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VERNAGO RESERVOIR SOUTHERN SLOPE STABILIZATION AND OPERATION SHAFT REINFORCEMENT

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ABSTRACT

In the 60's, after the initial impounds of Vernago Dam in Senales Valley (Bozen-Italy), some relevant deformations of the southern slope of the reservoir were observed, involving the detritic-glacial soils, where the intakes of the outlet and diversion tunnels and the upper part of the operation shaft are located.
Such phenomena originated some structural damages in the shaft, located 250 m upstream of the dam, obliging in the 80's to construct an interior lining in reinforced concrete and to connect the structure to the rock with a system of horizontal and vertical cables.
At the end of the 90's, as the deformations continued, studies and investigations have been developed in order to better understand the phenomena and consequently to define the design of the stabilization works.
At the same time the control instruments network was extended and improved.
The paper refers about the stratigraphic and geotechnical reconstruction of the slope soils and the results of the investigations on the operation shaft structures and the reinforcing cables.
The overall stability of the slope and several numerical models developed in order to reproduce the deformations are then illustrated. The results of the studies suggested the possible solutions.
The different examined options and the adopted scheme, with its chronological execution during short exceptional drawdowns in winters 2001-2003, are finally presented.
The carried out solution consists of the withdrawal of about 30-40 m of the intakes inside the reservoir, partially executed with precast elements, and of the reinforcement of the slope with an embankment, about 250 m long, around the operation shaft.
The structure of this shaft was also strengthened with an interior steel lining 18 mm thick.
The measures after the works show a reduction of the deformations, even if the adjustment of the slope and of the structures to the new configuration is still in act.

RESERVOIR FEATURES

Vernago Reservoir (see Fig.1) is a seasonal impound with hydroelectric use which has been realized by A.E. S.p.A. (Azienda Energetica di Bolzano) in ′50s and ′60s by the construction of an embankment dam with silty core in High Senales Valley (Bozen).
The dam has been built in two phases and is 65 m high, with a capacity of 41,7 hm ^{3} and maximum water level at 1.692,0 m a.s.l. . The outlet and the diversion tunnels are located in the southern slope. They consist of the bottom outlet, of the half-bottom outlet, from which the diversion starts, and of the morning glory spillway. The regulation is performed through a system of accessible gates which is located at the base of an operation shaft. This last structure is founded in rock, but its central and upper parts until the crest elevation have been dug in soils for a depth of more than 30 m. The operation shaft is in massive concrete, only

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partially reinforced. It consists of three connected elements, developed on all its height, respectively for the gates, for the trash rack and for the staircases.

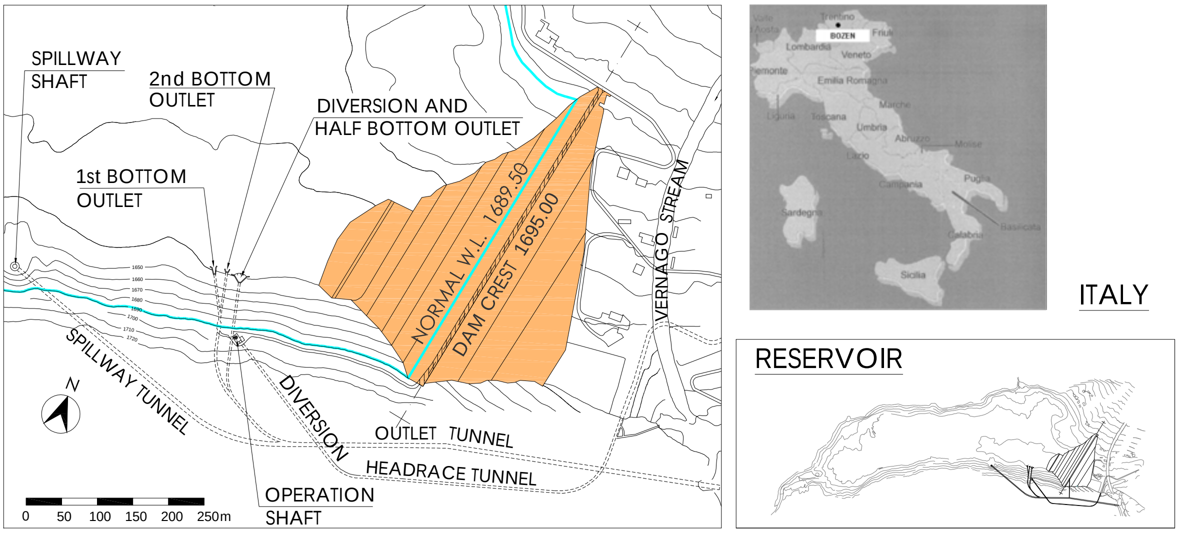

Fig.1 –Vernago Dam and Reservoir (initial situation)

DEFORMATION PHENOMENA AND CONSEQUENT ACTIONS

In 1965, following the conclusion of the dam construction, the filling operation of the reservoir started. In 1968 the normal water level at 1689.5 m a.s.l. was achieved and the spillway shaft was tested. It nevermore operated afterwards.
Already in the first '70s it was necessary to pay particular attention to the southern slope behaviour because relevant displacements were observed at the top of the operation shaft, due to a progressive rotation of its axis, and some cracks were noted inside it.
The shaft was immediately subjected to specific controls, monitoring, and extraordinary maintenance: in particular an inverted pendulum was installed in the staircases section of the shaft (1971).
In 1978 a systematic levelling along two alignments on the southern slope was started and in 1981 six deep inclinometers were realized along the access road to the operation house on the top of the shaft and on the slope behind the structure. In this phase the control network was completed with the installation of two open pipe piezometers and two Casagrande piezometers.
In 1983-'84, after the seam of the main cracks around 1655-1661 m a.s.l., the internal sides of the gates section were strengthened with a reinforced concrete lining (35cm thick), connected with the existent walls by ϕ20mm steel bars. Furthermore, an anchorage system formed by 56 sub-horizontal cables connecting the structure with the bedrock was installed. At the same time 8 vertical cables between the top and the bottom of the shaft, to produce an internal compressive stress, and 7 sub-vertical ground anchors from the top, to oppose the rotation trend, were executed. The technical characteristics of the single elements were: cable with 7 strands 0.6 inches, $L_{tot.}$ ~35m average, $L_{foundat.}$~8m, $\phi_{foundat.}$=120 mm, $N_{nominal\ load}$=1050 kN.
In 1986 three multipoint extensimeters, anchored in the bedrock, were installed in the gates section of the shaft, too.
The above mentioned activities reduced, but did not nullify the deformation trend, that was still present in the '90s, starting to concern the integrity of the shaft structure.
During a spot test of the cables conditions in 1998 a substantial loss of tension in part of them was discovered and attributed to the corrosion of the lock wedges.

For these reasons A.E. S.p.A. decided to intensify the studies with the technical support of Geotecna Progetti Consulting Engineers. The complex of investigations, measures and numerical analyses, developed in a quite long period, have permitted to reach useful conclusions concerning the interpretations of the phenomena which interested the southern slope and the shaft, and to establish the criteria to adopt in order to carry on a regular and safe plant operation. The activities to stabilize the system were consequently identified, designed and realized in the period 2000-2005.

DETAIL INVESTIGATIONS

The investigations have been essentially oriented:

1. to identify the stratigraphy and lithology of the slope and to define the geotechnical characteristics of the soils laying on the bedrock, in the area of the shaft;
2. to verify the structural conditions of the shaft.

Slope investigations

Bedrock structure

The investigations and studies carried out in the higher parts of the slope (until about 2500 m a.s.l.), where the rock diffusely crops out, have allowed since 1999 to improve the knowledge of the rock mass structure, in relation with the possible existence of deep seated gravitational deformation phenomena with slow evolution. This research activity was executed after some natural trenches, potential indicators of these deep gravitational movements, were discovered near the top of the slope.

Anyway the investigations pointed out that the main trenches are not genetically connected with gravitational phenomena of the slope, but rather with the presence of old high to medium angle tectonic faults, along which the trenches developed.

Therefore, the actual morphology of the trenches probably reflects subsequent relaxation phenomena caused by the melting process of the quaternary glacier of the Senales Valley, which has produced progressive opening of the trenches.

Some investigations, such as the stratigraphic-structural analysis of the filling deposits of the trenches, the radiocarbon dating of some organic levels discovered in these deposits and the topographic monitoring of the width of the trenches, have assured that these phenomena are exhausted since long time (no deformation has occurred in the last 3000 years): they prove the achievement of a substantial equilibrium of the rocky slope.

Reconstruction of the slope stratigraphy

The reconstruction of the slope stratigraphy between the bottom of the reservoir and the level 1790 m a.s.l. is shown in the vertical section (see Fig.2), located in proximity of the operation shaft and traced along the maximum slope line. The section results from the elaboration of all the investigations carried out in the area, including both direct type investigations as geologic surveys, boreholes with soil samples collection, geotechnical tests and laboratory testing, and indirect ones, as seismic refraction/reflection surveys and borehole sonic logs.

The investigations have allowed to recognize four main stratigraphic *units*: the quaternary deposits (three units) and the bedrock (one unit). This partition of the subsoil has been well confirmed by the seismic investigations executed near the operation shaft, where the seismic velocity has been measured along sections oriented both parallel and orthogonal to the contour lines of the slope.

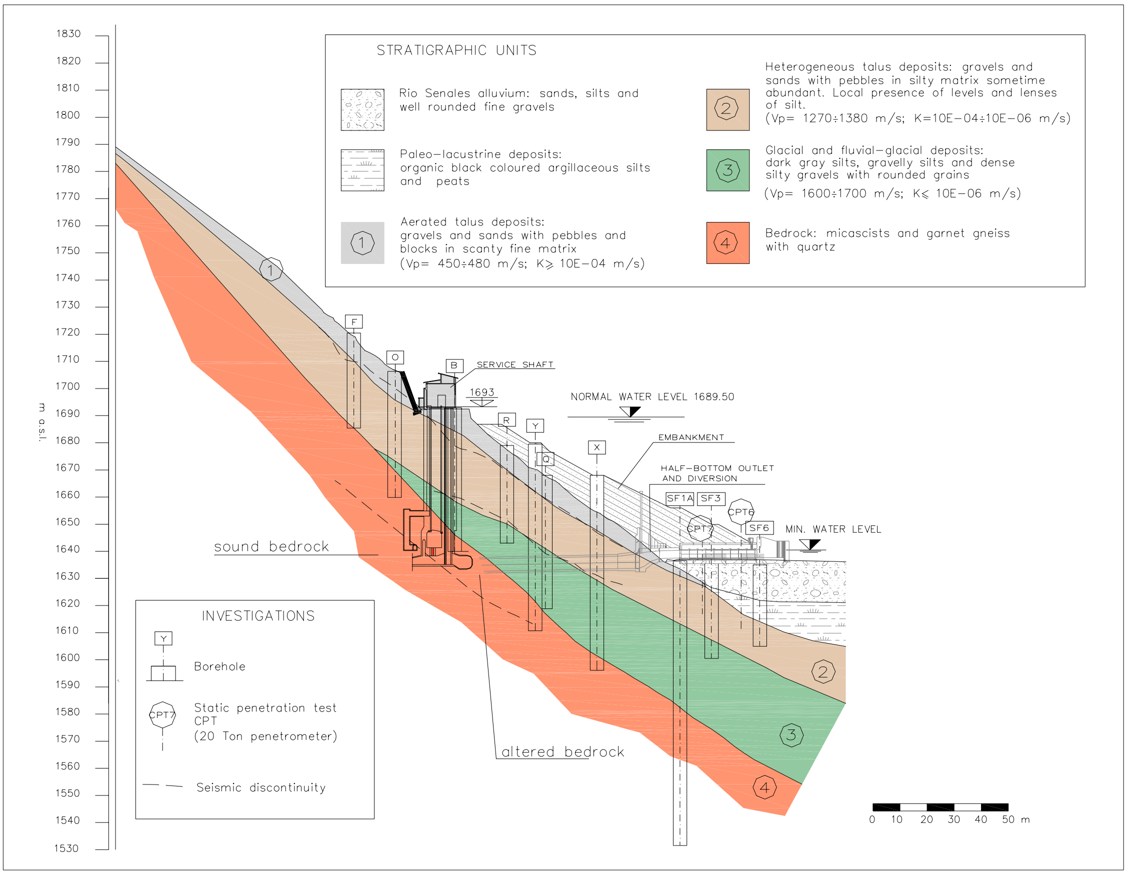

Fig. 2 – Geological section of the slope corresponding with the operation shaft area

Lithological and geotechnical characterization of the stratigraphic units

The main units which have been recognized on the slope, from the surface to the depth, are here below described:

Quaternary Unit 1) – *aerated talus material* (5÷7 m thick). It is predominantly constituted by coarse-grained deposits (gravels, sands and pebbles) and rough-edge blocks with fine matrix, which varies from scanty to absent.

$V_{p\ \text{seismic velocity}}$ ~ 450 m/s. Shear strength parameters: c’=0 kPa, φ’= 41°;

Quaternary Unit 2) – *heterogeneous talus material* (~15÷20 m thick). It is essentially constituted by gravelly-sandy deposits with fine matrix, which vary from scanty to locally abundant. It is relatively more dense in comparison with the previous unit 1. Fine sediment lenses (max ~ 1 m thick), made of fine silty and silty sand sediments, are present.

$V_{p\ \text{seismic velocity}}$ = 1270÷1300 m/s. Shear strength parameters: c’=0 kPa, φ’= 38°;

Quaternary Unit 3) – *glacial and fluvial-glacial deposits* (~15÷20 m thick). It includes silty gravels, gravelly and sandy silts and, locally, well cemented gravels. The fine fraction is always abundant. Such deposits are a very heterogeneous mix, both in vertical and horizontal direction.

$V_{p\ \text{seismic velocity}}$ = 1600÷1700 m/s. Shear strength parameters: c’=0 kPa, φ’= 35°;

The permeability of the quaternary units is strongly affected by the marked heterogeneity of the granulometry of the deposits and it influences the flow of the saturation water in the body of the slope which originates during the reservoir drawdown.

It is identified, from the surface to the depth, a first coarse layer (*unit 1)*, which is surely

characterized by a high to medium permeability and which is not able to retain the saturation waters.
Below this layer the following *unit 2* is characterized by a coarse component associated with a rich silty-sandy fraction, included sporadic fine silty-sandy lenses, that determines a lower average permeability ($k=10^{-4}\div10^{-6}$ m/s). In sudden and occasional drawdown conditions, the water flowing in the body of this layer can locally cross these lenses inducing a delay of the time of dissipation of the pore pressures, in comparison with the behaviour of the reservoir in normal conditions of drawdown.
In addition, this unit, being formed by not very compacted talus soil, is subject to deformation phenomena which can, locally and occasionally, increase the pore pressures inside the not-permeable lenses.
The permeability of the *unit 3*, which is characterized by abundant and compacted fine fraction, especially the part lying directly on the bedrock, is estimated to be low ($k\leq10^{-6}$ m/s). In condition of very sudden drawdown the water level inside this material can remain at a higher level than the reservoir.

Bedrock Unit 4) – The bedrock, constituting the overall southern slope of the Vernago Reservoir, is represented by metamorphic rocks as micaschists, gneissic micaschists and gneiss with quartz and garnets, characterized by a well developed vertical schistosity, mainly oriented parallel to the axis of the Vernago valley.
The bedrock shows, especially in the operation shaft area, a 10÷20 m thick superficial band formed by altered and very fractured rocks ($V_{p\ \text{seismic velocity}}$ ~ 2500÷2600 m/s), which overlay the unaltered rock mass formation, where the seismic velocities highly increase.
On the whole the bedrock formation consists of high strength intact rocks. At the rock mass scale, however, its geomechanical behaviour seems to be very influenced by the presence of many discontinuities of various type and origin, as the well developed schistosity and some sets of joints and faults.
In conclusion the bedrock formation is not involved in the deformation phenomena affecting the slope.

Shaft Investigations

During the years 1998-2000 appropriate investigations were carried out to verify the structural consistency of the shaft and of the cables.
It was confirmed that the shaft is composed by three concrete hollow elements which are in contact along the vertical sides, but structurally separate: the staircases shaft (concrete), the gates shaft (concrete) and the trash rack shaft (weak reinforced concrete) elements (see Fig.3)

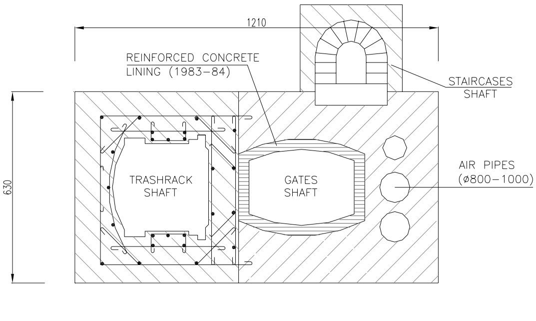

Fig. 3 – Shaft – Horizontal cross section

Shaft - Firstly the survey and the updating of the cracks situation in the gates and trash rack shafts was developed. In the gates shaft some drillings were performed and laboratory tests on cylindrical concrete samples from the walls were carried out. Only not destructive tests, such as georadar and concrete test hammer, were possible on the trash rack shaft walls. The results pointed out a considerable difference between the concrete cubic strength of the trash rack ($R_{ck} \approx 20 \div 25$ MPa, sufficient) and that of the gates shaft ($R_{ck} \approx 12 \div 15$ MPa, weak).

Ground anchors - All the sub-horizontal cables were checked and re-tensioned as diffused losses of tension were verified.

Monitoring improvement

During the site investigations in 1998-2003 some inclinometers and piezometers were installed in order to increase the monitoring network, taking into account also the slope area between the dam and the spillway shaft.
Furthermore two vertical TDR system indicators along the slope section corresponding with the operation shaft were placed on the reinforcement embankment at different levels and some benchmarks were positioned at the new intakes of the bottom and diversion/half bottom tunnels.
In 2000 during the testing and re-tensioning operations of the ground anchors 7 of them were provided of pressure cells and data acquisition system.
Recently also the acquisition system of the main control instruments has been centralized and automatized.

SLOPE STABILITY AND STRESS-STRAIN ANALYSES

Stability of the quaternary deposits

On the basis of the acquired elements an up-to-date slope stability analysis has been elaborated. For the zone between the dam and the shaft the values of the safety coefficient were acceptable. On the contrary they resulted lower in the proximity of the shaft (≈1.1, see Fig.4). The presence indeed of the rigid body of the shaft played a stabilizing effect on the slope movement, inducing however big stresses in the structure.

The results of the investigations and of the analyses confirmed the stability of the bedrock. Nevertheless they required to find an adequate solution in order to improve the stability condition of the quaternary deposits around the shaft and in the zone of the outlet /diversion intakes. The most convenient solution was judged to reinforce the existent slope with an appropriate stabilization embankment.
In this way a significant increase of the security factor would have been obtained.

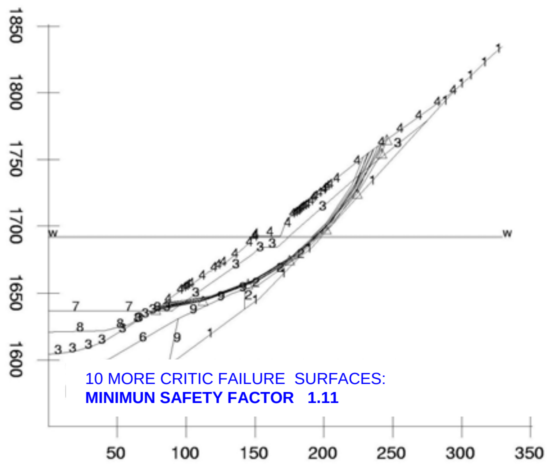

Fig. 4 – Southern slope close to the operation shaft - Stability in maximum water level conditions

Quaternary deposits stress-strain numerical models

In order to understand the deformation phenomena which interest the quaternary deposits, several numerical models of the slope around the shaft were developed during the years 1999-2003.
In the first group of these analyses, assuming a viscous behaviour of the upper detritic and fluvial-glacial layers, the shaft was sketched in the first crack conditions and the following contribution of the ground anchors installation (1983-84) was considered, too. The anchors forces were also reduced to take into account the results of the 1998 check. These analyses considered the water in the reservoir next to the normal level.
A following scenario considered both the new stabilization embankment and the cables re-tensioning effects. In this situation the obtained displacements were reduced (≈-50%).

A second group of analyses (see Fig.5) based on the elasto-plastic behaviour of the quaternary deposits and on the presence of pore pressures in the upper detritic and fluvial-glacial layers during the drawdown phase, was executted. The contribution of the shaft elements was not considered in the model.
In particular the assumptions concerning the pore pressure were justified in analogy with the classical behaviour of slope stability in conditions of rapid drawdown, as in winters 2001 and 2002 really occurred.

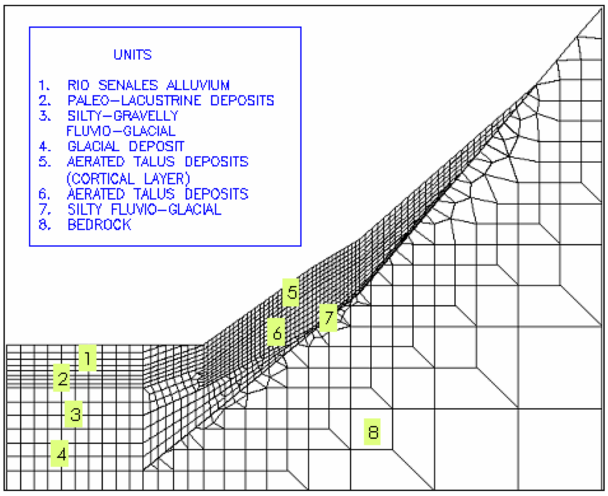

Fig. 5 – Southern slope in proximity of the operation shaft – Finite elements model mesh

The following two “water load conditions” have been considered:

1. superficial water table;
2. deep water table.

The first one appeared to be too severe. The second gave an interesting result: the local displacements obtained at the same level of the highest station of the inverted pendulum were similar to those recorded in 1988-1992, first representative period after the installation of the ground anchors (1983-84).
As the stabilizing embankment was added in the mesh, the analysis showed a progressive reduction of the deformation velocity.
The results of the calculations gave useful elements to understand the phenomena. Unfortunately, despite the numerous local investigations, it was not possible to measure the pore pressure variation in the quaternary deposits in relation with the filling and, especially, the drawdown velocity.
Even so it was possible to draw from this analysis the main lines of the future practical actions.

CHOICE OF THE STABILIZING SCHEME

The research of the best solution was oriented to assure both the safe exercise of the reservoir, without interruptions, and the regular operation of the bottom outlet and of the half-bottom outlet-diversion. For this reason it was necessary to warren the security of the tunnels intakes and of the operation shaft.
Moreover, in order to minimize the emptying of the reservoir, the works had to be realized during the short winter period of possible exceptional drawdown.
Due to economical considerations, the scheme with the extension towards the interior of the reservoir of the existent outlet and diversion tunnels was chosen.
The two alternative solutions, i.e. the displacement towards West of the same tunnels or the construction of new tunnels on the opposite side of the valley, were abandoned as very expensive and owing to the long time requested.

DESCRIPTION OF THE WORKS

In the 2000 summer the ground anchors installed in 1983-84 were tested and re-tensioned. During these operations it was observed that the part of the strands immediately under the head was highly corroded.
At the same time the final design, concerning the following activities to be executed in the short and rigid winter period of reservoir emptying, was developed:

- upstream extension, artificially, of the bottom and half-bottom-diversion tunnels, with partial prefabrication of their structural elements and displacement of the intakes 30-40 m inside the reservoir;
- realization of the stabilization embankment to reinforce the slope in the shaft zone;

After a careful examination of the final design, at the end of December 2000, the Servizio Nazionale Dighe (Dams Security National Office) authorized the reservoir exceptional emptying from February to May 2001, in order to verify the integrity of the outlet and diversion tunnels, to investigate the foundation ground at the lake bottom and, once obtained positive results, to start the works.
As the tunnels appeared in good conditions, this first operative phase allowed to extend the soils knowledge with the identification of some peat and organic silt strata in the zone of the tunnels extension.
In a very short time, a treatment of these strata was planned and organized through the realization of a net of vertical columns formed by soil consolidated with bi-fluid high pressure injections technology (jet grouting).
The treatment affected a whole area of 2400 m^2 and it was executed with 20 m deep columns of 1.5 m diameter, spaced 2.3÷3 m.
Cement mixtures were used, composed by cement 32.5 with a ratio cement/water 1.5/1, adopting a minimum mixture consumption of about 1000 kg per column meter.
The consolidated ground characteristics and the actual columns diameter was verified during and at the end of the injection operations through the execution of vertical core sampling along the column axis, at a variable distances from it and at the barycentre of some groups of three columns realized for the purpose. During these activities undisturbed samplings were collected to be tested in laboratory.
The tests results have confirmed the predicted minimum column diameter of 1.5 m and the achievement of the requested mono-axial compression strength of the treated ground (σ_{ave} > 800 kPa). The obtained stress-strain modulus was greater than 75000 kPa.

As the updated design was finally approved in 2002, the fulfillment of the works was executed during a new reservoir emptying in the first months of 2002 and an exceptional drawdown in 2003 spring.
The gates of the bottom outlet and of the half bottom-diversion tunnels were contextually substituted.
Between January and June 2002 the extensions of the tunnels were realized together with the new stabilizing embankment until the elevation 1653 m a.s.l. In addition a operation road was prepared on the riprap of the dam at 1656 m a.s.l., in order to supply the construction site despite an increase of the water level in the reservoir.
The extension tunnels (bottom outlet 54.1 m and half bottom outlet-diversion 53.3 m long) are respectively constituted by single and double cast in place reinforced concrete (r.c.) elements (walls minimum thickness 80 cm) carried out to protect the inside pre-cast r.c. elements which constitute the real hydraulic culverts. The main typical internal dimensions of the cast in place tunnel sections are 6m(width) x 4.45m(min. height) x 6m(max. length). Each pre-cast culvert, 3m x 3m x 2.5m(length) and with minimum thickness 25 cm, is composed by two “U” shape elements tied with 12 vertical ϕ26mm Dywidag bolts (835/1035 MPa). They are placed on special plastic concrete in order to absorb and reduce the movements due to the settlements of the external tunnel elements. The double tunnel section is shown, as an example, in Fig.6.

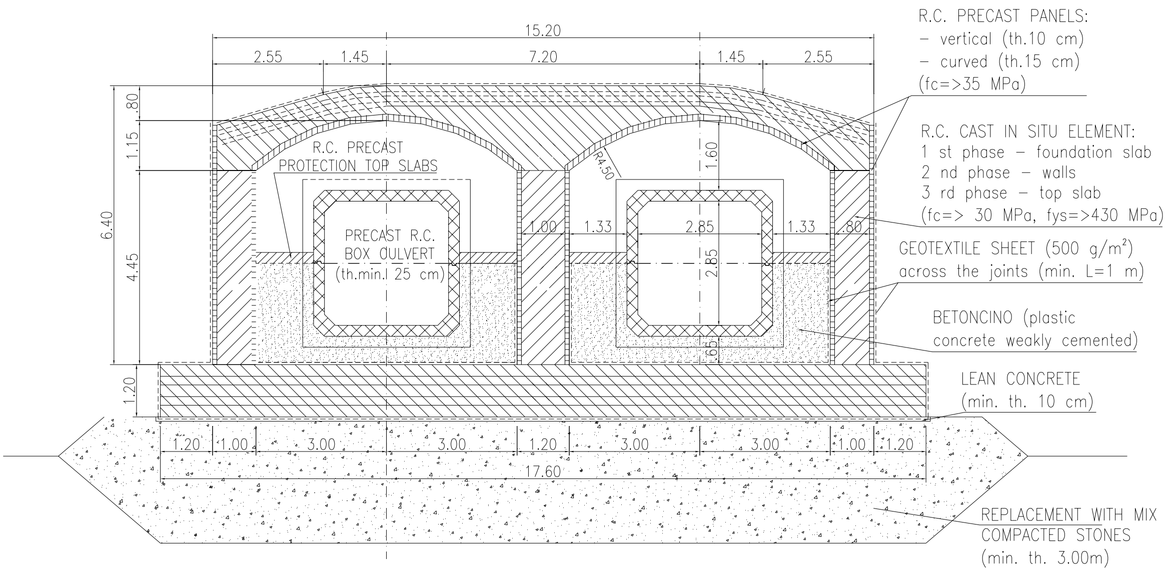

Fig. 6 – Double tunnel – Typical cross section

The longitudinal profile of the new half bottom outlet-diversion tunnel (the bottom outlet is similar) is composed by the new intake structure, the tunnel extension and the two joint elements, which realize the transition between the new double tunnels and the existent single one (see Fig.7).
The part placed immediately before the existent intake, in which the chute is located, has been opportunely protected by a steel armouring 12 mm thick.

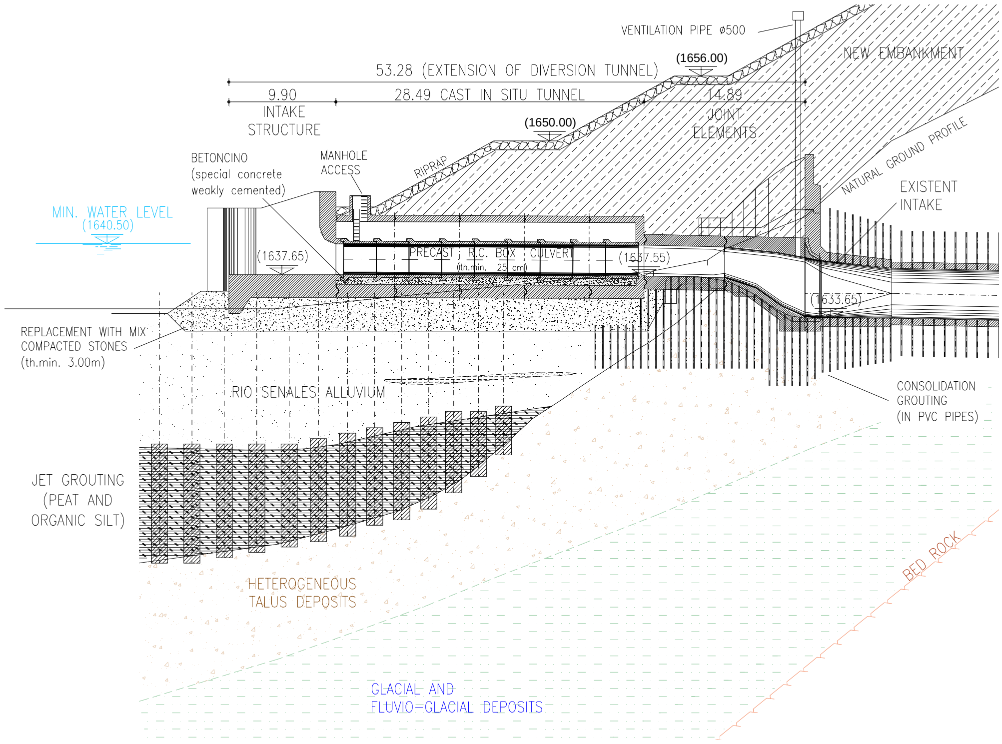

Fig. 7 – Half bottom outlet-diversion tunnel – Longitudinal profile

On the 3rd of April 2003, after the first filling, a technical inspection of the completed tunnels extension works was executed with monitoring data collection. The obtained positive results allowed to complete the stabilizing embankment (see Fig.8) before the 2003 summer.

Fig. 8 – Southern slope – Stabilizing embankment

Furthermore between the end of 2004 and June 2005 the following scheduled works were realized in the gates section of the operation shaft and around it:

- additional steel lining formed with steel welded plates (18 mm thick, Fe 510 C), stiffened with vertical and horizontal steel profiles (12mm thick). The plates are anchored on the internal face walls with ϕ27 mm bolts (between 1649÷1673 m a.s.l.) (see Fig.9);
- external waterproofing grouting at the ground contact (between 1656÷1683 m a.s.l.).

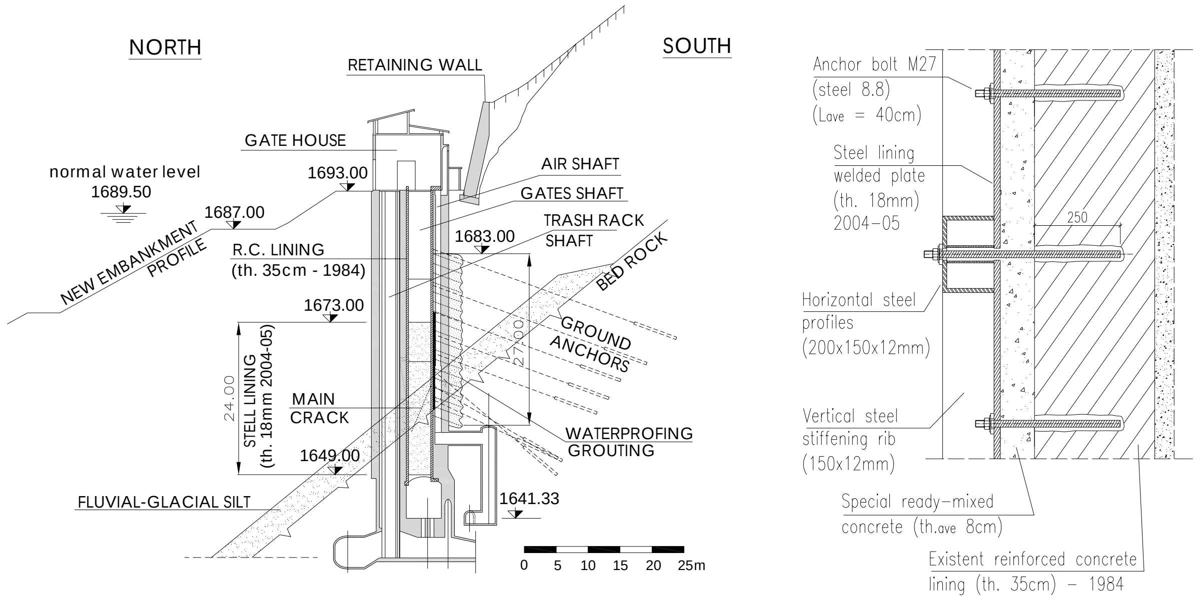

Fig. 9 – Operation shaft – Vertical section and steel lining detail

INITIAL BEHAVIOUR AFTER THE WORKS

The monitoring measurements after the conclusion of the works point out a gradual reduction of the displacements and the settlements, which are smaller than those which have characterized the phase following the first ground anchors installation (1983-84).
The slowing down is more marked in the lower part of the slope (1660÷1670 m a.s.l.), directly interested by the stabilizing embankment effects. Also in the upper part, however, where this contribution is reduced, it is possible to note a gradual decrease of the deformations (see Fig.10).

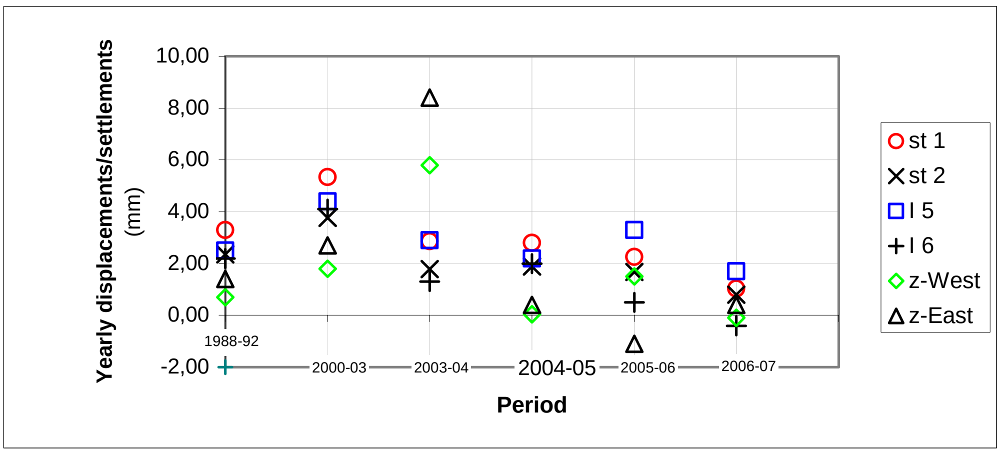

Fig. 10 – Recorded displacements and settlements

In the Fig.10 it is plotted the diagram of the average yearly displacements and settlements measured by some representative instruments in correspondence with the following main events:

- after the ground anchors installation (1988-1992);
- during the tunnels extension and the embankment construction (2000-2003);
- in the first impound-partial drawdown yearly cycles after the works.

The most significant data are those furnished by the East and West inclinometers (I5, I6), located in the slope behind the shaft, as they are less affected by the structure presence. The average values of the settlements measured in some benchmarks (z-West, z-East) of an alignment at 1720 m a.s.l. have been considered, too. Eventually the inverted pendulum displacements of the two top stations (st1, st2), the most representative, are plotted.
As the whole is it possible to see an increase of the movements during the reservoir exceptional emptying operations, due to the works execution inside the basin, and the first filling-drawdown. It is followed by a gradual decrease in agreement with the numerical models results.

PREDICTED DEVELOPMENTS

In the period 2001-'04 the load trend of the first ground anchors has recorded an accelerated deterioration with both increment of steel stress and collapse of singular strands, due especially to the local corrosion just below the head cable plate. From the data elaboration of the 7 monitored cables it has been discovered a loss of stress of about 25% with a trend to loose the anchor effects in a few years.
The problem has been studied by Geotecna Progetti in 2005-'06 and faced by A.E. S.p.A. with the approval of a program of substitution of part of the cables with 33 new similar ones. The activity has been scheduled to be developed in the first months of 2008 and 2009, in order to slow down the loss trend of the existent cables and to restore their effects. The new elements are going to be installed inside the gates shaft at 11 levels along its height. It has been established to monitor 13 cables in order to keep the structure as much as possible controlled.
Meanwhile in 2006 an extraordinary maintenance operation was operated to protect the existing cables against the corrosion below the head cable plate with the injection of special grease. A particular attention to the protection of the new cables heads is part of the substitution plan.

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REFERENCES

BALDOVIN E., BORGONOVO G., GIODA G., (2002). *Stability assessment of the slope hosting the operation shaft of a large earth dam*. - IALAD (Integrity Assessment of Large Dams) Mini-symposium on monitoring and inverse problems in dam engineering, Politecnico di Milano

BALDOVIN E., CHISSALE' L., MORELLI G., (2007). *Stabilizzazione e controllo del versante meridionale della Diga di Vernago*. - Proc. XXIII National Conference of Géotechnique, pp.131-138, Padova - Abano Terme

CROCE A., MARTINELLI D., (1978). *Il sovralzo della Diga di Vernago e le opere di fondazione durante 20 anni di esercizio*. - Proc. XXIII National Conference of Géotechnique, pp.137-145, Merano