Slope stability monitoring:
an Ethiopian case study - Gibe III power house

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Introduction

Gibe III is the third plant of the Gibe–Omo cascade that comprises Gilgel Gibe (IP = 200 MW) and Gibe II (IP = 420 MW), both operating, together with Gibe IV and V which are planned. The general layout of the Gibe III project is shown in the rendering model of Fig. 1.
The project includes a 246 m high RCC dam that will become the world's highest of its kind.
The open-air Power House (PH) is located on the left bank of the river, about 500 m downstream of the dam axis, at the base of a very steep slope. The PH building, 235 m long, 38 m wide and 50 m high, will host ten Francis turbines for a total installed power of 1870 MW.

During the PH excavation and support works, the measurements acquired with the available monitoring system (consisting in topographical targets and single point extensometers) indicated a general trend of displacements of the excavation front directed prevalently towards the river. Although topographical readings were quite scattered and no quantitative measurements of skin-shotcrete fissures and upper berm crack opening were available, these data were attributed to a probable distressing of the PH excavation front which required appropriate and immediate counter-measures in order to avoid potential rock mass instability. This interpretation of the available data was strengthened by the visual observation of a tension crack on one of the front excavation berms and fissures on the skin-shotcrete of the manifolds galleries (most probably related to the opening of one of the main rock joint-set). Consequently the following actions were taken:

- temporary suspension of the excavation works;
- execution of additional support works on the excavation front of the PH and on the rock pillars between the manifold galleries;
- implementation of an optimized excavation sequence;
- up-grading of the available monitoring system.

This paper briefly illustrates the stability analysis of the Power House excavation front and the relevant support works designed to assure the completion of the excavation works with adequate safety factors and minimum delay.

The completion of the excavation works and the effectiveness of the additional support works have been thoroughly checked over a period of more than one year by means of an improved monitoring system which is described in this paper and is, as a matter of fact, an integral part of the design of the excavation front stabilization works.

1. Description of the area and interpretation of the first available monitoring data

The excavation works carried out in the Power House area have determined an approximately 95 m high excavation front, divided from bottom to top as follows:

- LOWER SLOPE (el. from 660 to 700 m a.s.l.)
  A temporarily exposed excavation front between PH foundation and transformer yard with a slope of 1:5 (H:V) (to be recovered by the PH concrete structure).
- INTERMEDIATE SLOPE (el. From 700 to 730 m a.s.l.)
  A permanently exposed excavation front between the transformer yard and el. 730 m a.s.l. The intermediate slope comprises three berms 10 m high with a slope value ranging from 1:5 (H:V) to 1:2 (H:V).
- UPPER SLOPE (el. From 730 to 755 m a.s.l.)

A permanently exposed excavation front above el. 730 m a.s.l. The upper slope is constituted of two berms with a height of 10 and 15 m and a slope of 1:2 (H:V) and 1:1 (H:V) respectively.

Fig. 1. General layout of the project (Rendering model)

Fig. 2. Panoramic view of the Power House excavation front during concreting

The trachy-basaltic rock mass affected by the PH excavation is characterized by the presence of the two main sets of ubiquitous and very-persistent sub-vertical joints, called K1 and K2, as shown in the following figure.

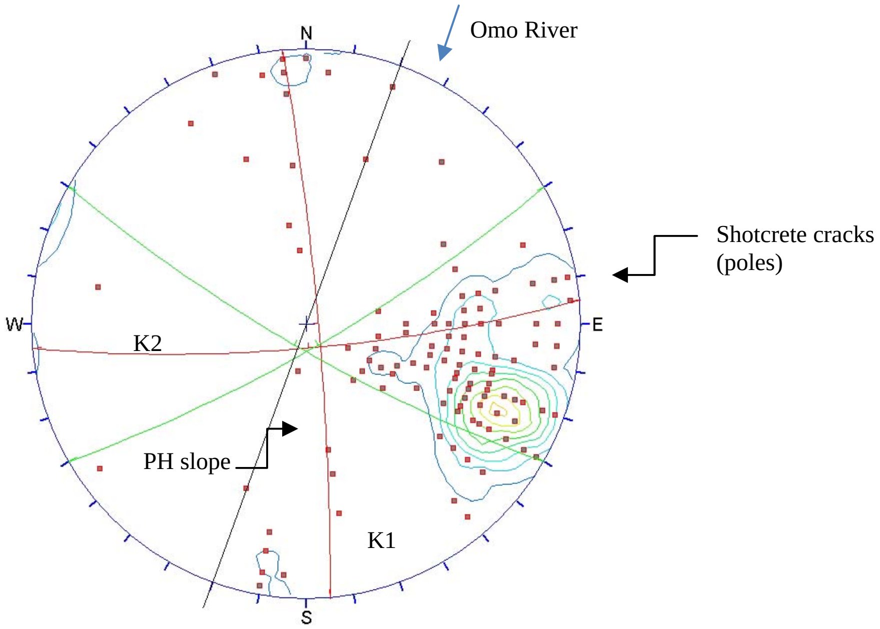

Fig. 3. Excavation front stereonet

Although quite scattered, the topographical measurements and extensometer readings, as well as the cracks occurred in the skin-shotcrete of manifold galleries and the low velocity zones identified by the geophysical survey carried out on the rock pillars between manifolds, correspond to the signs that generally appear in a rock mass as a consequence of operations involving significant distressing, such as large scale excavations.

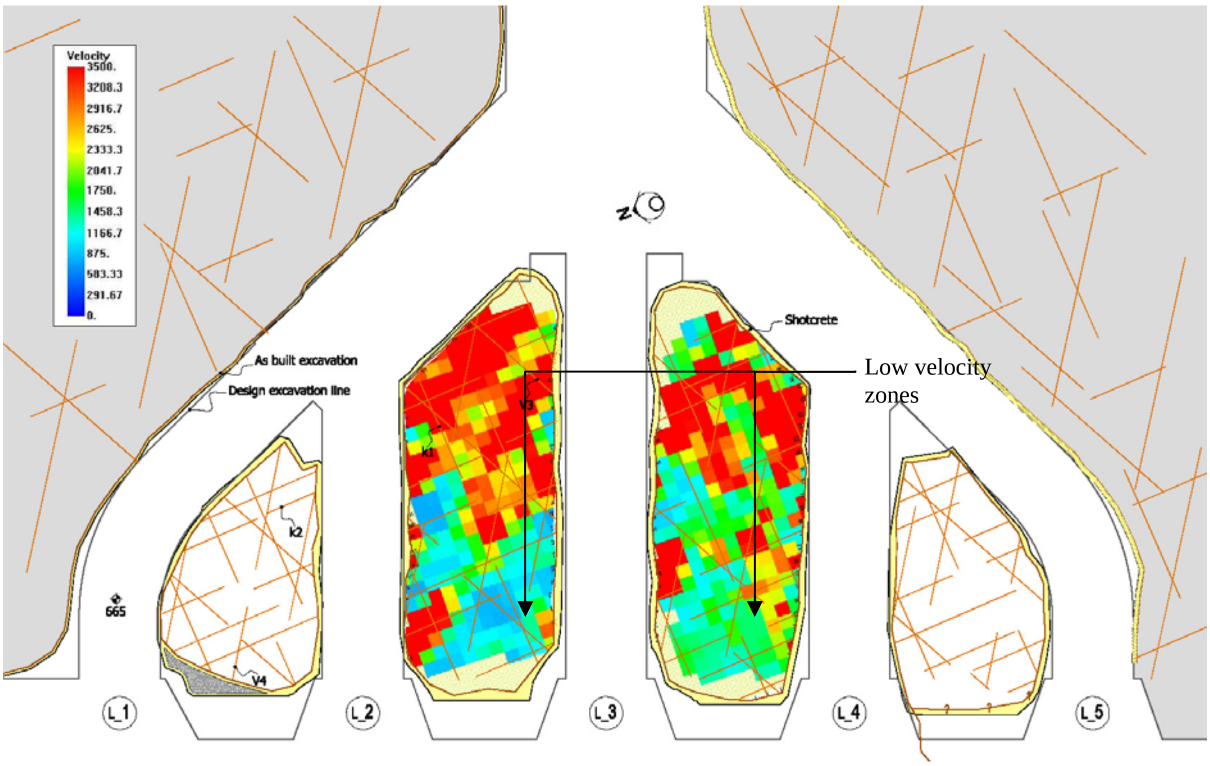

Fig. 4. Rock pillar tomographies

The K2 joint set, sub-parallel to the excavation front, is particularly susceptible to a decrease in the horizontal stress, which certainly occurred as a result of the excavation works. The cracks observed on the skin-shotcrete of the manifold galleries, with smaller spacing near the excavation front, are most probably related to joints of set K2 that, following the rock mass distressing, tended to open.

2. Stability calculations

2.1 Rock pillars

The stability analysis of the pillars between the manifold galleries was performed using both bi-dimensional finite element modelling (FEM) and empirical criteria based on best-fit equations derived from observed pillar performance data.

The bi-dimensional finite element model was carried out by means of the program PHASES[2] adopting, for the rock mass, the Hoek-Brown and Brittle Hoek-Brown elasto-plastic failure criteria.

The empirical formulations evaluate the pillar safety factor as the ratio of pillar strength and average axial pillar stress level. The pillar strength is estimated using two empirical formulae proposed by “Hedley and Grant” and “Stacey and Page” with the same form:

$$S_p = \sigma_c \cdot \frac{W_p^a}{h^b}$$

where:

σ_c	[MPa]	=	pillar rock’s uniaxial compressive strength
W_p	[m]	=	pillar width
h_p	[m]	=	pillar
a, b	[-]	=	exponents depending on pillar’s volume and shape

Both calculations (FEM and empirical formulations) indicated that, in the actual conditions, the rock-pillars between the manifolds have a safety factor greater than the required minimum.

Nevertheless, considering the critical importance of the area, a precautionary additional support was designed consisting in two rows of longitudinal tendons (with reinforced concrete beams for load repartition) tensioned at 1560 kN and two rows of transversal active self-drilling anchors tensioned at 360 kN. These supports give the pillars a lateral confining load corresponding to about the 5% of the vertical stress.

The support works have been completed with consolidation grouting of each pillar and structural lining of the manifold galleries (300 mm of shotcrete with 3 layers of welded mesh 5x150x150mm). In correspondence with the main pre-existing shotcrete fissures “windows” have been left for monitoring purposes and visual examination.

2.2 Excavation front

The stability calculations of the excavation front have been carried out with a conservative approach assuming the following simplified failure mechanism:

- planar sliding surface along secondary joint inclined at 45° dipping out of the excavation front;
- vertical tension crack from the internal side of the berm at el. 700 m a.s.l. up to the planar sliding surface;
- release lateral surfaces (represented by the K1 joints set).

The additional support works (calculated assuming that the safety factor is equal to 1 in the actual conditions) included:

- two row of dowels at the foot of the slope (type dywidag, ϕ32, L = 12 m, spaced 2 x 2 m, inclined at 10°);
- one row of tendons, L = 30 m, spaced 2 m, inclined at 5°-15°, tensioned at 1560 kN;
- consolidation grouting carried out from the berm at elevation 700 m a.s.l.

3. Upgraded monitoring system

3.1 General

The excavation front monitoring system originally implemented on site was composed of 4 single-point extensometers and 22 topographical targets installed at various elevations aligned approximately along 5 sections

covering the whole excavation front and surveyed with a total station providing the coordinates of each target (East, North and Elevation).
The data acquired with this system suggested a general trend of movements directed prevalently toward the river and attributed to the rock mass distressing induced by the excavation works. This interpretation was strengthened by the visual observation of a 20 m long crack in the internal part of the berm at 700 m a.s.l. and fissures (most probably related to K2 joint set opening) on the skin-shotcrete of the manifold galleries.

The aim of the upgraded monitoring system was to control, with a better accuracy, the slope movements over a period including at least one rainy season and also to verify the hypothesis assumed for the design of the additional supports which, in consideration of the importance of the works and the low quality of the data available, were very conservative.

Consequently, as part of the Power House slope additional interventions, the monitoring system was upgraded introducing additional instruments with the following objectives:

- large scale monitoring of the PH excavation front;
- monitoring of the crack in the internal part of the berm at el. 700 m a.s.l.;
- monitoring of pillar rock joints.

3.2 Large scale monitoring of the PH excavation front

The large scale monitoring of the PH excavation front was carried out by installing a system of 25 prisms on the excavation front. These prisms were surveyed with triangulation methodology using 3 fixed benchmarks built around the PH slope. This system was conceived to be quickly implemented on site in order to have, in a short time (covering the lapse of time until the full operation of IBIS radar), more precise and reliable displacement readings (compared to those from the targets) having also an estimate of the reading’s accuracy.

The IBIS radar was installed in an allocated building in front of the Power House slope at a distance of about 200 m. The system consists in a Stepped-Frequency Continuous Wave (SF-CW) coherent radar with Synthetic Aperture Radar (SAR) and Interferometric capabilities, dedicated to the remote monitoring of terrain (landslides, unstable slopes, glaciers, etc.) or structures (dam, bridge, etc.). The general principle of microwave interferometry is the detection of amplitude and phase of an electromagnetic wave transmitted by a sensor and reflected by the object illuminated by the antenna beam. The IBIS apparatus presents great advantages: remote sensing, simultaneous monitoring of all targets within the beam, independence from daylight and weather conditions, high accuracy and spatial resolution. The main limitations are related to the phase ambiguity (which limits the maximum measurable rate of displacement) and to the direction of displacement (line of sight only).

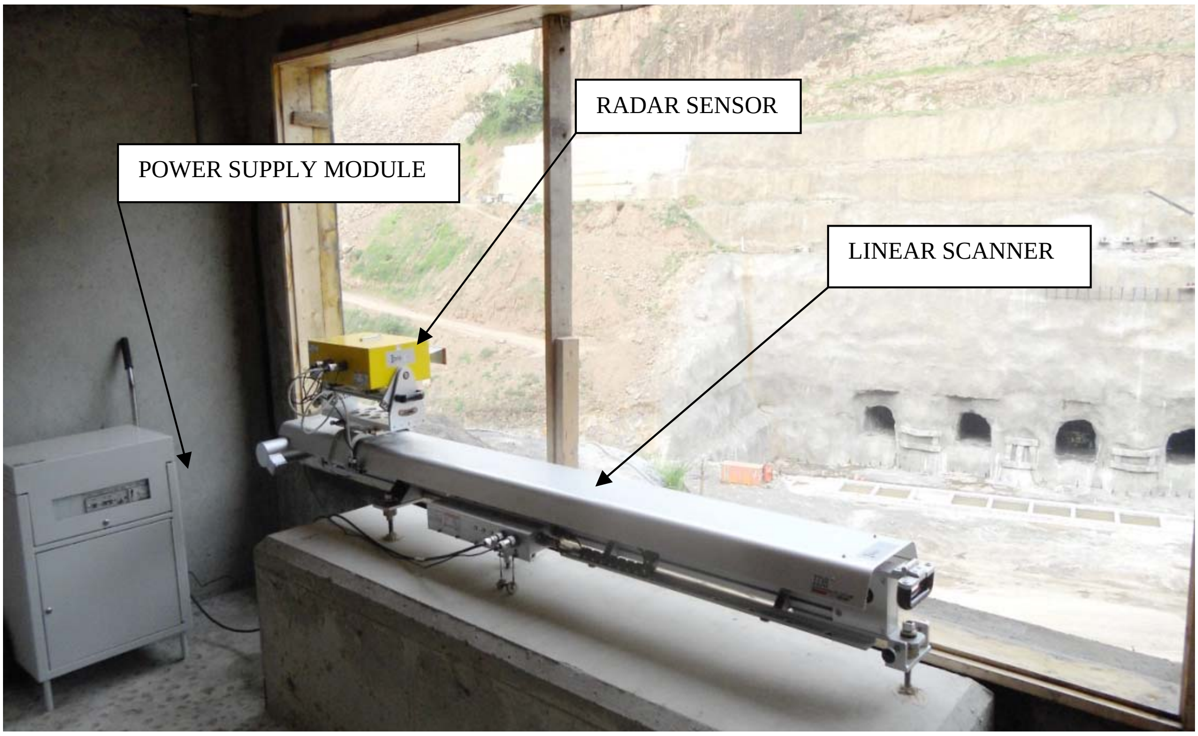

Fig. 5. IBIS radar apparatus

3.3 Monitoring of the crack in the internal part of the berm at el. 700 m a.s.l.;

The opening of the crack at the 700 m a.s.l. berm was monitored with a Laser Distance Meter (Leica DISTOTM D3), measuring the distance between 15 couples of fixed points positioned at the turn of the crack and spaced 5 m. One point was materialized by an expansion nail fixed on the internal part of the berm. The other one, on the side of the excavation, was the instrument fixed base (steel pillar). Two readings every day (before and after blasting) have been collected during the whole monitoring period.

3.4 Pillar rock joints monitoring

The opening of the rock joints in the manifold galleries were monitored by means of a centesimal comparator (deformometer) and fixed clamps installed at the turn of the rock joints. No. 16 reading positions have been selected inside the manifold galleries based on the skin-shotcrete crack mapping data.

3.5 Monitoring results

The analysis of the data obtained with the upgraded monitoring system (more than a year and half of monitoring) indicated no opening of the pillar rock joints and of the tension crack at 700 m a.s.l. (which possibly was a pre-existent joint of the K2 set brought to light by the excavation works).

The prism measurement system, albeit an improvement in comparison with the pre-existent target measurement system, was still not accurate enough to evaluate the large scale slope movements.
Reading fluctuations were probably due to the geometry of the three benchmarks (dictated by accessibility constraints) which caused non-optimal angles of intersection between the different survey lines. Moreover the visibility between each prism and the three benchmarks was not perfect (the installed prisms were not of the “360 deg” type) and the prism mirrors were frequently dirtied by dust from the excavation works.

IBIS software provides coloured maps of cumulative displacements (or velocity of displacements) between 2 dates at choice draped on 3D digital terrain model. In order to obtain TIME vs DISPLACEMENTS graphs, a number of control points were selected on the illuminated area. The displacement time histories of each control point were extracted from the slope displacement software database.

Monitoring of the displacement of the Power House excavation front by means of IBIS interferometric radar gave the following results:

- no evidence of a general trend of movements of the whole PH slope face (or part of it):
  - more than 60 % of the control points showed cumulative displacements of less than ±5 mm in 1 year;
  - almost 90 % of the control points showed cumulative displacements of less than ±15 mm in 1 year;
- long-term fluctuations (regressive displacements over a period of some weeks) and short-term fluctuations (regressive displacements over a period of days or hours) were observed in some control points;
- no correlation between displacements and/or fluctuations with blasting, rain and temperature was observed;
- larger displacements (and also larger fluctuations) were concentrated mainly in the control points above the berm at 700 m a.s.l. and close to units L_5 and L_4 where IBIS measurements were particularly disturbed by crane activities and the start of concreting respectively.

Visual inspections of the PH front carried out periodically in those areas where IBIS showed some displacements/fluctuations did not indicate any sign of movement.

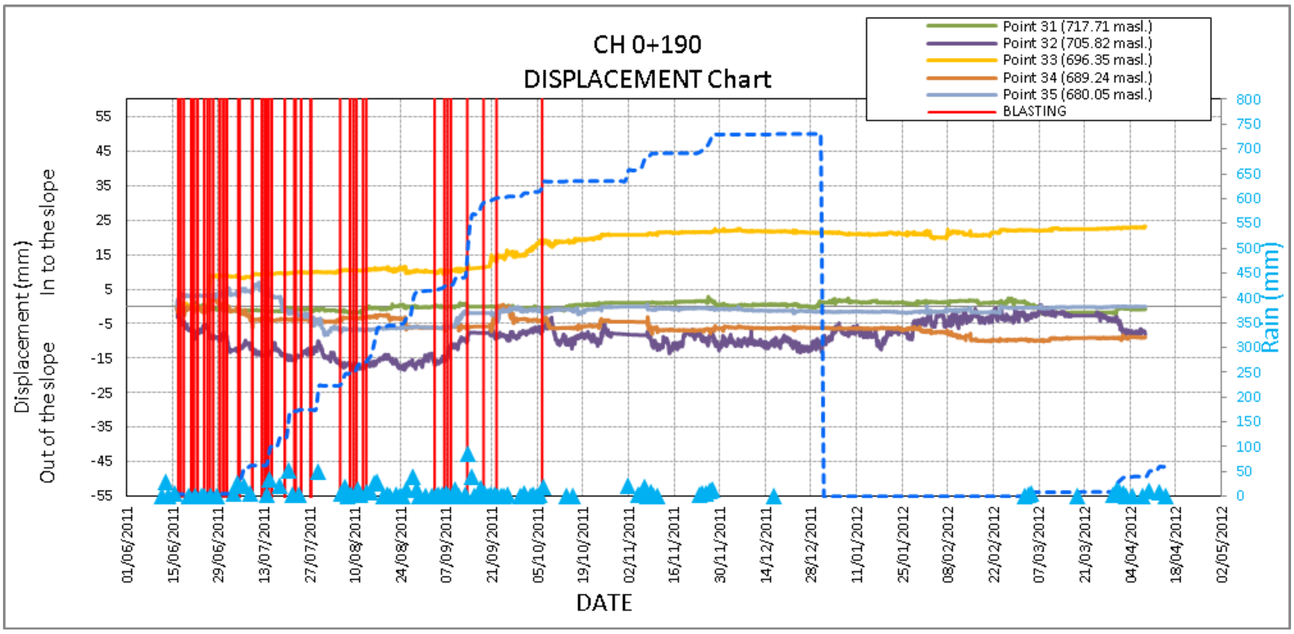

Fig. 6. IBIS monitoring. Displacement vs Time (control points located at ch 0+190)

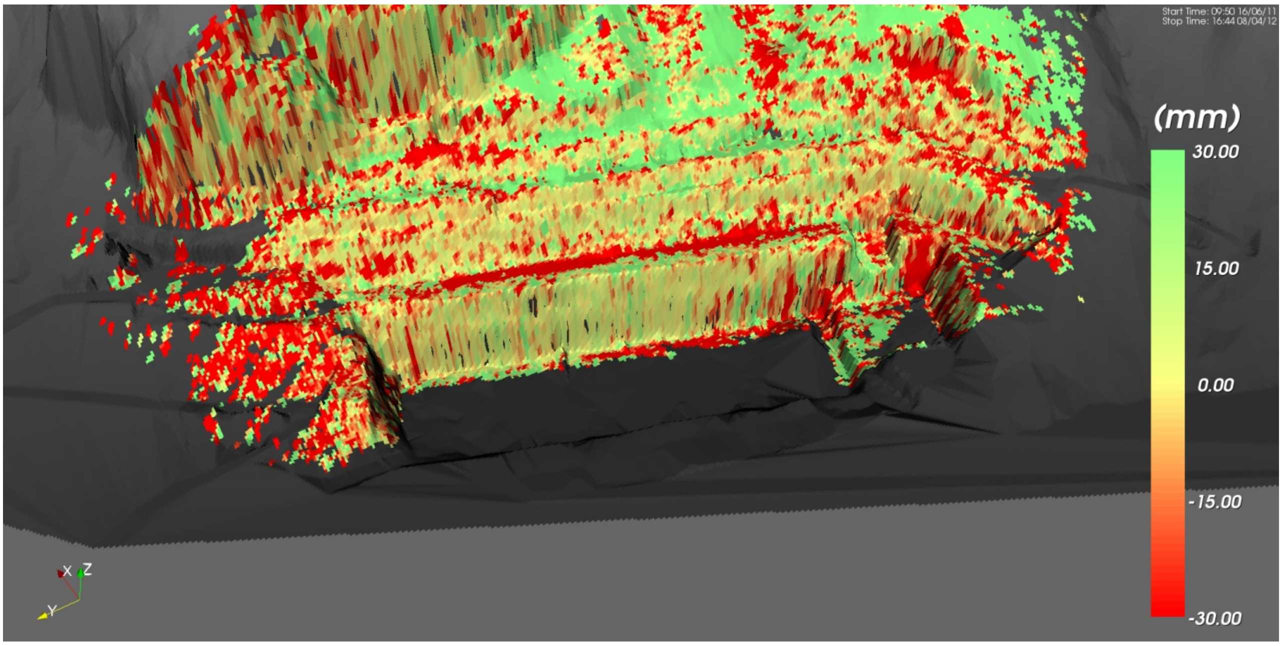

Fig. 7. IBIS monitoring. Axonometric view of the cumulative displacement of the PH slope in the whole monitoring period

4. Summary and conclusions

The PH slope observations gathered during the execution of excavation and support works suggested a possible general trend of slope movements directed prevalently toward the river and attributed to the rock mass distressing induced by the excavation works.

As a result of these observations the following actions were taken:

- temporary suspension of the excavation works;
- execution of additional support works on the excavation front;
- execution of additional support works on the rock pillars between the manifold galleries;
- upgrading of the available monitoring system in order to check:
  - upper berm tension crack opening (distancemeter)
  - pillar rock joints opening (deformometers)

- large scale displacements of PH slope (prisms, IBIS radar)

The excavation works were progressively resumed during the installation of the additional supports implementing an optimized excavation sequence.

The analysis of the data so far obtained with the upgraded monitoring system (over 18 months of monitoring) has showed:

- no opening of the pillar rock joints;
- no opening of the tension crack at 700 m a.s.l. berm;
- no evidence of a general trend of movements of the whole PH slope face (or part of it).

Consequently, considering also that during the monitoring period the stability boundary conditions of the slope were certainly worse than those expected at the end of the works (i.e. excavation fully open, blasting on course, no concreting inside the manifold galleries, no PH structure at the toe of the slope), the stability conditions of the excavation front are considered to be satisfactory and installation of further systematic supports has been excluded.

A slope monitoring system must be part of the design of slope excavation and stabilization works. The monitoring system must be custom designed taking into account the geological and geotechnical peculiarities of the site affected by the excavation works and with the objective of verifying both the hypothesis assumed for the slope stability calculations and the performance of the support works.
The IBIS interferometric radar has given satisfactory results for the monitoring of the whole excavation front. The main advantages of this system are: remote sensing, simultaneous monitoring of all targets within the beam; independence from daylight and weather conditions; high accuracy and spatial resolution. The main limitations are related to the phase ambiguity (which limits the maximum measurable rate of displacement) and to the direction of displacement (line of sight only).

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