Behavior of hydropower plant "La Yesca" Mexico, after two year of built

Comportamento da usina hidrelétrica "La Yesca" México, após dois anos de construção

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Humberto Marengo-Mogollón
Doctor of Engineering
Institution: Comisión Nacional del Agua (CONAGUA)
Address: Av. Insurgentes Sur, número 2416, Colonia Copilco El Bajo, Alcaldía Coyacán, CP. 04340, CDMEX
E-mail: hmarengom@gmail.com

Rigoberto Rivera Constantino
Doctor of Engineering
Institution: Universidad Nacional Autónoma de México (UNAM) - Facultad de Ingeniería (UDELAR)
Address: Av. Circuito Interior Universitario, S/N, Ciudad Universitaria, CP. 04510, Alcaldía de Coyoacán, CDMX, México
E-mail: riverac@unam.mx

Évert V. Hernández López
Civil Engineer
Institution: Comisión Federal de Electricidad (CFE)
Address: Av. Paseo de la Reforma, 164, Col. Juárez, C.P. 06600, Ciudad de México, México
E-mail: evert92139@gmail.com

Enrique Mena Sandoval
Maestro en Ingeniería
Institution: Comisión Federal de Electricidad (CFE)
Address: Av. Paseo de la Reforma, 164, Col. Juárez, C.P. 06600, Ciudad de México, México
E-mail: jenriquemenas@yahoo.com.mx

ABSTRACT
This paper presents the program instrumentation, monitoring and performance of Hydropower Plant “La Yesca”, which is a concrete face rockfill dam, with 207,5 m high (inaugurated on November 6, 2012). During de filling of the reservoir, high leaks where measured on the drainage gallery of the curtain and on the injection and drainage galleries, particularly on the right bank. To reduce the anomalous seepage in the galleries a series of studies were done to define the source of the leaks and implement corrective measures. Currently the embankments of the dam and the concrete face show a good behavior. They present less than 20 cm of maximum settlement from the reservoir filling and two years

of operation. The instruments placed on the joints of the slabs of concrete face show less than 15 mm in opening or settlement as result of the good performance.

Keywords: CFRD, leaks, seepage, instrumentation, performance.

RESUMO
Este documento apresenta o programa de instrumentação, monitoramento e desempenho da usina hidrelétrica "La Yesca", que é uma barragem de enrocamento com face de concreto, com 207,5 m de altura (inaugurada em 6 de novembro de 2012). Durante o enchimento do reservatório, grandes vazamentos foram medidos na galeria de drenagem da cortina e nas galerias de injeção e drenagem, particularmente na margem direita. Para reduzir a infiltração anômala nas galerias, uma série de estudos foi feita para definir a fonte dos vazamentos e implementar medidas corretivas. Atualmente, os aterros da barragem e a face de concreto apresentam um bom comportamento. Eles apresentam menos de 20 cm de assentamento máximo a partir do enchimento do reservatório e dois anos de operação. Os instrumentos colocados nas juntas das lajes da face de concreto apresentam menos de 15 mm de abertura ou assentamento, como resultado do bom desempenho.

Palavras-chave: CFRD, vazamentos, infiltração, instrumentação, desempenho.

1 INTRODUCTION

In this work, we describe the behavior of Hydropower Plant (HP) La Yesca since the finish of its construction (2 years ago), particularly at the end of the construction stage and initial filling, started in April 13, 2012. The NAMINO (EL 518 m) was reached in August the same year, and the maximum reservoir water level was reached in mid-October 2013 (EL. 574 m), a meter below the NAMO (EL. 575 m).

Using information from the instrumentation installed in the curtain and other structures part of the HP, the behavior of the embankments of the curtains is analyzed in terms of measured settlements to the end of the construction, the first filling and long-term step. Also it were measured in the hydraulic pressures generated in the embankment by reservoir filling, the deflections of the slabs and the movement of the joints between adjacent slabs of concrete face and between the concrete face and the plinth (perimeter joint). Special mention is made to the study of the phenomenon of leakage observed following the first filling and implementing remedial measures taken.

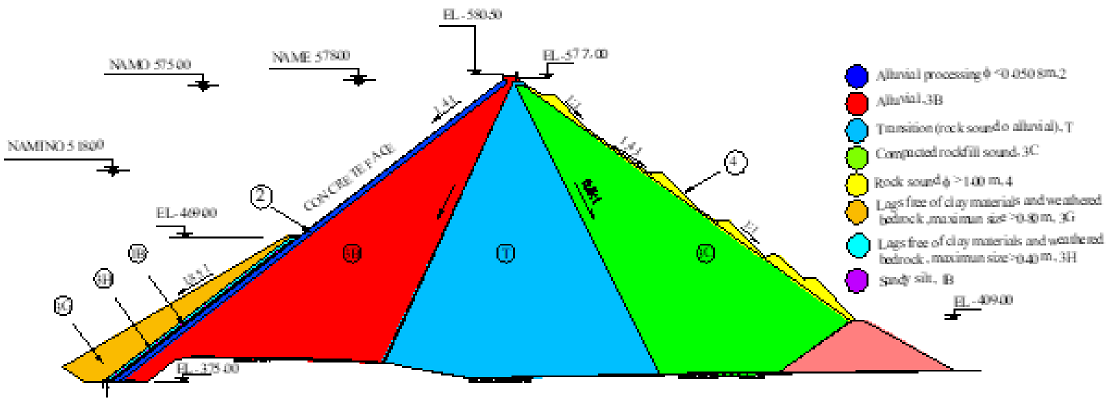
Using information derived from the instrumentation installed in the curtain and others structures that are part of the HP, the behavior of the embankments of the curtain are analyzed in terms of measured settlements to the end of the construction, the first filling and long-term step. Special mention is made to the study of the phenomenon of leakage observed following the first filling and implementing remedial measures taken.

2 OVERVIEW

The HP La Yesca is located in the Santiago River,105 km NW of the city of Guadalajara, Mexico, and 22 km NW of the small town of Hostotipaquillo in the state of Jalisco, Mexico. The dam site is located 4 km downstream at the confluence of the Bolaños and Santiago Rivers, upstream of El Cajón dam and downstream from HP Santa Rosa.

The HP La Yesca was built according the geometry and zoning shown in Figure 1, been the main structures: 1) the diversion tunnels; 2) downstream and upstream cofferdam; 3) curtain; 4) open channel spillway and 5) underground powerhouse. A general layout of the project is shown in Figure 2.

Figure 1. Maximum cross section of La Yesca Dam



Regarding the Geology, the canyon of La Yesca dam site was carved by the Santiago River through a group of Cenozoic volcanic rocks. These including andesitic rocks, highly silicified crystal litchi hyalite tuffs (Title), rhyodacitic ignimbrite with fluidal texture (Tmird) and porphyritic dacite ignimbrites (Tmid), affected by various intrusive bodies. All this lithological variety is partly covered by lacustrine deposits and pumice (Qlp), alluvial terraces (Qta), talus (Qdt) and recent alluvial deposits (Qal).

The diversion works consisted in 2 tunnels of 14 x 14 m portal section, located on the left bank of river. These were excavated in rock, covered with hydraulic concrete on the template and shotcrete both walls and roof. The tunnels are able to transit a design flood of 7 578 m^3/s. This work is complemented by two cofferdam and a board on the stream "Carrizalillo". The upstream and downstream cofferdams were constituted by an impermeable core linked to a grout curtain constructed through alluvium to bedrock bottom of the riverbed to prevent leaks affect the construction area of the curtain. The crown of the upstream cofferdam was located at elevation 435 m while the downstream elevation reached 409 m.

Figure 2. General layout of the project: 1, Diversion Tunnels, 2, Upstream Cofferdam, 3, Dam, 4, Open Channel Spillway, 5, Underground Powerhouse

The HP is equipped with two vertical Francis turbo generators 375 MW each, allowing a total annual average generation of 1,210 GWH with a plant factor of 0.19. Also it has a calling channel and two armored pressure pipes of 7.7 m of diameter and design flow of 250 m^3/s. The underground powerhouse and the oscillation gallery were excavated in rock, with dimensions of 97,5 m x 49,5 m x 67 m and 22 m x 53 m x16 m, respectively. Finally, a tunnel vent of 15 m in diameter and portal type section evicts turbinate waters.

The spillway structure is located on the left margin of the nozzle and is an open-air canal. It consists in three channels of 630 m long and 18,5 m in effective width in the control region (with six radial gates). Its design flow is 15 110 m^3/s, for a probable maximum flood of 15 915 m^3/s (return period, Tr =10 000 years).

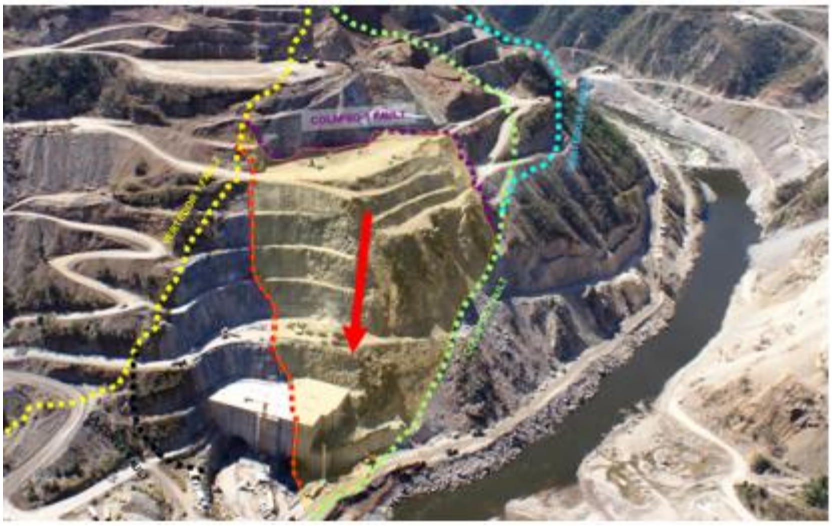
The curtain is gravel-sand type and concrete face rockfill dam (CFRD), with a crown elevation at 579 m. The total height of the dam is 207,5 m, including a parapet height of 4,5 m, the outer slopes both upstream and downstream are 1,4:1 m.

3 STABILIZATION WORKS IN THE LEFT MARGIN

In October 2007, excavations began to build open portals of the tunnels that were used to divert the Santiago River then late February and early March 2008, excavations began underground entrance gates of the two tunnels, the portals of a tunnel out and cruise, respectively. The volumes of surface excavations entry portals were the order of 400 000 m^3. This caused the slope of the left bank lost support, which combined with the vibrations induced by blasting rock for underground excavations, restarted a movement of a large block of rock through a preexisting fault plane consists of a clay layer with a thickness of 10 to 80 cm.

The instrumentation of the rock block revealed that reactivation of motion displacement reached speeds of up to 9 mm/day in its critical stages, between the months of June and July 2008, therefore the "CFE", conducted during 2009 execution a series of activities to evaluate and analyze the stability of the area mentioned. These studies allowed defining the boundaries of the unstable block bounded by faults being Collapse, Collapse 1 and Collapse 2, with an area of involvement involving Diversion Tunnels, an area of the plinth and the final closing port (Figure 3).

Figure 3. Block in unstable left margin faults delimited Collapse, Collapse 1 Collapse 2 and the granitic dike entrance portal diversion tunnels



Due of damages caused by the unstable block, the dam axis turned 12 degrees to the left bank downstream, thus eliminating this problem partially, being to ensure the stability of the rock block during construction and service life of the dam, which led to the design and construction of some stabilization works.

3.1 DESIGN AND CONSTRUCTION OF STABILIZATION STRUCTURES

The stability analysis showed that rock block to ensure minimum safety factors of the work under different conditions of actions would be subject to the dam, first during its construction and later in its life, it was necessary to implement various works of stabilization [1]. As a first measure withdrew a significant amount of material in the upper part of the block, in stages, of the order of 1 550 000 m^3. Additionally we constructed a concrete monolith at the foot of the rock block to support it, which represented a volume of 110 000 m^3. The impact of each of these works in the stability of the block is shown in Figure 4, with a speed of movement at the beginning of this of 117,4 mm/month to 6,5 mm/month at the end of the first stage material removal at the top of the block. Subsequent stages of excavation and construction of the monolith reduced movement speed values on the order of 1 mm/month, with an outspoken tendency to stabilization.

Due the rapid drawdown, water levels in the dam and the seismic stability conditions in the rock block were critical. It was necessary to design and construct additional stabilization works as the case of a wall formed by six plain concrete piers 9 m diameter each one and varying depths between 60 and 85 m (Figures 5 and 12), which crossed the fault Collapse and were embedded in the underlying bedrock. Additionally, diversion tunnels were filled with reinforced concrete in a certain length (190 m in the tunnel 1 and 175 m in the tunnel 2), at its intersection with environment collapse failure, to make them work as a large anchor (Figures 5 and 12).

For each of these structures analysis and design models were developed, due it was unconventional structures, this allowed their construction on the site. Finally, these structures were implemented properly to keep track of their behavior during the stages of construction and life of the dam. In the next chapter, we analyze the results from the monitoring carried out for each of the stabilization structures and discuss its performance.

Figure 4. Evolution of unstable block displacement a function of time in terms of the effectiveness of stabilization works.

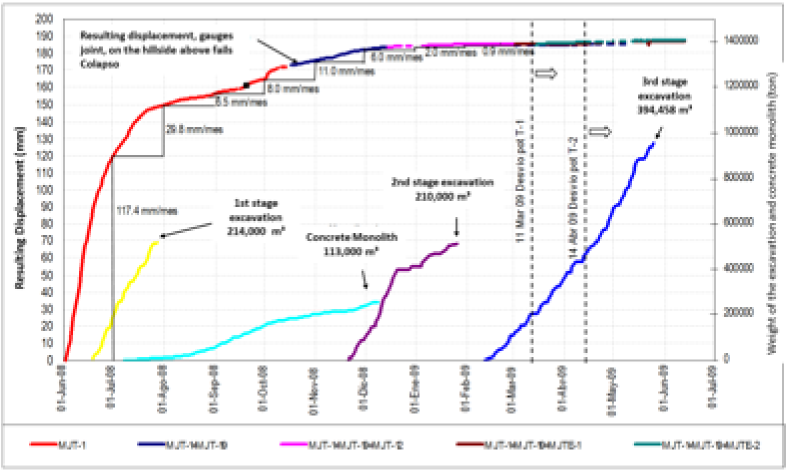
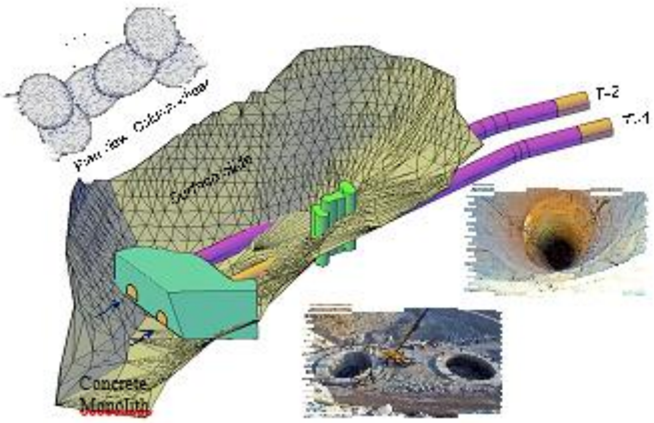


Figure 5. Implemented stabilization structures: concrete monolith; anchor plugs diversion tunnels 1 and 2 and shear-piers.



4 INSTRUMENTATION, MONITORING AND PERFORMANCE OF HP LA YESCA

Auscultation systems dams are intended to understand the behavior of structures, from the construction phase to verify assumptions and design criteria, material specifications to fit and placement, and over the life of the structure, to detect promptly present any anomaly. This knowledge allows to evaluate the performance and determine the security of the structures, particularly after the occurrence of extraordinary events such as earthquakes, floods or severe storms and especially to observe the long-term trend of the variables that may reveal anomalous behavior that threatens the integrity of the work.

In the case of HP La Yesca, a rockfill dam with concrete face, the main factors influencing their behavior are: a) Geological hazards, slope stability and movement of boulders or rock wedges; b) Cracking of slabs of concrete face and leaks into the embankment; c) Movement of the joints between slabs and slabs with a plinth; d) Leaks and water levels through the foundation and slopes; e) Deformation of each of the materials making up the embankment; f) Stress state on the embankment; g) Loss of freeboard; h) Seismic risk; the occurrence of moderate and strong earthquakes affecting the structure; i) Hydrology risk, which could cause overflow.

The auscultation system fitted out in the civil works of the HP La Yesca covers these main factors. The specific design of the type, number and location of devices, was established based on the variables you want to monitor and topographic, geological, geotechnical and seismotectonic features of the site.

The instrumentation to be installed was designed in accordance with the objectives and based on the expected behavior defined in the numerical analysis realized, taking into account its expected value and selecting the most suitable instruments.

Many instruments were installed on the embankment body (Tables 1 and 2): a) Electrical piezometers in the bottom, immediately behind the plinth, on the riverbed; b) Inclinometers in the materials T and 3C; c) Topographic reference lines on surface; d) Gauging weir at the bottom of the curtain; e) Hydraulic levels of settlement; f) Groups of six pressure cells, and six strain gauges on three parallel sections to the river, on platforms at different elevations.

Table 1. Instruments placed on the body of the embankment of La Yesca Dam.

Curtain Item	Instrument	Location	Elevation	Number
Body Embankment	Hidraulic levels	maximum cross section	425,463,500,540	105
	Inclinometers	Slabs L-21, L-15 L-30	468, 468.6, 463, respectively	3
		Slabs L-21, L-15, L-30 IC2, IC3, IC4, IC5, IC6, IC7, IC8, IC9 IC10	468, 468.6, 463 505, 497, 442, 423 391, 391.36, 387 430, 425, respectively	9
	Piezometers	PZE-1 al 13	376,376.12, 375,383,382,386,389, 386,393,386.9, 388, 387, respectively	13
	Surface topographic benches	L-6 to L44 L-7 to L-45	581 580	44
	Pressure cells	G-1 to G-6	467	38
	Extensometers-Plate	Slab 21: EXL-1 to EXL-11	563	11
		Slab 30: EXL-12 to EXL-22	563	11

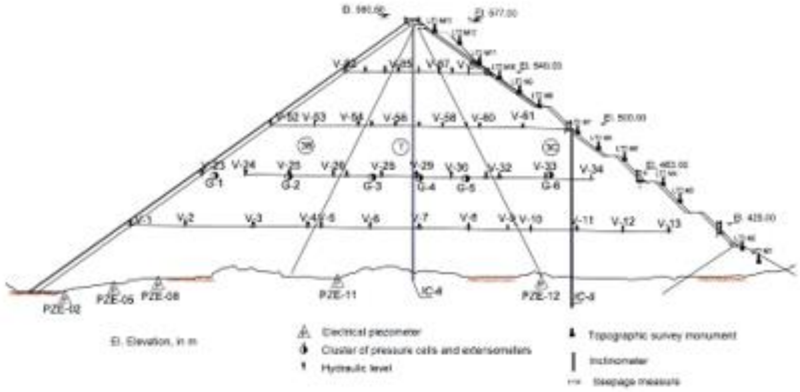
	Thermocouples	Shear-piers: T-1 to T-5 Wall support: T-1 to T-9		5 9

Table 2. Instruments placed on the concrete face of the La Yesca Dam.

Curtain Item	Instrument	Location	Elevation	Number
Face concrete	Strain rosettes	Grupos de 3 R-1 a R-12	527, 526,527, 527.03, 526.9, 527, 379, 421, 465, 509, 553, 573 Respectivamente	12
	Extensometers	Unidirectional (1-D) EXU-1 a EXU-30		30
		(2-D) EXB-15,EXB-18,EXB-21, EXB-24, EXB-30		
		(3-D) EXT4, EXT-7, EXT-9, EXT-12, EXT-14, EXT-15, EXT-21, EXT-28, EXT-30, EXT-34		10
	Clinometer	CL-1 a CL-10	378, 397, 421, 445,465, 487, 509, 532, 554, 575	10
	Extensometers joint	MJE-1 a MJE-9	519, 519, 467, 467,467, 467, 379	
	Surface topographic benches	BN-1,LR8-1 a LR8-43, BN-2	575	45

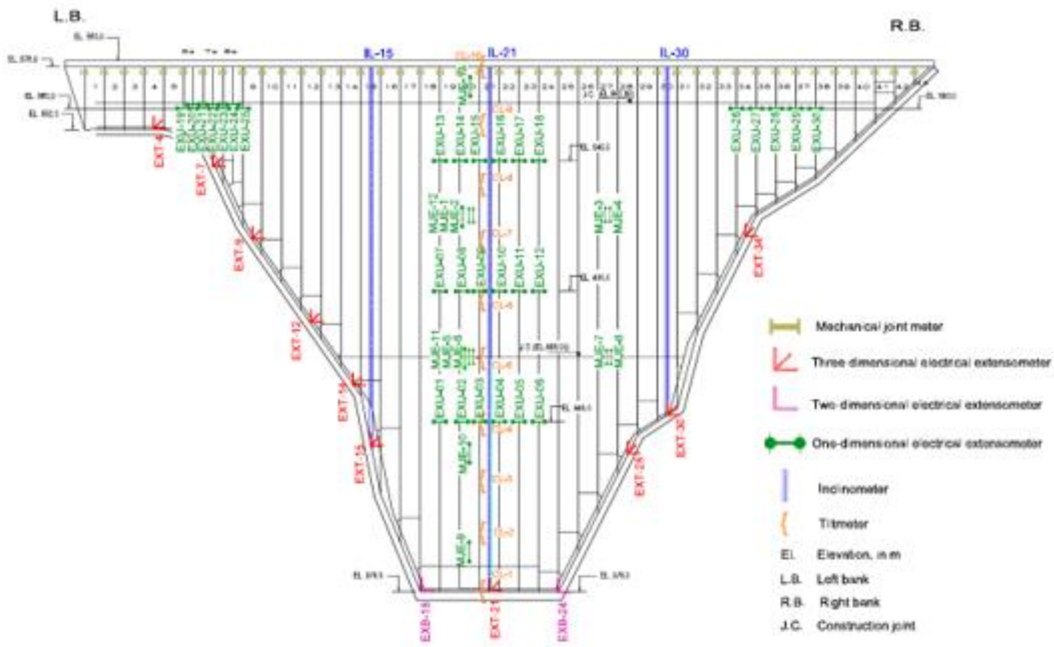
In Figure 6 are presented the most important instruments installed in the body of the embankment of La Yesca HP for maximum cross section of the curtain.

Figure 6. The instruments installed in the maximum cross section of the La Yesca Dam.



The instrumentation in the dam concrete face aimed to determine the magnitude and evolution of movements that pose a risk to its integrity and functionality; for this reason were installed the following instrumentation: a) inclined inclinometers embedded in slabs; b) unidirectional strain gauges on the vertical joints between slabs; c) bidirectional strain gauges on the board of parapet slabs and perimeter board and plinth; d) strain gauges rosettes on the perimeter-plinth board; e) topographical references and f) meter-gauge type joint plate in the slabs and vertical joints with parapet. In particular, the meter seal between the curb and the slab, at the maximum dam section, horizontal construction joint and the elevation where the double curvature of the slab was predicted, clinometers, and load cells were installed. Figure 7 shows the location of the instruments placed on the concrete face of La Yesca HP.

Figure 7. Location of instruments on the concrete face of the La Yesca Dam.



Measurement devices and piezometers were installed in the inspection and drainage galleries. These were placed intercepting major rock discontinuities. The gallery at the bottom of the curtain goes through the downstream cofferdam and allows drain infiltrated water to the embankment, which also has its gauging device.

The description of the behavior of HP La Yesca was based on references [2] and [3].

4.1WATERWORKS CONTAINMENT

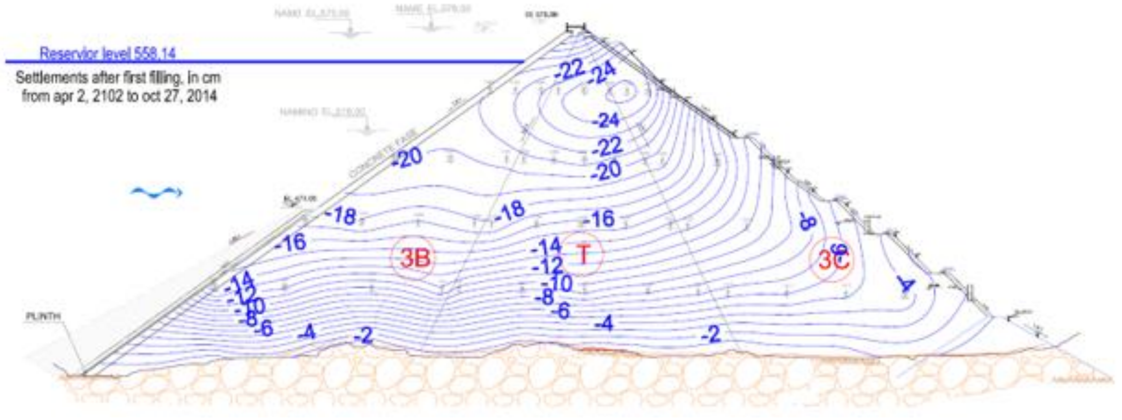
4.1.1 Rock Embankment

Figure 8 shows contours of the settlements measured of the embankment during the filling of the reservoir and the two years of operation.

It also notes that the settlements in the 3B and T materials are not very different due to their similar characteristics deformability and increased confinement of the material T. The calculated values of modules deformability at the end of construction for 3B material, T and 3C, were 174 MPa, 164 MPa and 92 MPa, respectively.

The influence of increased deformability of the material 3C and push the reservoir is only appreciated in the upper third of the rockfill, where settlements reach just over 24 cm.

Figure 8. Contours of settlements in the maximum section (L-21) of the rockfill during the filling of the reservoir and the two years of operation.

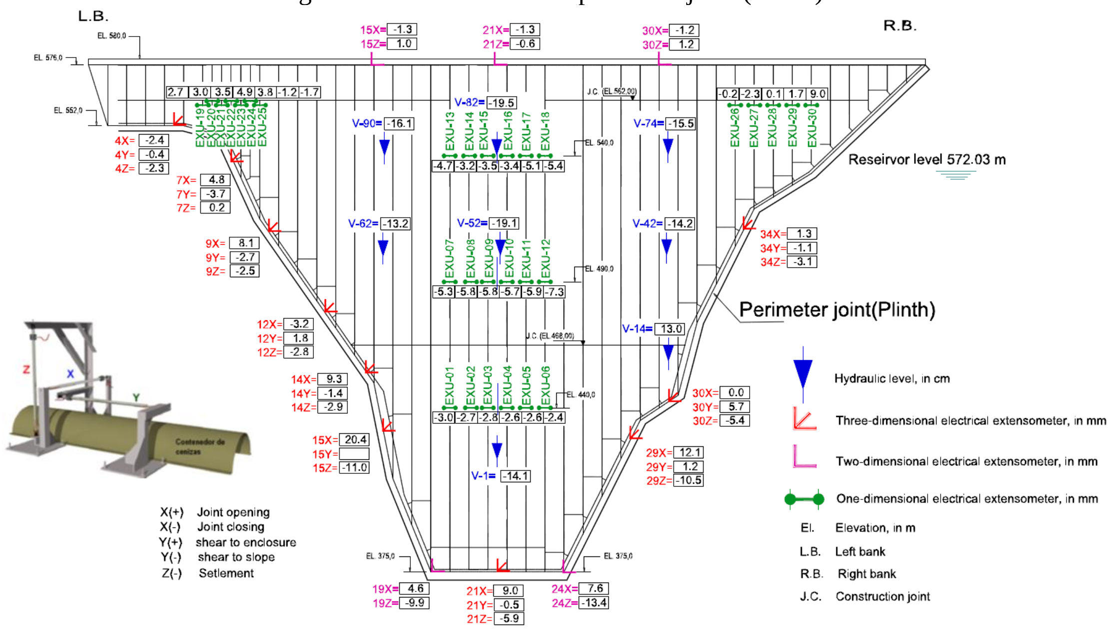


The downstream displacement of the crown due the thrust of the reservoir measured in the vertical inclinometers, placed in the body of the rockfill, and the topographical references, are of the order of 7.5 cm.

4.1.2 Concrete Face

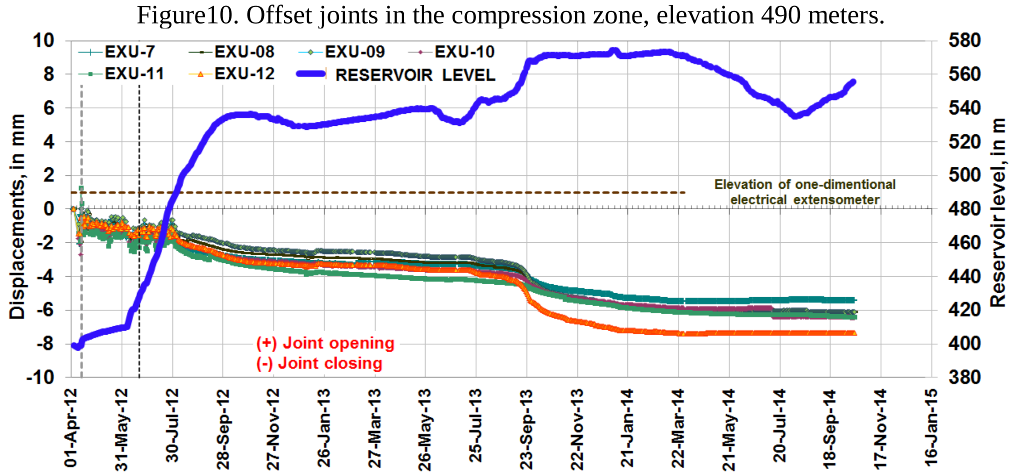
Deformation measurements made in the concrete face by the effect of filling reservoir were very small, which reflects the excellent performance of the embankment. Larger apertures of joints are registered in the EXT-15 extensometer left margin (22 mm), associated with the configuration and slope of this hillsite. Moreover, higher slabs settlements occurred in the lower right margin, without exceed 25 mm.

Figure 9. Movements in the perimeter joint (Plinth)



The strain gauges installed at elevations 440, 490 and 540 m, in the joints between the L-18 and L-24 slabs compression zone, all recorded closure magnitudes less than 9.0 mm (Figure 10); evolution did not show significant changes in October 2012 to August 2013. From this date the reservoir level increased which caused an increase in displacement registered with strain gauges placed at elevation 490 m, which did not happen in the other extensometers. By design considerations, the spacing between the slabs of the compression zone is 20 mm, filled with "EPDM" rubber.This gap should allow movement without generating compressive stress that can harm them (spalling).

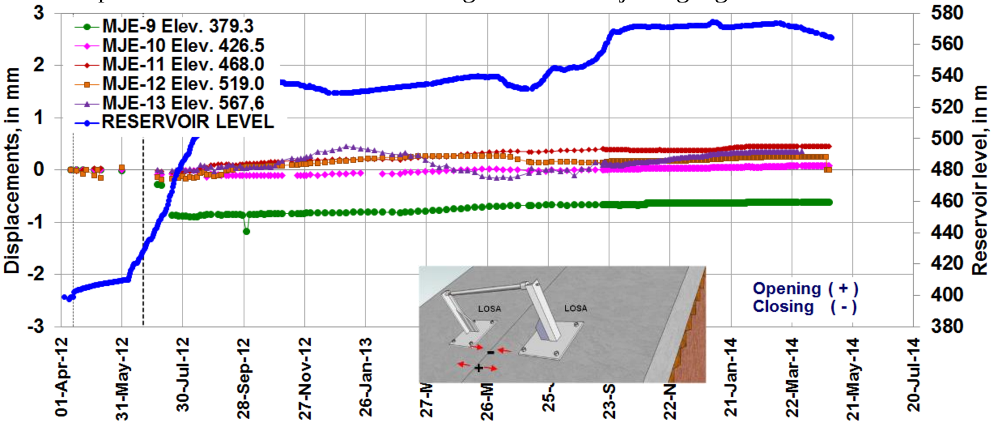
Figure10. Offset joints in the compression zone, elevation 490 meters.



The deformation measurements, made with inclinometers placed on the slab 21, between elevations 375 and 575 m, indicate a maximum normal deformation from 20 cm to 518 m elevation. It results from the settlement of the rock enbankement by the pressure of the reservoir. In the slabs 15 and 30, the normal deformation is less than 18 cm.

Electric seal meters were installed to know the interaction between slab-curb; these were embedded along the slab L-20 (MJE-9 to MJE-13), which do not show relative displacement due to the hydraulic head. The observed decrease in the MJT-13 (Figure 11) installed to the elevation 567.58 m is mainly associated to room temperature, when the reservoir level is below the instrument.

Figure 11. Displacements in slab-curb contact registered in the joint gauges embedded in the slab L-20.



4.2 STABILIZED AREA IN LEFT MARGIN

4.2.1 Structural Plugs In The Diversion Tunnels

Piezometers installed in the vicinity of the diversion tunnel 1 (Figure 12) near the slope of the stabilized zone show slightly lower pressures at reservoir level; regardless of whether they were placed under, on or above of the “Collapse” fail and ascended gradually as the reservoir did. Due to the high permeability of the rock in the vicinity of the Collapse fault, the piezometers response is immediate to reservoir level changes, regardless of location.

In the diversion tunnel 2, the rise of water in the piezometers occurs after the completion of the work of injection of structural plug. The rise of water occurred with a pressure of 70% or less than the reservoir, while the furthest not shown piezometers variations.

Figure 12. Locating electrical extensometers at anchor-plug 1.

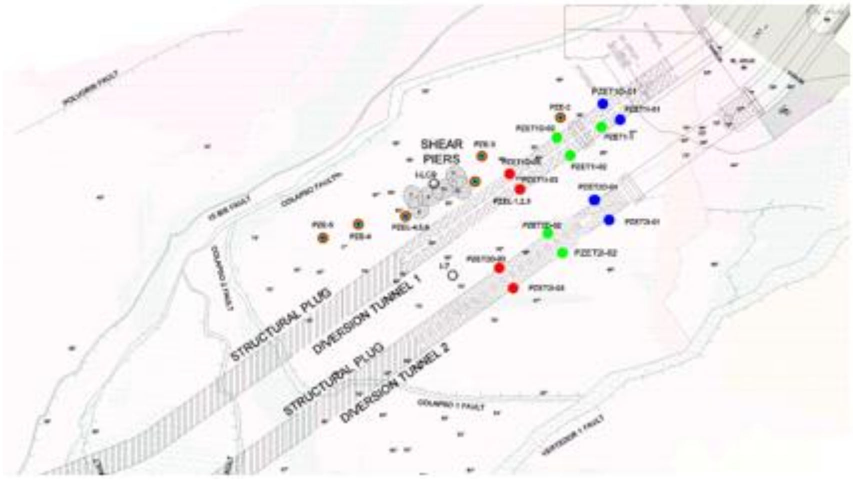


Figure 13. Water elevation recorded in electrical piezometers installed in the tunnel 1

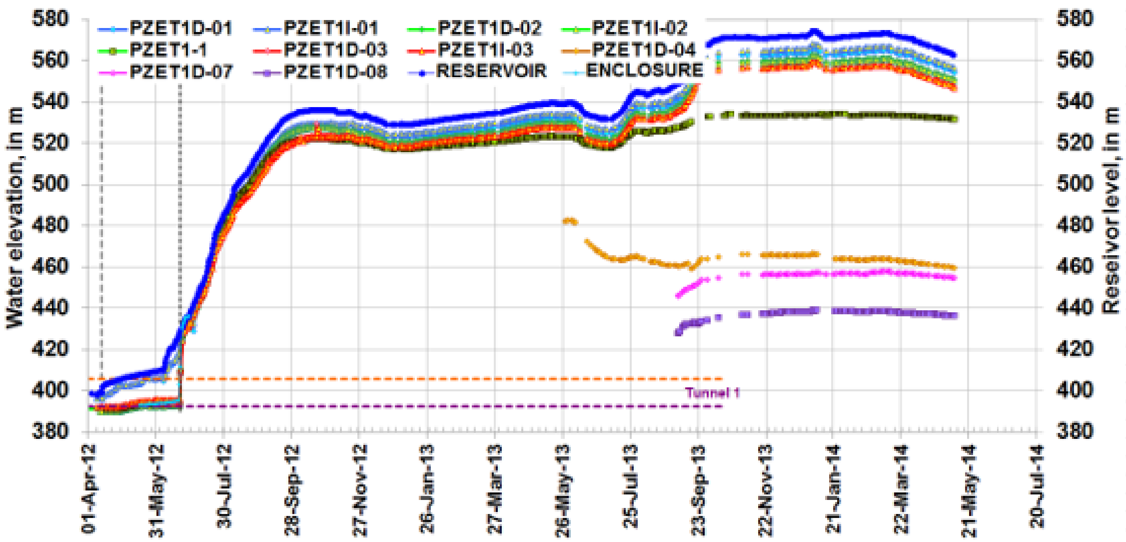


Figure 14. Locating electrical extensometers at anchor-plug 1

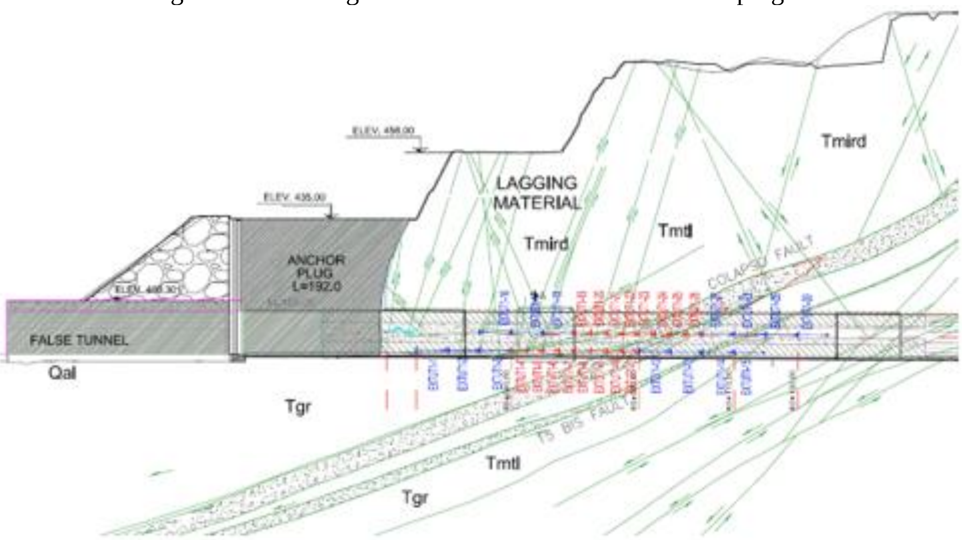
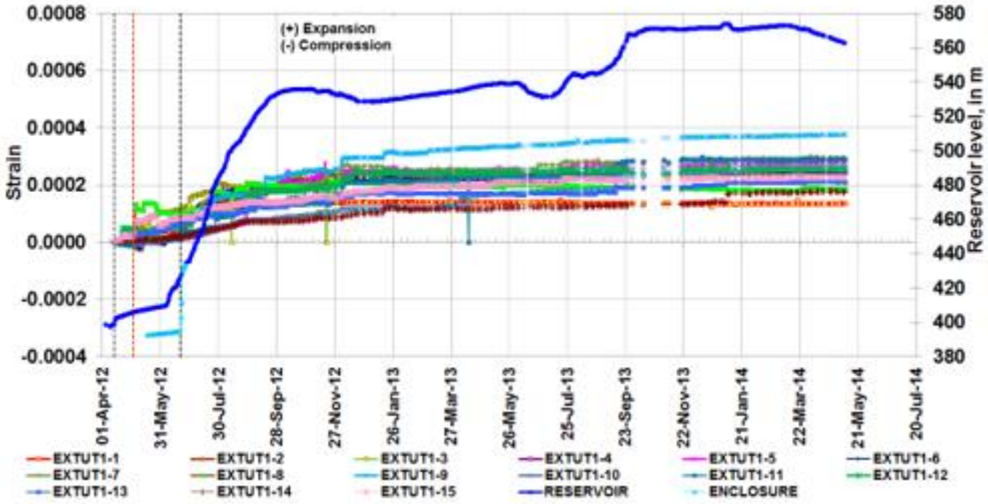


Figure 15. Strain gauges located on the lower bed tunnel anchor-plug 1



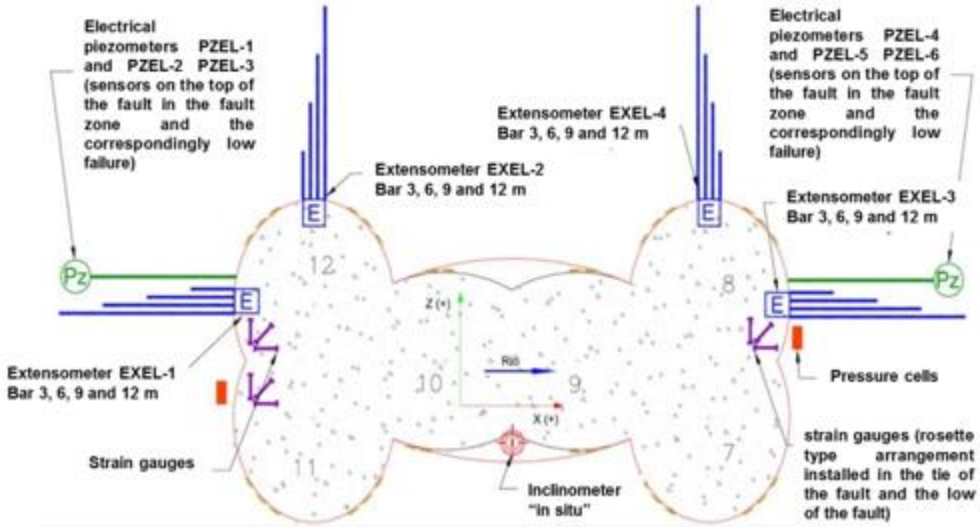
Leaks in the structural plug of the diversion tunnel 1, decreased from 125 to 12,8 l/s at the end of the injection contact; besides, these increased as the reservoir did (January 2013), reaching 28,4 l/s in May 2014, for a reservoir elevation of 557 m. In the diversion tunnel 2, the injections decreased the flow from 148,3 to 1,3 l/s and this rate remained almost constant with time.

No evidence of movement in the structural plugs at the junction with the trace of the fault Collapse, as recorded by the electrical strain gauges. The unit relative strain between the upper and lower bed diversion tunnel 1 is of the order of 0,0004 and is not indicative of movement of the rock mass by the failure; increases are associated with the thermal effect of the setting of concrete and grouting work on consolidation and structural closures. The main deformation increases plug diversion tunnel 2 due to the temperature variation by the setting of concrete and rock saturation; from December 2012, there are no significant displacements.

4.2.2 Shear-Piers

Bar extensometers, pressure cells and strain gages, were installed to determine the interaction between the rock mass and the shear-piers. In addition, these were installed to determine the possible revival movement left margin stabilized block (Figure 12). The extensometers indicate no movement in the vicinity of the columns shear, the largest accumulated displacement of 2,50 mm (elongation) was recorded in the extensometer EXEL-2, in the first 9 m, from the start of reservoir filling to date. The stresses measured in the pressure cells reflect the effect of the saturation of the rock, the recorded values are related to the hydrostatic pressure of the reservoir level and varies according this does.

Figure 16. Measuring instruments installed in the shear-piers



4.3 GENERATION WORKS

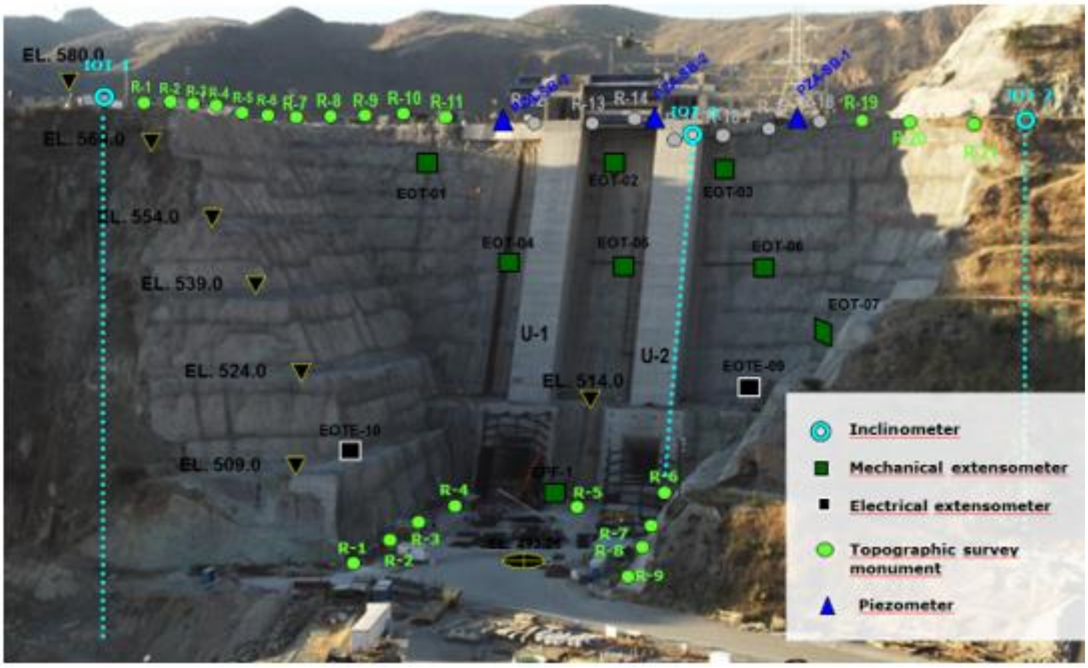
4.3.1 Intake Structure

Instrumentation was placed in the intake structure as shown in Figure 12. The slopes of this structure are maintained without significant increases in their movements from July 2013; movements before that date were due to injection work conducted from the platform located at elevation 580 m toward the gallery GD-1. No evidence of movement in depth, and the inclinometers installed in the slopes of the intake structure indicate minor displacement of 2.0 cm, from the start of filling to date.

The piezometers installed in the zone of the intake structure show variations associated with rainfall and reservoir level. The PZA-SB-3b (Figure 17) reacts quickly to fluctuations in the hydraulic head because approximately to the elevation of the bulb is the cracking that was generated during the screen injections made from the platform located at elevation 580 m.

There are no major displacements in extensometers installed on the U-1 unit with respect to the U-2, nor between the intake structure platform and surrounding area of pressure pipes of both units. The displacements accumulated from April 2012 to date are less than 2 mm.

Figure 17. Instrumentation on the slopes of the intake structure



4.3.2 Electrical Substation

There is no sign of movement in the slopes of the substation either on the surface or depth, the maximum displacement registered is 3,80 mm, which remains constant since January 2014. At the end of the section of the gallery GDS, there is input from rainwater that infiltrates the hillside through the rock fractures.

4.3.3 Powerhouse

Extensometers installed on the dome and wall rock of the powerhouse indicates stability, with a maximum displacement of 2 mm in the EECM-3 extensometer located in the dome, the start of filling to date. The small increases have been registered may be associated with reservoir pressure transmitted to the structure through the water infiltrated into the rock mass without representing unstable wedges formed by faults planes that intersect the powerhouse. The major inputs of water are presented in nearby drains to the eastern tympanum of the upstream wall favored by geological structures, and influenced directly by the emptying and filling pressure pipes.

4.3.4 Oscillation Gallery

Extensometers installed on the dome of the oscillation gallery indicate no stability problems, but have registered slight relaxation of the rock, possibly favored by moisture, with small increments that accumulate expansion values under 5 mm.

To improve the stability of the rock wall that divides the powerhouse of the oscillation gallery, were placed 26 struts. In five of these, electrical extensometers were installed; these were anchored to the base plates at the ends of the struts, to know the evolution in the load as a function of the relative displacement. Two extensometers were placed by each instrumented strut, one on the upper side and another on the lower side. The largest relative displacement is 0,32 mm, without indicating unstable conditions. The registered variations are due to the temperature effect with small increases due to stress compression; these are possibly associated with the relaxation of the rock when the units generate electricity.

4.4 WATERWORKS OF EXCEEDANCES

4.4.1 Control Structure

Most of the main beams heads are fissured especially at the top, although they also have some fissuring in side and front faces; in some cases this phenomenon coincide with the different stages of casting. To prevent water causes corrosion in the reinforcing steel of the beams and the prestressing of the support of the gates is forseen to seal these cracks and the horizontal joint between the beam and the pier after sealing besides applying a repellent of water over the entire area of the beans heads.

4.4.2 Left Slope Of The Spillway

To monitor the stability of the cuts on the left slope of the spillway, superficial references were placed on the berms elevations 485, 515, 580, 625, 670 and 730 m. Measurements indicate continued relaxation of the rock mass with predominant upstream direction toward the center of the channel. The displacement speed increased when the reservoir level drops; the accumulated displacement is less than 7 cm from March 2012 to date. The opening of the gates during the test operation of the center and left channels of the spillway on October 2, 2013, generated small increments mainly in the berms located at elevation 730, 670 and 580 m.

In depth, the displacements in the rock mass are smaller than 2 cm; small deformations observed are associated with compressible strata with no evidence of failure surfaces moving with prevailing direction upstream and into the channel.

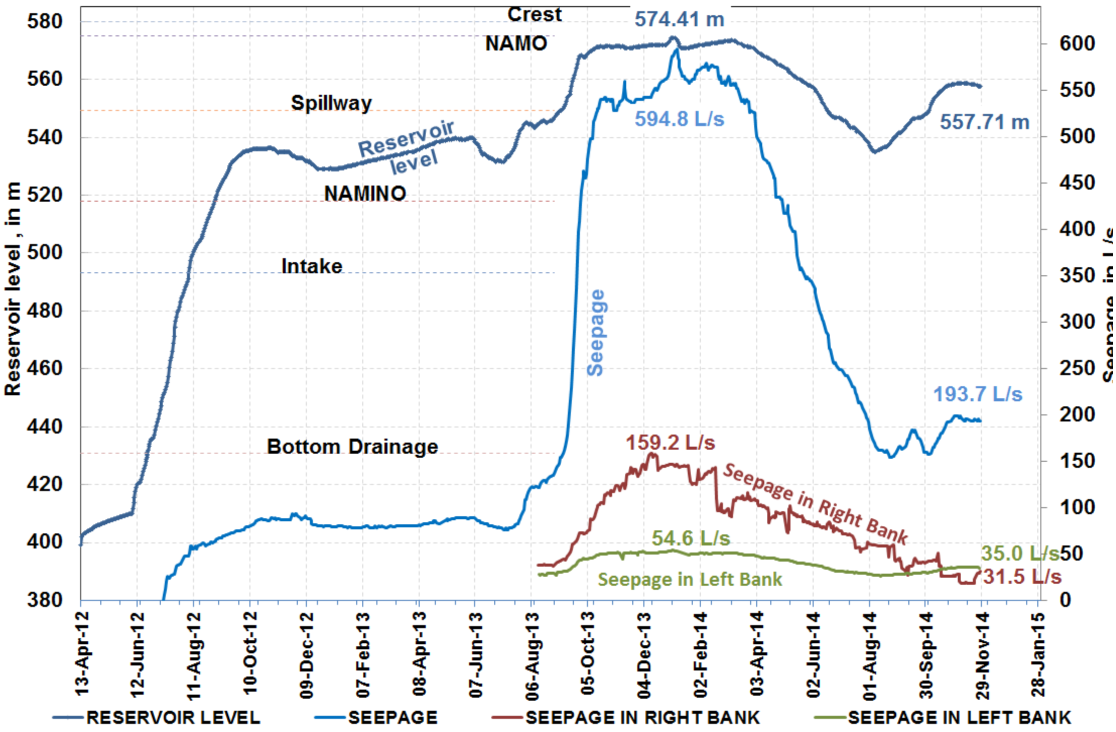
5 LEAK PHENOMENON

On April 13, 2012, the diversion tunnel gate 2 was closed, that stated the reservoir filling. This filling is made progressively reaching NAMINO (El. 518 m) in early September of the same year; a month and a half after the elevation 540, the maximum progressive filling level, is reached. From November 2012 to end of July 2013 the water level fluctuated between elevations 530 and 540 m. During this time, the leaks in the gallery at the bottom of the curtain and drainage galleries were maintained with relatively small discharge; total discharge of the order of 80 l/s (Figure 18).

In late July and early October 2013 the water level rose rapidly allowing to do performance testing of the spillway. During the months of September and October 2013 leaks at the bottom of the curtain markedly increased in 400 l/s, for a total flow of 540 l/s. This anomalous behavior was also observed in the galleries on the right bank, however, in the galleries on the left margin and diversion tunnels did not show this phenomenon.

It should be noted that due to the motion that was presented on the left bank during excavations of the portals of the diversion tunnels, it was necessary to turn the axis of the curtain, which caused the tympanum of the gallery GD-2 stay with low coverage, of the order of 10 m, with respect to the plinth. This lack of coverage and quality of the rock in this area could explain the fact that when the reservoir reached elevation 540 m, appeared on the gallery covering GD-2, near tympanum, two fissures, which contributed significant leaks in the order 45 l/s. As a remedial measure, a concrete plugs 30 m long was constructed, leaving a length of 15 m accessible for injections seal from the inside.

Figure 18. Evolution of the leaks with the reservoir level in the drainage galleries GD-1 to GD-4 and the bottom of the curtain.



To reduce seepage measured to values that do not constitute a risk to the security of HP, a series of studies to determine the causes of the observed phenomenon and implement corrective actions were initiated [2].

5.1 STUDIES OF LEAKS AND MONITORING

In order to know the causes of the abnormal increase of leaks at the plant, a series of studies were conducted; these were both analytical and field, aimed at identifying the areas of greatest contribution and define corrective actions.

Analytical studies were performed using some commercial numerical platforms (SEEP, FLOW3D, etc.), taking into account the geometry and permeability characteristics of the various structures that could contribute to the phenomenon of leaks observed (curtain, plinth, joints, galleries, etc.). The existence of possible cracks in the concrete face (cracks, openings joints, etc.), was considered in the model, to justify the registered leaks.

Leakage monitoring is carried out on both sides of the curtain, in the body and in surface. The leaks of the various branches of the injection and drainage galleries, of the slopes and the powerhouse are measured separately, in order to identify their origin and thus identify the areas of greatest contribution. Seepage of the body curtain are captured in the filtrations gallery at the bottom of the curtain and measured continuously in an outer spillway. Similarly, in curtain control house small leaks were found, that are measured and recorded at present.

According to the monitoring of leaks, and from the reservoir exceeded the elevation 543 m, were the GD-2, GD-3 and GD-4 galleries, on the right margin, which had the biggest leaks in the branch lines that go to the plinth. Except the gallery GD-2 where reinjection into the concrete stopper area led to a leaks reduction from 50 l/s to 3 l/s. For gallery GD-3, leaks are associated with the hydraulic load, while in the GD-4 the flow magnitude is directly influenced by filling or emptying of the pressure pipes, coupled with the variations of the reservoir.

Since early September 2013, leaks were observed in some booths instrumentation (platforms located at elevations 425, 463 and 500 m); it was particularly observed in the protection brackets beams hoses of hydraulic levels, with an evolution of seepage that follows the variations of the reservoir.

The piezometers installed from the lower galleries both sides GD-4, GD-3, GI-4 and GI-3, indicate variations associated to reservoir on bulbs closest to the plinth, but a lesser magnitude by the loss of energy to infiltrate the rock mass. In GD-2 gallery the corresponding piezometers had a gradual increase of pressure from the concrete construction of the cap, but at present they experiencing a similar configuration of the reservoir.

One aspect that is relevant in the evolution of the leaks is that during emptying at equal elevations, the discharges recorded at the bottom of the curtain and in right margin galleries are higher than those recorded during the first filling. This may be associated with an unblocking of important waterways, due to washing of fine material from some discontinuities of the massif

5.2 FIELD WORK AND REMEDIAL MEASURES

The field work focused on reducing the hydraulic conductivity of the rock mass. By sealing the possible ways of filtration in order to create a continuous sealed flat to prevent the infiltration of water from the reservoir into the body of the curtain; taking care not to generate hydraulic pressures that can lead to slope instability upstream or downstream of the curtain.

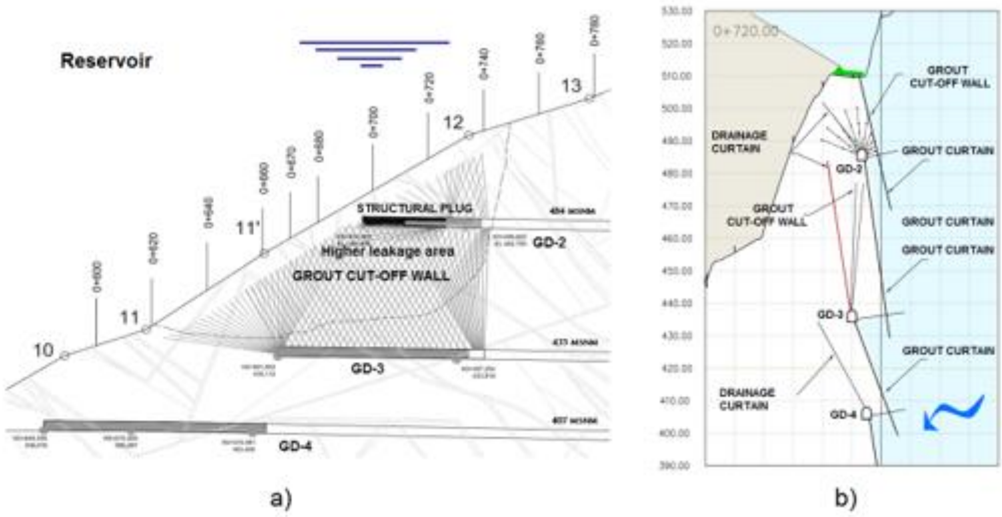
From the reservoir, brine tests were performed to verify whether there were preferential flows to the galleries of right bank or to the filtrations gallery. Brine was placed at various points of the edges of the plinth, between elevations 475 and 518 m. Residues of brine were detected in the Gallery GD-2 and the leaks registered at the foot of the curtain. This fact revealed a communication directly between the reservoir at the elevation 490, and the embankment of the curtain.

The fieldwork consisted in building a structural blind stopper in the gallery GD-2. This was done to restore conditions of initial coverage, which were changed due to the rotation axis of the curtain that was necessary because of the movement that occurred in the left margin during excavations portals diversion tunnels.

Similarly, the waterproof screen was reinforced from the GD-2 and GD-3 galleries. Such reinforcement consisted of injecting boreholes in the form of fans, umbrella and injection lines, thereby seeking to intercept and seal the lines flow and give continuity to the waterproof, closing the existing windows. From the GD-2 gallery boreholes are inclined in various directions, down and up, while in the gallery GD-3 are inclined upward (Figure 19). This reinforcement is complemented by injection of mortar on boreholes from surface (Elev. 510 m) to the vicinity of the same gallery GD-2 for the treatment of Pillars geological system. It is expected to place an inverted filter between the plinth and the right slope between points P11 and P13, to try to seal the flow channels from upstream (Figure 19).

In order to identify possible communications between the drainage galleries GD-2, GD-3 and GD-4, several measurements controlled were done by equipping the corresponding drainpipes with nozzles and shuters. The tests consisted of close and open the valves of each gallery simultaneously controlling the evolution of flow by measuring the transport of suspended solids, rejection pressures and hydraulic loads.

Figure 19. Reinforcement sealing plane through the galleries injection and drainage. a) Longitudinal section; b) Cross Section.



5.3 CONCLUSIONS AND RECOMMENDATIONS ARISING FROM THE STUDIES CONDUCTED

The main results of the studies conducted to establish the origin of the leaks that have occurred in HP La Yesca, and the impact of the remedial measures taken are described below. Should be noted that some remediation activities is still in process or are at planning level, because although the phenomenon of leakage is minimized to date, is necessary to ensure that in future such flows shall be maintained in permissible values for this type dam. To achieve this purpose some additional recommendations are also emitted.

- The leaks observed at the bottom of the curtain may come from the rock mass or for water filtration through some of the joints in the slab or plinth; so it must be done a direct or indirect inspection, of the surface concrete face in order to detect possible fissures, cracks or leaks.
- The cause of the sudden increase of the leaks in the injection and drainage galleries and at the bottom of the curtain (September and October 2013), can be attributed to unblocking of major waterways, probably due to the washed out of fine solid material filling some rock mass discontinuities.
- The analysis of all tests conducted, allowed to define the possible extension of the area of the rock mass where it have the highest infiltration. In addition, it has confirmed the existence of channels of direct communication between the reservoir and the gallery GD-2; communication between GD-2 and

GD-3 galleries, communication between galleries and curtain embankment. It seems that there is no direct communication between GD-3 and GD-4 galleries.

- The studies confirm that there is direct communication between the reservoir and galleries, by the drains and between the drains and the curtain embankment. Apparently, waterways are not closed by the screen sealing, which makes possible the existence of windows. Clearly these windows must be closed by injection materials, which seal the free waterways (like dense grout, mortar, mud, etc.) that have flow rates up to 50 l/s.
- During the shutting tests, at the beginning of the valves opening, turbid water was observed for a few seconds. Sediment transport and the fact that the flow of filtration measured at the same elevation in the reservoir, are higher than emptying, suggest that the sediment transport through the drains is partially washed some of the waterways. Then it is expected that during the next filling, the flows will increase if the sealing is not achieved.
- Inspection of the water of the spillway at the bottom of the curtain revealed that there is no sediment transport from the embankment of the curtain.
- It is necessary to continue the monitoring of leaks to check the effectiveness of treatments.

6 CONCLUSIONS

This paper presented a description of the behavior of the different structures that comprise the HP La Yesca, after two years of its construction and operation, based on the results of the installed instrumentation and analytical studies performed.

It was particularly described the behavior of stabilizing structures (columns shear and tension anchors of diversion tunnels), which was necessary to implement to control the movements of a large mass of rock through a plane of a preexisting failure in the left margin; this started moving during the construction of the portals of diversion tunnels. The measurement results of the monitoring show that since the start of the filling and with a varying reservoir level, there are no signs that reveal movement in this area.

With respect to the other structures that comprise the HP La Yesca, the electrical substation, powerhouse and oscillation gallery, instrumentation results indicate that their behavior have been satisfactory. A relaxation movement of the rock mass with predominant upstream direction toward the center of the discharge channeI was observed in the case of the left slope of the spillway area, nevertheless the operation of the HP is not putting at risk.

The dam and the concrete face shows an excellent behavior up to now, with less than 20 cm of maximum settlement in the rockfill embankment during the reservoir filling and the two years in operation. The instruments in the concrete face joints have less than 15 mm in opening or settlement as result of the good rockfill behavior.

Finally, the phenomenon of leaks that has taken place on the curtain and drainage galleries and injection after the filling of the reservoir has been controlled almost entirely by implementing of a series of fieldwork, some already executed and others per execute. It has been defined very precisely the area of strong contributions and have implemented the necessary corrective measures. To date (November 2014), with an elevation reservoir of 557.82 m, missing 17 m to reach the elevation of NAMO, flow infiltration at the bottom of the curtain are 195.5 l/s, which pose no risk for the operation of the HP; however, is necessary to ensure that in future such leaks will remain at manageable values for the plant.

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