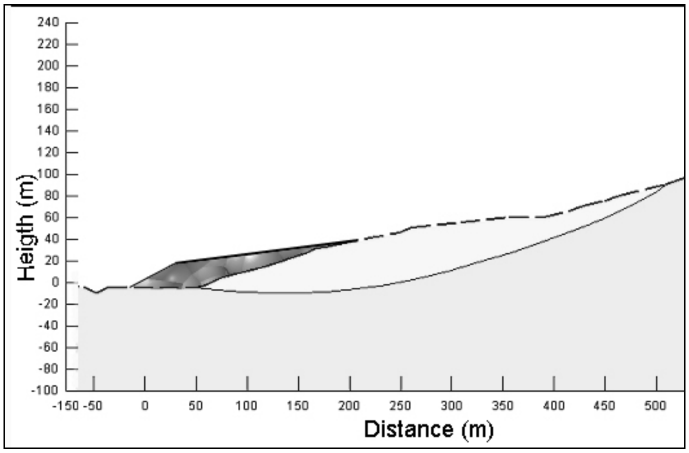

Figure 8. Cross-section of the landslide showing one of the failure surfaces and the fill at the toe area

Table 2 presents a summary of the calculations (surface fig.7) carried out, as well as the resulting factors of safety.

Table 2. Summary and results of analyses carried out (surface fig. 7)

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Cohesion	**Intial state**	**failure state**	**fill+drai.**	**fill without drai**
T/m^2	SF	SF	SF	SF
0,5	1.268	0.798	1.545	1.308
1.0	1.262	0.828	1.514	1.297
1.5	1.256	0.852	1.498	1.288
3.0	1.243	0.911	1.455	1.264

From the results obtained in these analyses, it was inferred that the hillside stability presented a factor of safety between 1.14 and 1.26 before the excavation works, situation that being not very satisfactory, was though sufficient. These low values were due to the fact of the slope being formed by a paleo-landslide.

As to the second case, with the fill and the effect of the drainage, factors of safety increased significantly to values between 1.28 and 1.54, considered sufficient for the slope stability, taking into account the assumptions adopted in the analyses, in particular, the values adopted for the strength parameters, deduced from back-analyses.

In the third case, factors of safety between 1.14 and 1.31 were obtained, clearly below the values calculated with drainage.

<u>Description of the stabilization measures undertaken</u>

The stabilization measures were carried out to fulfill to main objectives:

- Increase of safety against sliding at the toe area by placing there a surcharge fill;
- Lowering of water table at the bottom of the landslide area.

The fill was constructed with the same material extracted from the excavation, but improving its properties by a proper placing procedure and compaction.

Besides, additional weight was provided by increasing by 30% the height of the pre-existing material.

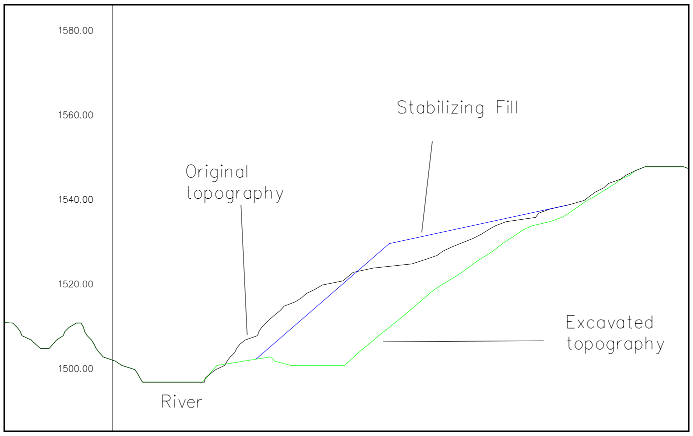

Figure 9. Detail of final cross-section after construction of the fill compared with the pre-existing geometry

In some areas, where the landslide material was softened or presented unsatisfactory properties, it was substituted also by proper compacted material.

At the lowest part of the slope, by the river, buttresses of granular material, 0.6 m wide and 3 m deep, oriented according to the maximum slope direction, were constructed to provide further reinforcement.

To allow drainage of the surface prior to the placement of the fill, a system of draining trenches, also oriented in the direction of the maximum slope, was constructed. These trenches were 0.5 m wide and 0.5 m high, and were provided with a filter geotextile, granular material and slotted tube at the bottom of the trench. Distance between trenches was 10 m.

Some other trenches were made following an oblique direction in order to facilitate interconnection between them and a better way out for the drainage water.

Figure 10. Construction of the drainage trenches

Figure 10 shows the construction of the trenches, whereas figure 11 shows the works carried out to place and compact the fill.

Figure 11. Placement and compaction of fill material

Finally, figure 12 shows a panoramic view of the slide after completion of the stabilization works. In the figure, the broken lines represent the main cracks at the head of the slide, whereas the location of the drainage works and stabilizing fill is drawn in continuous line.

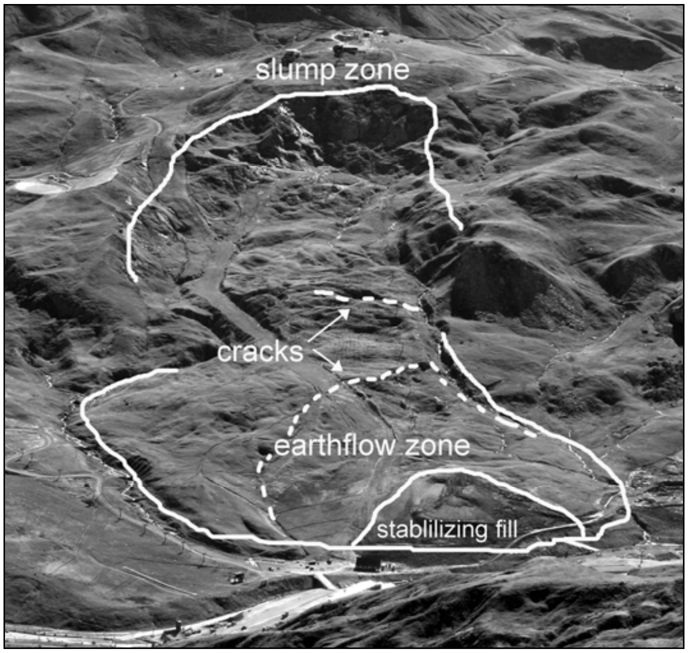

Figure 12. Panoramic view of the location of the stabilizing fill

RESULTS OF THE STABILIZITION

After carrying out the stabilization works, an intense control of movements was implemented to observe the behaviour of the landslide and to verify the efficiency of the solution adopted. This control was performed by GPS techniques using the same net of surface points as before the stabilization.

Figure 13 shows how the displacement rates of the control points have decreased after the construction of the drainage system and stabilizing fill.

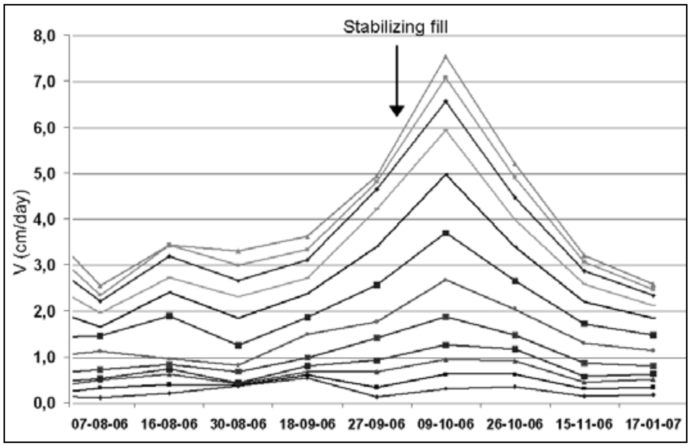

Figure 13. Displacement rates at control points located at the ski lift pillars

Although the whole slid mass movement continues, as a consequence of its considerable big inertia, it is foreseeable that these displacements will diminish in time.

The drainage trench system has proved to be very effective, as a considerable volume of water, up to 3 l/min in September 2006, has been observed in some of the trenches.

CONCLUSIONS

The reactivation of the paleo-landsilde was caused by the excavation works carried out at its toe zone.

Control of surface movements by means of GPS techniques permitted to make a preliminary analysis of the kind of failure mechanism that was occurring, which was confirmed from the data obtained with the inclinometers that were installed in the slid mass.

The construction of an adequate drainage system and a stabilizing fill has led to the deceleration of the landslide. The fill was constructed using materials obtained from the slid mass, but improving their properties by adequate compaction.

Present-day control with GPS techniques permits us to carry out an adequate observation of the kinematics of the landslide and has shown that the stabilizing measures undertaken turned out to be effective.

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