ITUANGO HYDROELECTRIC POWER PLANT
COLLAPSE OF THE AUXILIARY DIVERSION TUNNEL
CONSTRUCTION ALL RISK POLICY No. 2901211000362
ROOT CAUSE ANALYSIS

REPORT OF

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DATED

August 2, 2019

Subject Matter: This root cause report pertains to the Ituango Hydroelectric Power Plant, Antioquia, Colombia, that was under construction when the Auxiliary Diversion Tunnel collapsed. The event was recorded as April 28 to 30, 2018.

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1. DISCLAIMER

1.1. This report has been prepared by Dr. Christopher Snee, Prof. Luiz Guilherme de Mello, Mr. Bernard Murphy, and Dr. Rafael Prieto, jointly referred to as the technical experts to the Adjuster, or the Experts. This work is entirely of the Experts unless identified otherwise and was developed from information provided and observations made on the site visit dates. The request for information (**RFI**) and the status of responses to the RFI are included in Appendix A.1. Citations are referenced as provided. The Experts make no representation or warranty concerning the accuracy or completeness of information provided. The Experts reserve the right to amend any of the content in this report if further information is provided or comes to light that, in the opinion of the Experts has a material effect on the investigation, analysis and conclusions.

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2. EXECUTIVE SUMMARY

Background to the Ituango Hydroelectric Power Project

2.1. The Ituango Hydroelectric Power Project (the **Plant**) is about 170 kilometers from the city of Medellin in the north-west of Colombia at 75°3946"W ; 7°0722"N , on the Cauca River, in the Department of Antioquia. It is located about 8 km downstream of the Pescadero bridge, on the road to Ituango. The Cauca River is about 1,350 km long, with a basin of 37,800 km^2 and discharges into the Magdalena River, which flows to the Caribbean Sea to the north (Figure 1).

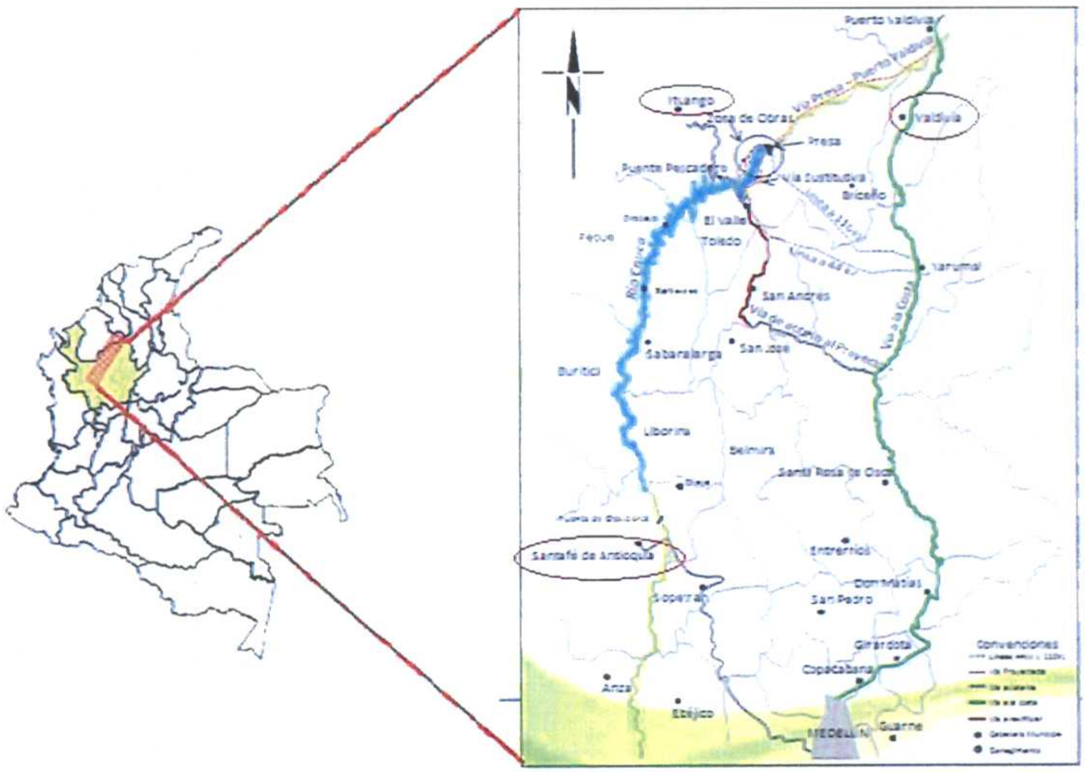

Figure 1. Location of the project in Colombia

2.2. The Plant includes a 225 m high, 550 m wide Earth-Core-Rock-Fill (**ECRF**) dam of the Cauca River, diversion tunnels for the river and a water conveyance system of tunnels to an underground power plant system in the rock mass forming the right abutment (Figure 2)

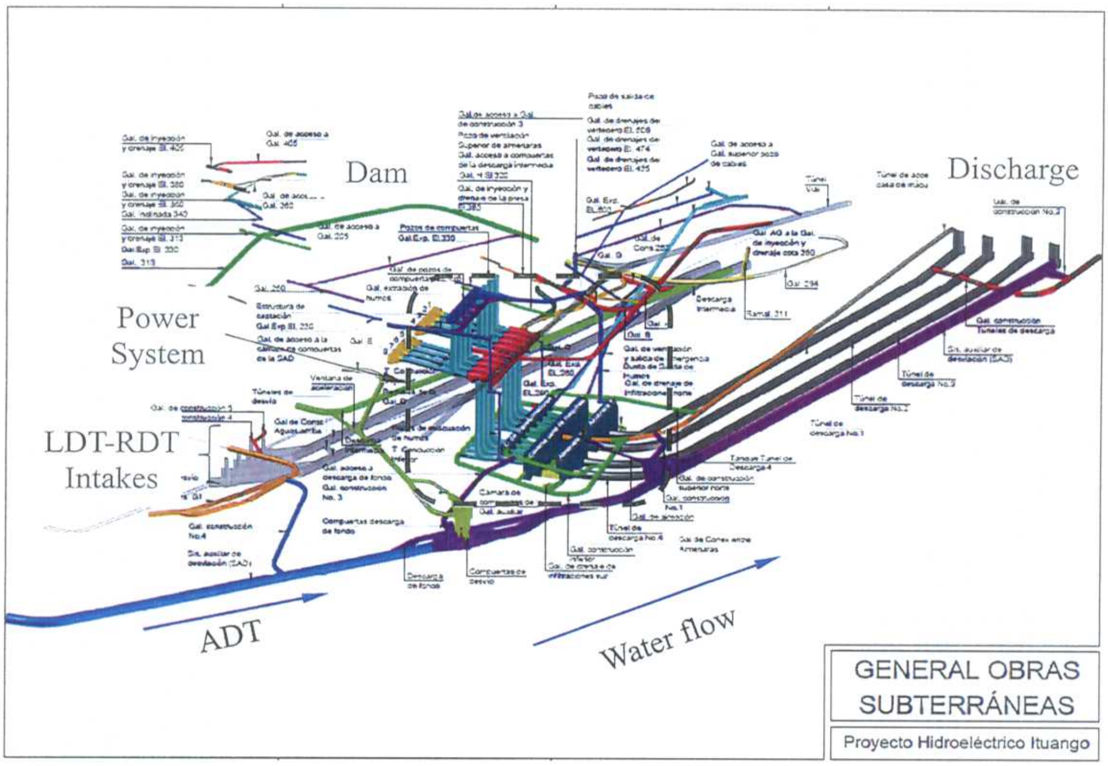

Figure 2. Layout of the underground water conveyance and power systems[1]

2.3. The power plant will generate 2,400 MW[2] and is being undertaken by *Empresas Publicas de Medellin* (**EPM**).

2.4. The tunnel where the event occurred, the Auxiliary Diversion Tunnel (ADT), was divided in four sectors, as shown in Figure 3. The sector number is used throughout the Plant to identify areas of the works.

[1] Integral (Undated) Project Layout.
[2] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0. Page 3.

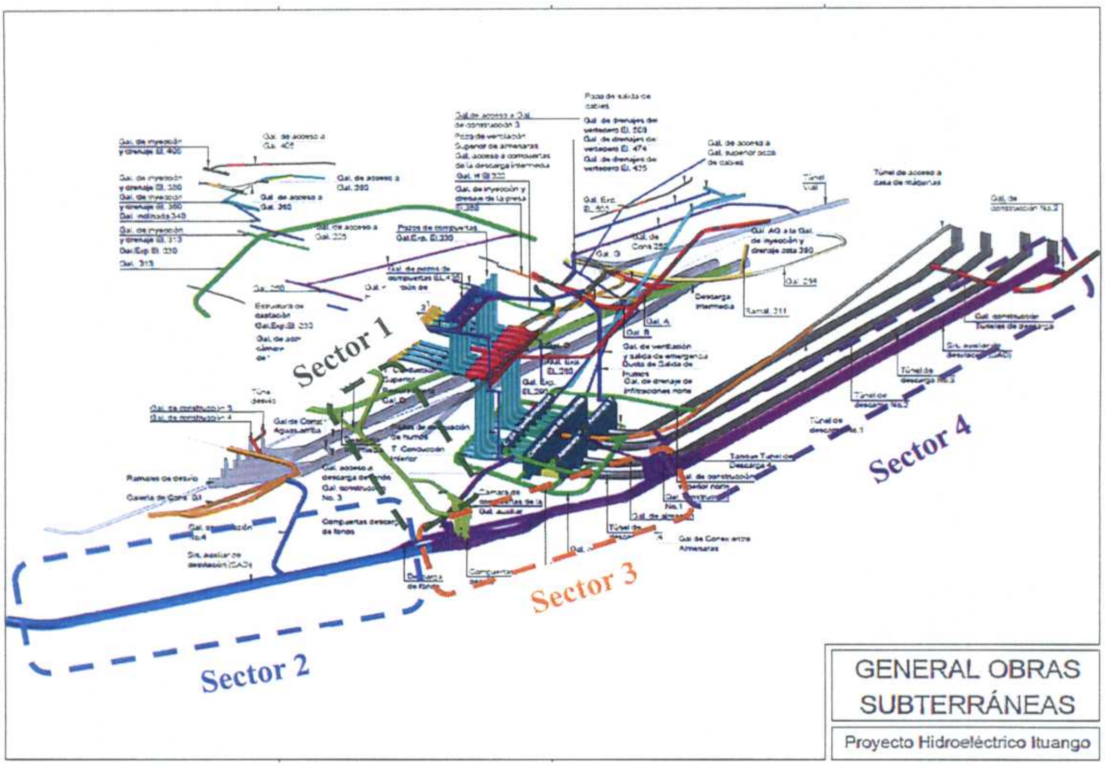

Figure 3. Sectorization of the ADT

Design and Construction of the Tunnels

2.5. The tunnels were designed using empirical, analytical and numerical methods for excavations in rock[3]. The design produced a system of support for the tunnel that varied from minimal support for very good stable rock (Type I) to the heaviest support for poor, weak rock (Type IV). The support was a combination of rockbolts[4], shotcrete[5], wire mesh[6] and arches[7].

[3] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, I-I-2194-034-REV-01-R0, I-M-2194-034-EST-01-R0, and I-M-2194-034-HID-02-R0

[4] **Rockbolt** is a metal bar installed in a hole drilled into the rock and fixed against the wall of the hole (grouted) to hold the blocks and layers of rock together. It is typically 20 mm to 32 mm diameter. The grout can be cement or resin. The exposed end of the bar is threaded and a plate and nut are attached and tightened against the tunnel wall. The length of the bar depends on the condition of the rock. The bar can be tensioned (active) or untensioned (passive).

[5] **Shotcrete** is concrete that is sprayed by air pressure at high velocity against the wall of the tunnel. It adheres to the rock and sets relatively quickly to seal and stabilize the rock mass.

[6] **Wire mesh** is steel reinforcement placed against the rock or within the shotcrete to improve the properties of the shotcrete.

[7] **Arch** is a generic term for a steel beam formed to the idealized shape of the tunnel and placed against the rock to support the ground.

2.6. Construction of the Project started in September 2010 and the original river diversion works were two tunnels, the Left Diversion Tunnel (**LDT**) constructed between August 16, 2012 and June 6, 2013, and the Right Diversion Tunnel (**RDT**) constructed between July 17, 2012 and September 8, 2013, both of which passed through the right abutment[8]. These two tunnels required gates to stop the flow of river water. However, these gates could not be built in time to meet the diversion schedule. Therefore, a third diversion tunnel was proposed, the Auxiliary Diversion Tunnel (**ADT**[9]), allowing the planned diversion of water through the LDT and RDT (without gates) in February 2014[10].

2.7. The Cauca River was diverted through the LDT and RDT on February 17, 2014[10]. Construction of the ADT started in July 2015[11] and was completed in September 2017[12]. The Cauca River was diverted through the ADT on September 22, 2017[12], followed by plugging of the LDT in October 2017 and the RDT in February to March 2018[13].

2.8. The loss that is the subject of this report was a collapse of the ADT. The tunnel was a D-shape (curved roof, vertical walls and flat floor) with a nominal height and span of 14 m (Figure 4).

[8] These dates refer only to the main tunnels and not the portal works or access galleries
[9] The ADT is also referred to as the Second Auxiliary Diversion Tunnel (**SAD**) and as the Galería Auxiliar de Desviación (**GAD**)
[10] Ingetec-Sedic (2014) PHI-IFF-LC1-001-R0 Informe Final de Obra de Contrato. Page 1.
[11] Figure 14.2 PHI-IFF-LC-011-R0. July 1, 2015
[12] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0. Page 3.
[13] Ingetec-Sedic (2019) Proceso de Cierre de los túneles de Desviación Izquierdo y Derecho. PPT Summary.

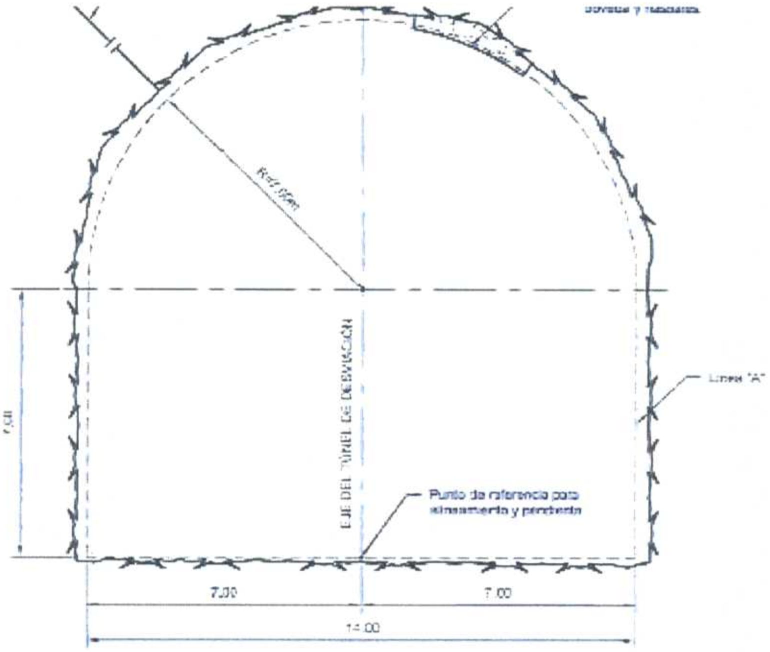

Figure 4. Dimensions and shape of the ADT

Summary Description of the Event

2.9. On April 28, 2018 the ADT collapsed near the intake portal, ground vibrations were recorded by a local network of seismographs, and workers on the reservoir described a wave and air emanating from the ADT intake portal towards the reservoir. The collapse progressed and ultimately "daylighted" on April 30, 2018, as a 100m wide sinkhole that blocked the tunnel, causing unplanned impounding of the river (Figure 5).

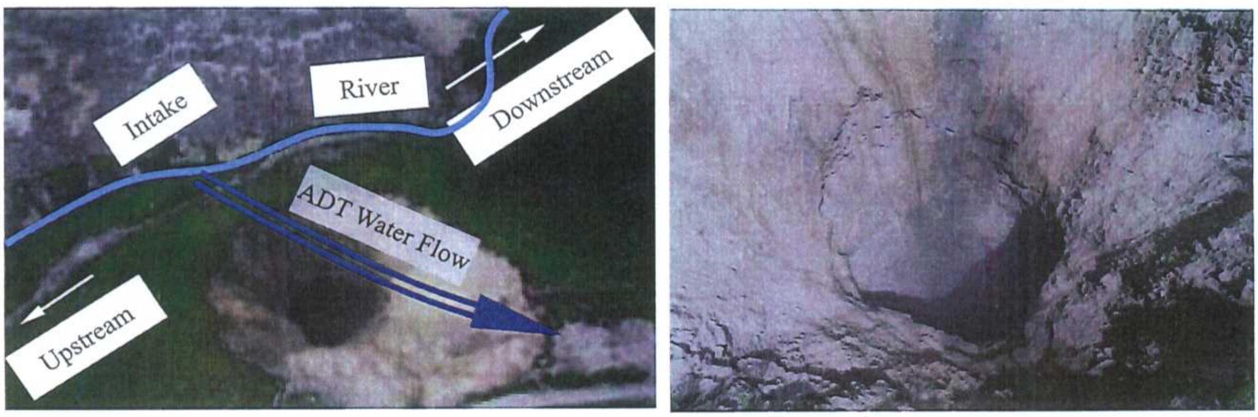

Figure 5. Aerial view of the sinkhole that appeared on April 30, 2018 above the ADT

2.10. At this time the ECRF dam was at elevation 385 masl[14], 50 m below the planned final elevation of 435 masl, and 16 m below the spillway crest. The water level was rising at a rate that would cause the incomplete dam to overtop so on May 10, 2018 the water was diverted through four of the eight intakes of the power system (1, 2, 7 and 8) to the underground power complex that was being fitted with the power generation equipment. This prevented the water level overtopping the dam, in turn allowing the dam to be raised to the level of the Spillway by a process called "priority filling".

2.11. During this time other parts of the Plant, including tunnels, caverns and slopes were recorded as unstable, collapsed, moved or deteriorated; the extent and nature of which was provided to the Experts by their investigations, repairs of the Project and documents.

Mechanism of the Collapse

2.12. It is our opinion that the collapse occurred at a geological shear zone at or about chainage 0+540. The most probable mechanism of the collapse at this position was as follows:

a) A flood event between April 1, 2018 and April 16, 2018 raised the reservoir level to a peak river level on April 13, 2018, which increased the river level at the upstream portal of the ADT to elevation 262.2 masl, which meant that the pressure of the water in the rock mass of the shear zone at chainage 0+540 increased to about 48.2 m equivalent head of water above the floor of the tunnel.
b) The river level dropped faster than the water could drain from the ground such that the pressure on the outside of the tunnel was greater than inside the tunnel, overloading the existing support causing it to fail, leading to fall out of key blocks of rock which were now unsupported.
c) This process of fall out of blocks of rock and overloading of the remaining support continued until there was a major collapse of the tunnel on April 28, 2018, that temporarily blocked the tunnel at around 0+540, but was breached on April 29, 2018 by the pressure from water backed up behind the debris eventually washing out, eroding and transporting the debris into the tunnel. This was followed by a final major

[14] **masl** – meters above sea level

collapse at the same location that completely blocked the tunnel and the collapse progressed to the surface on April 30, 2018 where it appeared as a sinkhole.

d) It is probable that while the water was flowing through the tunnel between September 22, 2017 and April 28, 2018 the support installed in the ADT was degraded by turbulent flow because it had a highly irregular shape and the floor was eroded which undermined the support, all adding to the vulnerability of the stability of the tunnel.

Root Cause of the Event

2.13. The root cause of the event was a combination of unresolved issues with the design, supervision and construction processes in an area of a shear zone that made the tunnel vulnerable to a particular set of hydraulic conditions that occurred in mid-April 2018. This is developed in more detail below.

a) The Plant's design for a diversion tunnel was weakened for the ADT, so it was not robust for a known shear zone. This support design was not robust enough because:
    i. The spacing of the rockbolts in the crown was increased such that 25% fewer bolts were installed in the same section of tunnel crown.
    ii. The reinforcing mesh was changed such that the shotcrete had less resistance to ground movements pushing against it.
    iii. There was no concrete floor to prevent erosion of the rock.

b) This weaker design was sensitive to deviations from the idealized circular roof geometry expected by the design so that it was marginally stable, and unstable under certain operating conditions.

c) Even though a shear zone was encountered the support and protection that were both obligatory and prudent according to the design, were not implemented and the tunnel face was advanced before the full support was installed.

d) The excavated profile was very irregular, deviating significantly from the curved roof assumed in the design, preventing a structural arch forming. The overbreak of the crown resulted in higher vertical sidewalls than the design making them less stable for the installed rock bolts.

e) The rock mass relaxed and the tunnel probably converged during construction beyond the safe limit because the advance lengths were excessive, and the support was not installed fully before the face was advanced. and the instrumentation for convergence monitoring was installed and monitored after most of the convergence had occurred.

f) The Observational Method (**OM**)[15] was not correctly applied during the implementation of the design of the ADT. The convergence was not measured in time and therefore, the design could not be adjusted for the encountered ground conditions as required by the OM, specifically.

   i. Convergence of the tunnel had to be measured to validate the constructed ADT. This was not done correctly. Consequently, there was no way of knowing how the installed support was performing.

   ii. The stability of the tunnel roof could not be controlled resulting in loss of the profile due to significant overbreak, even with substantial additional support that was not envisaged in the design.

   iii. The support and rock fell out of the tunnel roof several times during construction which demonstrated that the support was not working as intended which should have triggered design checks under the observational method.

g) Consequently, the built tunnel was even weaker than the already marginal design.

2.14. The parties advising EPM (Asesoria and Interventoria) and the Contractor were aware of where the work did not follow the design, where the construction did not produce the tunnel with the expected shape and support and where the tunnel collapsed during construction, as evidenced by their involvement in the meetings and correspondence relating to these issues. Many of these issues were not remedied because those parties did not agree on the cause or whose responsibility it was to resolve them. Ultimately the tunnel was put into service with these vulnerabilities, presumably because those parties believed that the operation of the LDT and RDT for four years without incident was enough proof that the design was satisfactory. However, this confidence was misplaced, the support installed in the ADT was degraded by turbulent flow because it had a highly

[15] The Observational Method is a recognized procedure in which a robust design is constantly verified through the interpretation of monitoring data and readjusted whenever necessary. The OM is used to verify that the design implemented during construction is performing in accordance with the project requirements and predicted behaviour. The expected performance of the works is defined during the design. The performance is measured and monitored during construction through observations, which typically come from instrumentation and monitoring. The designer is meant to proactively interpret and review the observations during construction against the design, and this comparison is the basis to modify the design to respond to actual field conditions in a timely manner.

irregular shape and the floor was eroded which undermined the support from the start of operation. On or about April 13, 2018, with peak river level these vulnerabilities manifested when the groundwater pressurized leading to an imbalance with the water pressure in the tunnel that exceeded the structural capacity of the support.

3. MANDATE

3.1. This report was requested by the Loss Adjuster, Advanta Global Services, Wakefield House, 41 Trinity Square, London, EC3N 4 DJ, United Kingdom (the **Adjuster**). The Experts were appointed by the Adjuster to investigate a collapse of the ADT claimed to start on April 28, 2018 and which developed into a series of collapses and ground movements in and around the ADT at the right bank of the Cauca River.

3.2. The Experts and their respective specialization are as follows.

Dr Christopher Snee, Sneegeoconsult, inc. Geology, tunneling, construction means and methods, forensic engineering

Prof. Luiz Guilherme de Mello, Vecttor Consulting. Soil and rock mechanics, tunnel analysis and design

Mr Bernard Murphy, BMA Geoservices inc. geology, hydrogeology

Dr Rafael Prieto, Gannett Fleming, inc. Soil and rock mechanics, instrumentation and monitoring, tunnel analysis and design, dams, slope stability

3.3. The Experts made and managed an RFI, met with EPM, the main construction contractor, the designer and the supervisor; and jointly and individually visited and inspected the surface and underground parts of the Project several times. The RFI is in Appendix A.1.

4. OUR INVESTIGATION

Activities of the Experts

4.1. The Experts visited the site as a group and individually on the following dates.

2018		
June	20	CS, RP
August	22, 23	CS, LM, RP, BM
October	2 - 5	BM
December	12	BM
2019		
February	20 - 22	BM
April	3, 4	BM
May	21, 22	RP, BM

4.2. The Experts inspected underground and surface works where possible as shown in Figure 6.

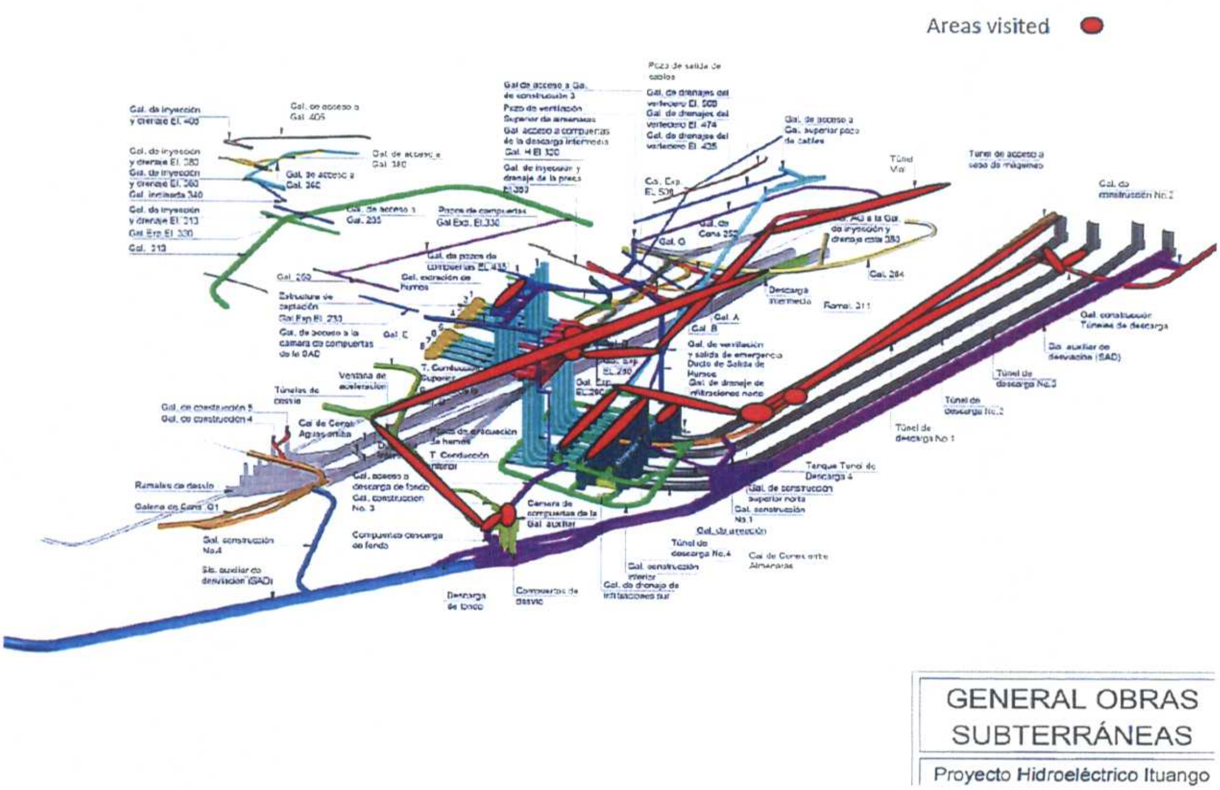

Figure 6. Location of areas reviewed and examined by the Experts

4.3. The objectives of the site visits were to meet with EPM, the Designer, Supervisor and Contractor to clarify the RFI, and to inspect various underground and surface areas and observe the investigations and repairs so that we would have good knowledge and understanding of the project. These main site visits are recorded in Appendix A.2. Also, the Experts had a conference call with two members of the Board of Consultants and met with SKAVA Consulting and EPM in Chile.

4.4. The Experts have examined the provided information up to June 24, 2019, interpreted technical documents, and carried out analysis using empirical and numerical methods to examine the ADT, particularly where the interpreted failure related to the April 28, 2018 event occurred. The methods of analysis are included in Appendix E.

4.5. Based on these activities the Experts present the most probable failure mechanism that ultimately lead to the blockage of the tunnel and the root cause of the event.

5. THE PROJECT ORGANIZATION

Basic Timeline

5.1. A timeline for the Project and investigation is in Appendix B.1. Key dates are as follows:

Start construction of the Project:	2010
Start construction of the LDT-RDT[16]:	August 16, 2012
End construction of the LDT-RDT[16]:	September 8, 2013
Deviation of Cauca River through LDT and RDT[17]:	February 17, 2014
Start construction of ADT[18]:	October 2015
End construction of ADT[19]:	September 2017
Commissioning and Operation of ADT[20]:	September 22, 2017
Plugging of LDT[21]:	October 2017
Plugging of RDT[21]:	February and March 2018
ADT solo operation[21]:	February 2018
Event:	April 28 to 30, 2018

Project Organization

5.2. The project organization was provided by EPM (Figure 7). A summary of the Project organization is in Appendix B.2, and the various contracts are in Appendix B.3. In summary, the Owner of the project is HIDROITUANGO, a group comprised of IDEA, Gobernación de Antioquia, EPM, CHEC, and minority stake investors. EPM and HIDROITUANGO executed a BOOMT (Build, Own, Operate, Maintain and Transfer) contract, and EPM is responsible for delivery.

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[16] Ingetec-Sedic (2014) Informe Final de Obra de Contrato PHI-IFF-LC1-001-R0. Page 342 Main tunnels, not including portals or galleries.
[17] Ingetec-Sedic (2014) Informe Final de Obra de Contrato PHI-IFF-LC1-001-R0. Page 1
[18] Ingetec-Sedic (2015-2017) Forma Clasificación del Terreno, Sector 3, Galeria de construccion k0+103.25 k0+110.00 Tipo II.
[19] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0. Page 430
[20] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0. Page 46
[21] Ingetec-Sedic (2019) Proceso de cierre de los túneles de desviación Izquierdo y derecho

Figure 7. Project Organization (Note: these do not reflect the actual contracts)

5.3. The construction contract for the dam and underground power complex was awarded to the Consortium CCC Ituango (**CCCI**) comprised of the Brazilian contractor Construções e Comercio Camargo Correa S.A. and the Colombian contractors Conconcreto S.A. and Coninsa-Ramón H.S., and the contract was executed on November 9, 2012 (Appendix B.3.1). The organogram does not show the consortium Consorcio Tuneles Ituango FS (**CTIFS**) comprised of the Chilean contractor Ferrovial Agroman Chile and Sainc Ingenieros Constructores, Colombia which executed a contract on September 14, 2011 (Appendix B.3.4) for the construction of the LDT and RDT but did not complete the work. CCCI took over the work on November 15, 2013 which included excavation of a small amount of bench, additional tunnel support and completion of the portal works. The CTIFS construction contract did not include the ADT. The ADT was awarded to CCCI by a contract modification executed December 22, 2015[22].

5.4. The contract for the supervision of the project was executed between EPM and to Consorcio Ingetec-Sedic, a joint venture of the Colombian firms Ingetec and Sedic (**SUPERVISOR** or **INTERVENTORIA**), and the contract was executed on October 7,

[22] EPM (2015) Acta de Modificación Bilateral No. 15 al Contrato CT-2012-000036: Construcción de la Presa, Central y Obras Asociadas del Proyecto Hidroeléctrico Ituango.

2011[23] (Appendix B.3.3). The scope of the supervision is auditing services during the construction of the civil works and the assembly of the electromechanical equipment of the Project. The responsibility is to verify, certify and document that the works are carried out according to the quality, cost, deadlines, environmental management, occupational health and industrial safety, and risk management, established by HIDROITUANGO[24]. This includes the following relevant duties:

i) Ensure their projects, drawings, specifications, and recommendations produced during the contract meet legal requirements, norms and codes.

ii) Conceive, design and submit for approval of HIDROITUANGO all organizational, operational and control elements required to carry out the works in accordance to the contract documents.

iii) Implement, induce and deploy the organizational, operational and control elements previously approved by HIDROITUANGO.

iv) Supervise the entirety of the civil works, and of the installation and commissioning of four power generation groups.

v) Once the Supervision of each work is completed, the Supervisor will prepare technical and administrative evaluations, and transfer all files and other information developed during the works to HIDROITUANGO and to dismantle the remaining organization.

vi) Design, implement and maintain a quality management system for the works being constructed.

vii) Design, implement and maintain a system for the analysis, follow-up and control of the schedules of the works being constructed.

[23] EPM (2011) Contrato No. CT-2011-000008 Servicios de Interventoría Durante la Construcción de las Obras Civiles y el Montaje de los Equipos Electromecánicos del Proyecto Hidroeléctrico Ituango
[24] Hidroituango (2010) Términos de Referencia No. PC-056-2010, Sección 6.3

viii) Design, implement and maintain a system for the analysis, follow-up and control of the changes or modifications to the drawings, technical specifications or contracting conditions of each of the contracts of the works.

5.5. The contract for the detailed design of the project was executed between HIDROITUANGO and Consorcio Generación Ituango, a joint venture of the Colombian firms Integral and Solingral in 2008. The contract for the construction phase design services (**ASESORIA**) was awarded to the same consortium in 2011 to provide Advisory services during the construction of the works and electromechanical equipment of the project (Appendix B.3.2). The responsibility is detailed design and consultancy during the construction of the project. This includes the following relevant duties[25,25].

i) Development of technical documents for bidding processes

ii) Advising on environmental requirements from authorities

iii) Evaluation of technical and economic proposals

iv) Changes to the plans and specifications necessary or advisable during the execution or assembly of the works for approval of HIDROITUANGO

v) Study and approval of construction procedures related to design

vi) Geotechnical surveys of underground works and foundations, prescription of temporary and final support of underground works, consolidation and grouting for water control of the rock mass and foundations

vii) Compliance of the designs, drawings, specifications and recommendations with legal requirements

viii) Control the criteria and techniques to meet quality and reliability requirements

[25] Hidroituango (2011) Contrato No. 2011-009 Asesoría Durante la Construcción de la Hidroeléctrica Ituango.

5.6. The four parties EPM, SUPERVISOR, ASESORIA and CCCI are jointly referred to as **The Project** throughout the report.

Standard of Performance

5.7. The contracts for the Main Works, Supervision, and Asesoria have a different Standard of Performance or Standard of Care, as follows:

5.8. The Standard of Performance for the construction of the Main Works is high and above normal industry practice. The free translation is that "*in any case when in the 'state of the art,' the technique or the practice of engineering it is not possible to define the way to carry out an activity, work or operation, or the form that the product must have once it is finished, the CONTRACTOR will be obliged to consult EPM on the way to proceed expressly, and must strictly comply with what EPM explicitly indicates. All the provisions established in this section must be construed applying the principle of good faith and are based upon the rules and criteria of 'state of the art' prevailing in each of the disciplines that serve as a basis*".

5.9. The Standard of Care for the Supervisor is consistent with normal industry practice. The Supervisor must execute the works subject of this contract, in a timely, planned, systematic and documented way through qualified personnel and validated work tools, according to the contract documents, HIDROITUANGO requirements and based upon recognized and accepted engineering good practices, in proven precedents, and according to the 'rules of the art,' the provisions of the applicable codes and norms, the current technology for the control of quality, opportunity, and costs. The Supervisor must execute the works with rational use of resources and costs.

5.10. The Standard of Care for the Designer and Asesoria is consistent with normal industry practice. The Asesoria shall execute the works subject of this contract in a timely, planned, systematic and documented way through qualified personnel and with validated work tools, according to the contract documents, the Hydroelectric requirements, and based upon good, recognized, and accepted engineering practices, in proven precedents, and according to the 'rules of the art,' the provisions of the applicable codes and norms, the current technology for the control of quality, opportunity, and costs. The Asesoria must execute the works with rational use of resources and costs.

Quality and Technical Specification

5.11. Control of quality in the main contract is the responsibility of the Contractor and any quality control measures undertaken by EPM do not relieve the Contractor of its responsibilities[26]. The Contractor did not comply fully with the Technical Specifications (**TS**) and there were numerous instances of non-compliance with quality for the tunnel construction works. These are discussed in various sections of the report.

5.12. Section 2.3.2 of the TS presented the requirements for the transport, management and use of explosives[27]. This included a requirement that blasting induced vibrations must not alter the natural state of the rock beyond the limits of the excavation or affect rock previously injected, lining, conventional, sprayed or rolled concrete previously installed or any type of structure[28]. The Asesoria alleges that the Contractor did alter the natural state of the rock by blasting causing overbreak and instability. The Contractor denied this, claiming that the overbreak and instability were due to pre-existing ground conditions.

5.13. The maximum advance length allowed was not specified but it was included in the excavation method statement by the Contractor and approved by the Asesoria and Interventoria. In fact, the Contractor did not comply with their own method statement with respect to advance lengths.

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[26] EPM (2012) Especificaciones Técnicas de Construcción, Technical Specifications, Section 1.3
[27] EPM (2012) Especificaciones Técnicas de Construcción, Section 2.3.2.3 requires the Contractor to submit the procedure for the use of explosives to EPM at least 30 days before their use in the works.
[28] EPM (2012) Especificaciones Técnicas de Construcción, Section 2.3.2.5 specifies Vibration control. The Contractor must always control and not exceed the vibration limits established by the U.S. Bureau of Mines RI 8507 which defines values of particle velocity and frequency.

5.14. Quality control measures for the tunnel support elements of rock bolts[29], steel arches[30], tensioned anchors[31], and shotcrete[32] were specified in the relevant sections of the TS. The depth of the shotcrete was specified to be measured by depth pins[33]. Steel mesh[34] and high strength steel fibers[35] were specified as shotcrete reinforcement for those areas shown on the plans. In the event the depth of the shotcrete was not measured by pins and high strength steel fibers were replaced by plastic fibers.

5.15. The standards and codes included in the TS are normal and typical for international hydro power projects and tunnels[36].

5.16. The compressive strength and energy absorption of the shotcrete was tested by the Supervisor and generally met the requirements of the specification, according to the available test results for the ADT.

Risk Management

5.17. EPM developed a risk management plan for the Plant with input from the plan developed by Delima Marsh and the associated firm Marsh Risk Consulting. Delima Marsh and

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[29] EPM (2012) Especificaciones Técnicas de Construcción, Section 3.5.5, Quality Control, Rock Bolts, ASTM D4435

[30] EPM (2012) Especificaciones Técnicas de Construcción, Section 14.1. All materials are specified to national and ATSM norms.

[31] EPM (2012) Especificaciones Técnicas de Construcción, Section 13.5. Requirements for the supply and installation of tensioned anchors for the protection of the exterior excavations, underground works and in parts of the works indicated by EPM. The materials are specified in Section 13.5.2 as complying with the ATSM norm A 416 Grade 270. Sections 13.5.3 specifies the installation, injection, tensioning and testing procedures.

[32] EPM (2012) Especificaciones Técnicas de Construcción, Section 11.4, for the preparation and application of sprayed concrete as construction or permanent support for the road tunnel at km 12+000 and in the conveyance tunnels, galleries, caverns, underground works and the over ground slope works of the Project. The materials to be used for the shotcrete are specified in 11.4.2 referencing national standards (NTC 30 and 31) for the cement type. The procedures for shotcreting are specified in section 11.4.3, including the specification of the use of depth control points and shotcrete thickness measurements. The Contractor is responsible for routine quality control of materials and the Inspector may carry out independent testing with the Contractor's cooperation and at their cost as set out in Section 11.4.4 where the required tests are also specified. Steel fiber content, control during application and repairs are specified in 11.4.5, 11.4.6 and 11.4.7 respectively.

[33] EPM (2012) Especificaciones Técnicas de Construcción, Section 11.4.3

[34] EPM (2012) Especificaciones Técnicas de Construcción, Section 13.2 sets out the procedures for the supply, transport and installation of steel mesh for use as reinforcement of structural concrete and shotcrete as shown on the Plans or as required by the Inspector. The steel mesh materials and their placement are specified per national norms (NSR-10) and ASTM.

[35] EPM (2012) Especificaciones Técnicas de Construcción, Section 13.3 sets out the procedures for the supply, transport, storage & addition of steel fibers as reinforcement for shotcrete to be used as shown on the Plans or as required by the Inspector. The fibers are specified per the ASTM norm A820.

[36] ASTM norms such as D4435 for rock bolts

Marsh Risk Consulting made several risk assessments of parts of the Project. The Project had oversight of the risk management for the insurers from the early construction stages by Nigel Legge Associates. Dr Legge made his risk surveys at least once a year on behalf of the insurers, and his reports included explicit references where the project complied with the ITIG Joint Code of Practice for Risk management of Tunnel Works. The last site survey by Dr Legge was in July 2017 and he wrote a project update in April 2018. These reports stated that there were specific risk items in the registers such as excavation and support of tunnels that were not closed out, but had been removed from the register, presumably because the tunnels were completed. However, the significant outstanding matter at the last inspection in 2017 was the lack of Project Construction Stage Risk Registers requested, as follows.

Recommend-ation	Title	Priority	Status	Comments
RIR 04/2012	Project Construction Stage Risk Registers	I/A	IP	The construction stage risk registers are reviewed on a regular basis in workshops involving the contractors and EPM, facilitated by Delima Marsh. Although an example from the risk register was presented during the survey, the current complete risk register has not been seen and this is requested. A copy of EPM's new risk management standard *Guía Metodológica Gestión Integral de Riesgos en Proyectos De EPM* has been provided.

Figure 8. Extract from Project Risk Engineering 2017-2018[37]

5.18. However, the risk assessment for the ADT was stated by EPM to comply with the Joint Code referred to above, and explicitly that this was a construction stage risk register[38]. The assessment stated that EPM was aware of the geological risk where the Project was developed and the Technical Specifications for all the underground works were developed to mitigate the risks. The assessment stated that even though the ADT arose after the TS was finalized it still applied to the ADT.

5.19. The Risks were evaluated during construction by an interdisciplinary team, consisting of the Asesoria, Contractor and Supervision. The Asesoria and the Contractor used the Q system defined in the TS to determine the quality index for each type of ground, following the document "Geological and Geotechnical Characterization" (the document

[37] Nigel Legge Associates Ltd (2018) Project Risk Engineering 2017-2018 Update Rev 1
[38] Unauthored (Undated) Gestión de Riesgos Galería Auxiliar de Desviación GAD V4. Received on May 8, 2019

is not referenced but is likely the 2010 design document[39]). The assessment explained that during the construction of the ADT, evaluations were made by the Asesoria and the Contractor, with the Supervisor as 'guarantor' and each event generated a specific document (Figure 9). We understand that Supervision in this role of guarantor served as witness of the type of support to be installed but did not participate in the classification of the terrain type nor the selection of the type of support.

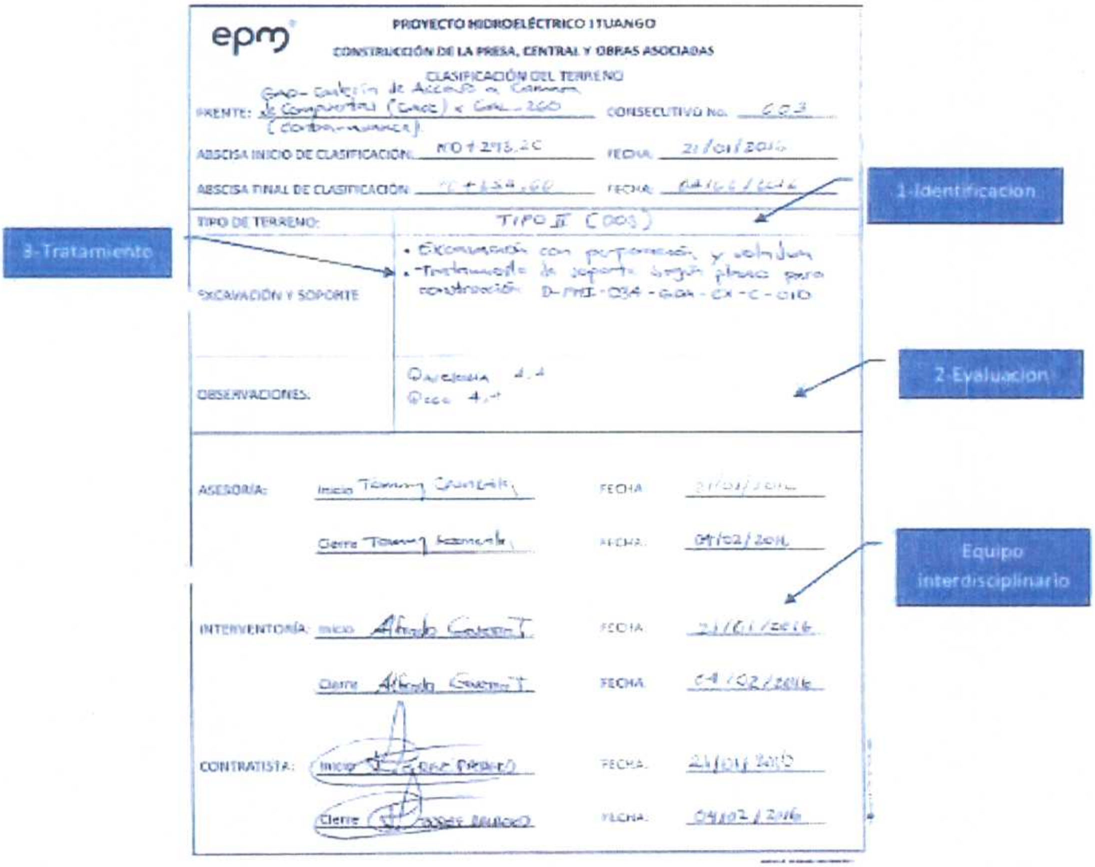


Figure 9. Example of the ground classification document used for risk management for the ADT construction

5.20. The selected treatment of the ground had to be in accordance with the design defined by the Asesoria. The *Gestión de Riesgos* document also stated that for this stage of risk management during the construction of the ADT, instrumentation was installed and

[39] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3

monitored according to what was established by EPM in the Terms and Conditions No. 6 of the Technical Specifications: Instrumentation of the works, Volume 2, October 2011[40] and drawing D-PHI-GEN-IN-ESQ-002 Rev. 3. The TS stated that the instruments were installed by CCCI and the readings and reporting were the responsibility of the Asesoria that had to provide a weekly report. The TS was explicit that because of the uncertainty about the behavior of underground excavations it was important to monitor the stability and evaluate the behavior of the rock mass as the tunnel progressed. The behavior had to be monitored by the installed instrumentation, to give information on the stresses and deformations that occurred during and after the excavation of the tunnel. In the case of the ADT the method was specifically to monitor the change in shape of the excavation by measuring convergence at several points.

[40] EPM (2011) Especificaciones Técnicas de Construcción.

6. GEOLOGICAL AND GEOTECHNICAL CONDITIONS FOR THE DIVERSION TUNNELS

6.1. The Cauca River valley marks a regionally important geological boundary between the central and western cordillera of the Colombian Andes. The western cordillera has a marine origin and the central cordillera has a continental origin, both dominantly affected by the Cauca-Romeral system of faults and the Sabanalarga and Santa Rita faults. This presents a variety of rock types in the dam area including Paleozoic Schists and Gneiss, and in the main works area the rock type was described as a Quartz-Feldspathic Gneiss.[41]

6.2. The principal fault system has a dominant NS to NE-SW orientation dipping between 60° to 90°. Two local faults crossing the immediate site area were the Tocayo (N70° W / 65° SW) and Mellizo faults (N50° -70° E / 60° -70° SE).[42]

6.3. A shear zone was mapped in the Tenche river valley[43] during the construction work orientated at 70° / 170° to 180° which was further investigated by a borehole P-GAD-02 in March 2016. Figure 10, Figure 11 and Figure 12 show the location of the borehole P-GAD-02 and the alignment of the ADT, the geological cross section with the projection of the shear zones identified in the borehole to the ADT tunnel horizon, and the photographic record of the shear zone in borehole P-GAD-02 as it reaches the excavation of the ADT. This is the shear zone that is at the location of the collapse.

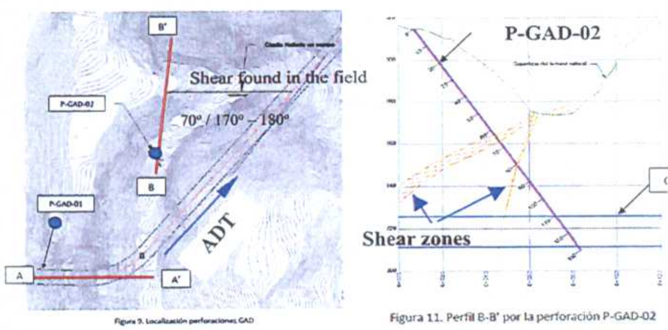


Figure 10. Location of P-GAD-02 and shear zone found in the field mapping[44] and interpreted shear zones[45].

[41] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3, Section 2.3 Page 11
[42] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3, Section 2.6.3 Page 43
[43] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0
[44] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, Section 3.11, Figure 9.
[45] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, Section 3.15, Figure 11.

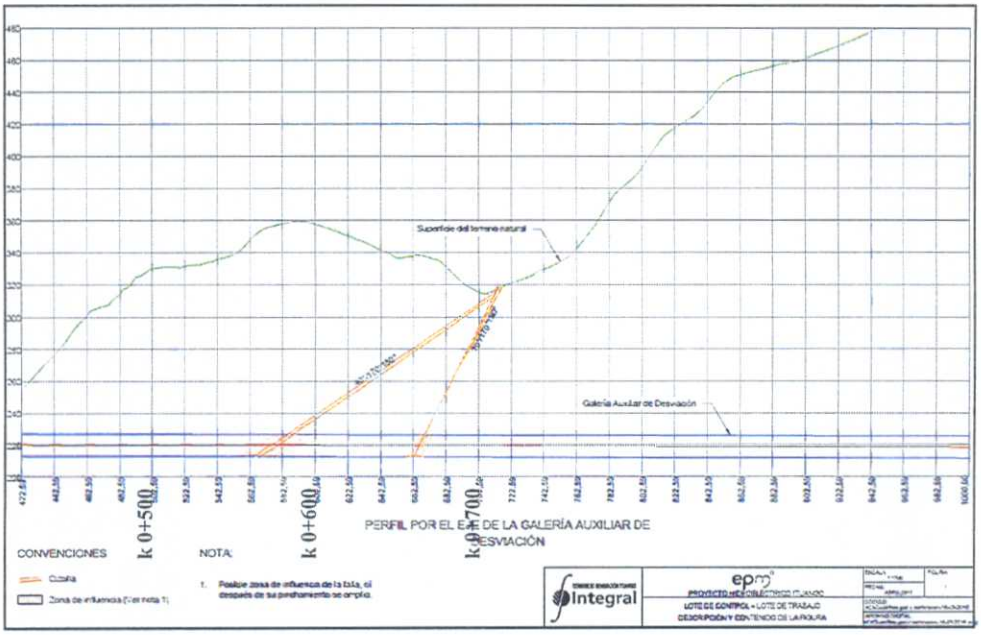


Figure 11. Figure showing projection by the Asesoria of shear zones in the P-GAD-02 core to ADT tunnel horizon[46]

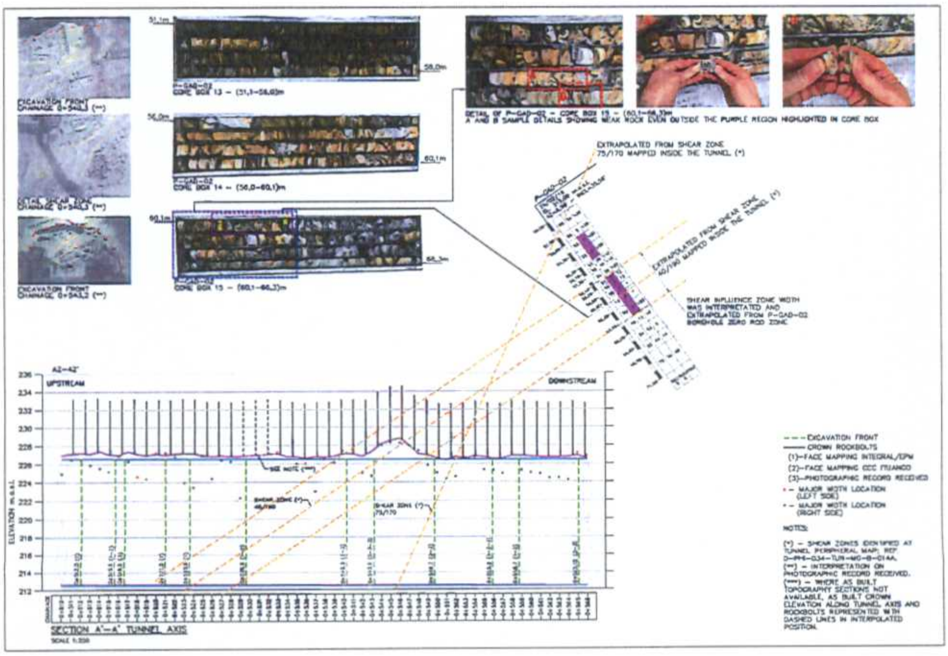


Figure 12. Figure showing Experts projection of shear zones in the P-GAD-02 core to ADT tunnel horizon.

[46] Integral (2016) Perfil por eje de la galería

6.4. Some of the faults in the region of the dam are seismically active, the nearest active fault is the Santa Rita Oeste Fault, about 10km to the west, which is classified as having a low grade of activity[47] (defined as displacements between 0.1 and 0.01 mm/year)[48].

6.5. For the portion of Sector 2 of the GAD between k 0+520 and 0+740 Gneiss is reported to be the encountered lithology with its foliation having a dip and dip direction of 20°/120°.[49] The other discontinuity sets are a generally N-S system of sub-vertical joints that define the river valley, a system of E-W sub-vertical joints that control the streams in the area such as the Tenche, and a system orientated N-S with a dip of around 60° that controls the formation of blocks on the right bank of the river.

6.6. The gneiss has been altered by weathering and classified into five zones (III, IIB, IIA, IC-IIA and IC) using the scale of Deere and Patton[50,51].

6.7. The extent of alteration of the rock mass is greater at the fault zones[52].

6.8. The bedrock on the right hillside of the project is overlain by a sequence of Quaternary sedimentary deposits including colluvium and the products of mass movements[53].

6.9. The main reference document for the geology and geotechnics for the Project is *Caracterización Geológica y Geotécnica*[54] from 2010 by the Asesoria Consorcio Generación Ituango in their initial role as Designer for the Project. This report builds on

[47] Sanchez et al. March 2003, Seismic Code of Segments of the Cauca Fault, Rev. Acad. Colomb. Cienc. XXVII, No. 102
[48] Integral (2007) Complementación de la factibilidad; Anexo 2, Anexo B, F-PHI-GGS-ANB-Sismología
[49] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2, Page 7.174
[50] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3, Section 2.6.2.2, Page 39
[51] Deere, D. U. and Patton, F. D., Slope Stability in Residual. Soils, Proceedings of the ASCE 4th Pan American Conference on Soil Mechanics Foundation Engineering, San Juan, P R, 1971, Page 87–170
[52] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3, Section 2.6.2.2, Page 40
[53] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3, Section 2.6.2.2, Page 36
[54] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3

earlier works from the 1970s onwards[55,56,57,58]. The investigation of the site was by surface mapping, remote imagery, boreholes, testing, sampling all of which were the responsibility of the Designer.

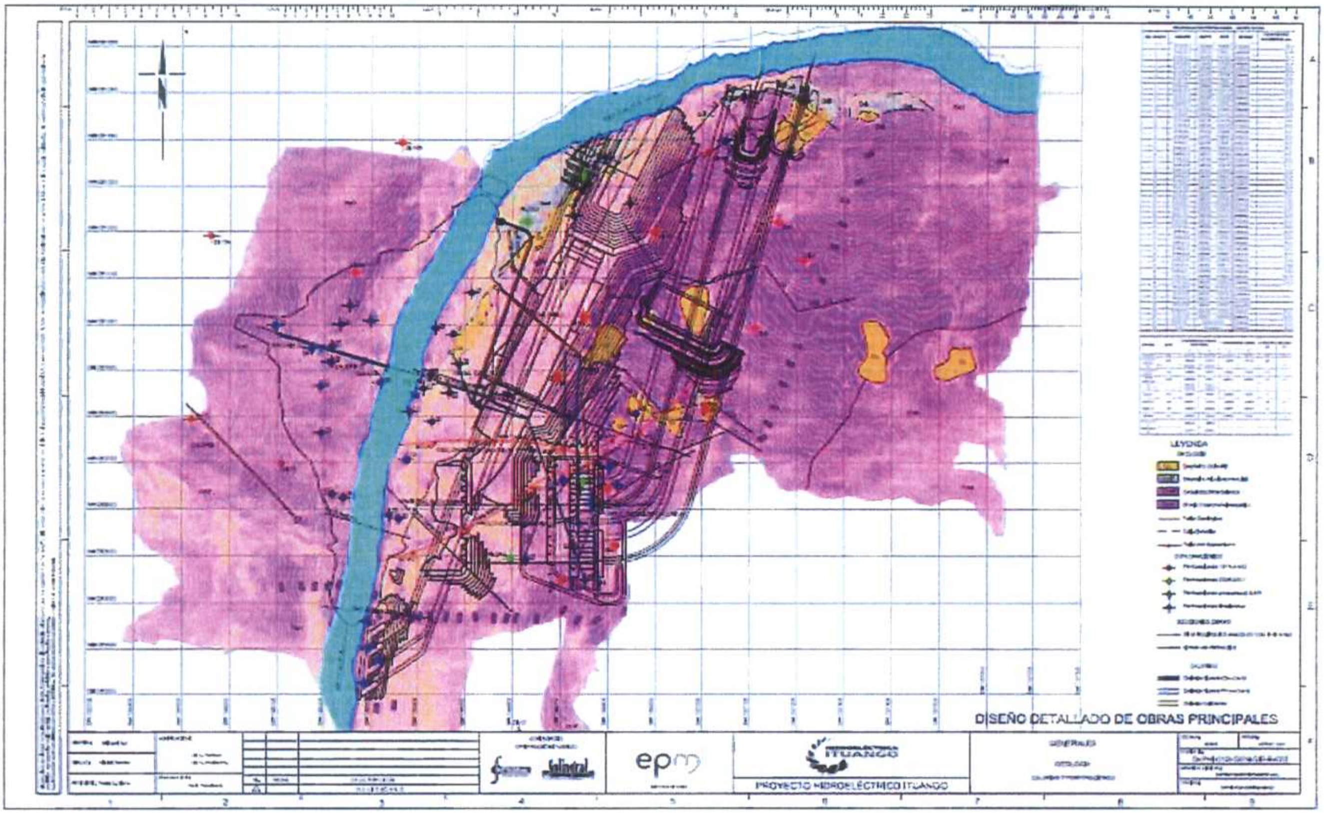


Figure 13. Location of boreholes and other investigations[59]

6.10. The investigations related to the diversion tunnels were concentrated at the upstream and downstream portals, the only investigation conducted approximately on the alignment of the LDT and RDT was a 50 m deep borehole to investigate the foundation of the spillway structure, which does not reach the elevation of the Diversion Tunnels. However, exploration galleries were constructed, and the later geological mapping of other tunnels generated a substantial amount of knowledge for the design of the ADT[60] which was to

[55] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3 Section, 2.4, Page 12
[56] Woodward-Clyde Consultants (1981) Preliminary Study of Geomorphology and Quaternary Stratigraphy Ituango Project, Colombia
[57] Integral (2007) Complementación de la factibilidad
[58] Integral (2007) Complementación de la factibilidad F-PHI-GGS-ANA
[59] Integral (2011) Geología Galerías y Perforaciones DC-PHI-012-GEN-GE-B-020
[60] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, Section 4.5

be constructed using the Observational method[61] which incorporates investigations in the tunnel.

6.11. A geological 'profile' was prepared for the ADT in the 'portal area'. The version presented in the 2018 Geotechnical memorandum was simplified compared to a version dated 2016 that was referenced in a hydrogeological study carried out in 2017 because only one shear zone was shown in the Tenche valley area. The simplified version appears to have also been used in the groundwater model used as part of the 2018 ADT "*lining design requirement*" study[62].

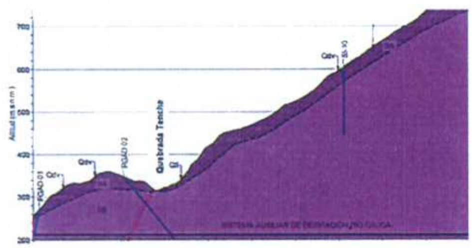
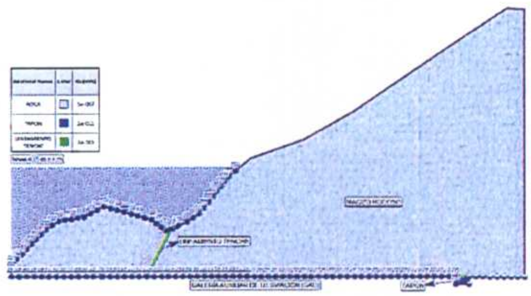


Figura 12. Perfil geológico en la zona del portal de entrada y quebrada Tenche



(a)

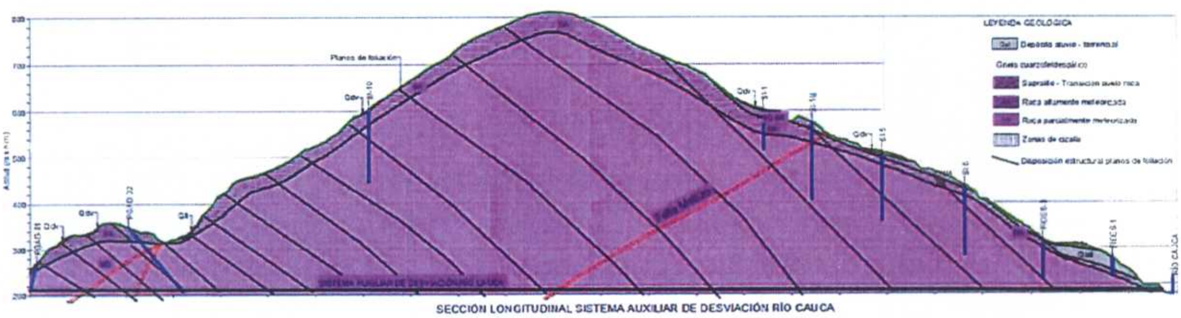


Figura 3. Perfil por el tunel de desviación de SAD.Fuente: Hidroeléctrica Ituango S.A., E.S.P., 2016.

Figure 14.Geological Profile in the ADT portal area, 2018[62,63] and 2017[64]

Seismic Conditions at the Plant Location

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[61] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3, Section 5.5
[62] Integral (2018) Compilation of Design Calculations: I-I-2194-034-REV-01-R0, Page 24
[63] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, Figure 12
[64] Servicios Hidrogeológicos Integrales (2017) Determinación del origen de las aguas infiltradas en el túnel auxiliar de desviación, Figure 3

6.12. Seismic events are vibrations that occur in the earth due to geological phenomena such as movement along fault lines. This area of Colombia is on a seismically active zone. The nearby town Ituango is in an intermediate seismic hazard zone[65] which is defined as having coefficients of peak acceleration or velocity between 0.1 and 0.2. Similar vibration signals to seismic events can occur when a tunnel collapses, or a major landslide occurs but with much lower coefficients of peak acceleration or velocity. Ground vibrations were measured at the site on a regional and local scale which was able to distinguish between seismic ground motions and ground vibrations from other sources.

Geological and Geotechnical Properties for the LDT and RDT

6.13. The 2010 Geological and Geotechnical Characterization Report[66] presented the basic rock material and rock mass parameters measured or derived from data collected from the ground investigations for the Project.

6.14. The rock mass quality was classified using a conventional empirical system, referred to as the NGI Q system, values for the estimation of rock mass parameter Geological Strength Index (GSI)[67] were also provided[68] (Table 1) and the percentage distribution of these support types (Table 2):

Type of Support	Q value	GSI value
I	>30	>65
II	4 - 30	56 – 65
III	0.15 - 4.0	27 – 56
IV	<0.15	<27

Table 1. Rock quality description and corresponding NGI Q-value and GSI values for the LDT and RDT

Terrain Type	Percentage occurrence [%]		
	LDT&RDT (Predicted)	LDT (Actual)	RDT (Actual)
I	50	0	0

[65] Asociación Colombiana de Ingeniería Sísmica (2010) Reglamento Colombiano de Construcción Sismo Resistente NSR-10
[66] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3
[67] **GSI** is the Geological Strength Index. It is a rock mass characterization system commonly used in engineering.
[68] Integral (2012) Túneles Principales Secciones de Excavación y Revestimiento Secciones Típicas DC-PHI-031-TDV-SE-C-020-R2

Terrain Type	Percentage occurrence [%]		
	LDT&RDT (Predicted)	LDT (Actual)	RDT (Actual)
II	20	72	76
III	22	25	24
IV	8	3	0

Table 2. Distribution of support type for the LDT and RDT Geological and Geotechnical Properties for the ADT

6.15. A design report for the ADT that pre-dated its construction was not provided. However, we were provided with a compilation of documents dated August 2018, that included the Geotechnical Calculation Memorandum[69] that contained the geological and geotechnical assessments for the ADT using data obtained during the construction of the LDT and RDT. The data were similar to the 2010 document[66].

Rock Mass Classification for the ADT

6.16. The 2018 Geotechnical Calculation Memorandum explained that statistical analysis of the Q values contained in the mapping of the LDT and RDT, construction galleries 1 and 2, the Powerhouse access tunnel, the upper and lower construction galleries to the Powerhouse and the auxiliary construction galleries was used to determine the Q values for the ADT. The Q values for the ADT were different to the LDT and RDT for the four terrain types (Table 3).

Terrain type	Q value
I	>20
II	3.0 - 20
III	0.3 - 3.0
IV	<0.3

Table 3. Q values developed for the ADT[70]

6.17. An objective of the analysis of the existing Q values was stated to be an increase in the amount of ADT that could be classified as terrain type I compared to the LDT and RDT[71].

[69] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0
[70] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, Table 8, Page 5.2
[71] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, Page 5.1

6.18. Predictions were made of the likely percentage occurrence of the ground types for the ADT based on the tunneling experience, including the crossing of the Tocayo and Mellizo faults and the "*Tenche deposit*" *(sic*[72]*)* (Table 4).

Terrain Type	**Percentage occurrence [%]**		
	ADT (Predicted)	LDT (Actual)	RDT (Actual)
I	5	0	0
II	60	72	76
III	30	25	24
IV	5	3	0

Table 4. Percentage occurrence of predicted ground type for the ADT[73] and actual for LDT & RDT[74]

[72] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, Page 5.3
[73] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, Figure 21, Page 5.3
[74] Ingetec-Sedic, Informe Final de Obra Contrato, August 2014, PHI-IFF-LC1-001-R0, Figure 4-26 and 4-27, Page 94 and 95

7. DESIGN

7.1. This chapter contrasts the designs of the LDT, RDT and ADT in general, and specifically the differences between the support recommendations for terrain type III in the LDT-RDT and in the ADT at the location the collapse is interpreted as having happened. The discussion about the design of the LDT and RDT is relevant because these tunnels were designed and built before the ADT, and the Asesoria has stated that information and experience from these tunnels in four years of operation was used to design the ADT[75].

7.2. The LDT diverted water from February 2014 to October 2017 and the RDT from February 2014 to February 2018. The tunnels reached a maximum pressurization of 37.5 m of water above the invert on May 17, 2017. The post-closure inspection of the tunnel found them to "show little or no effect" following operation[76] although there were reports of instability and difficulties controlling the roof and overbreak during construction that was greater than would normally be expected[77].

Design of the LDT and RDT

7.3. The LDT and RDT have D-shaped internal geometry with a nominal height and span of 14 m. However, the excavated span and height vary according to support type to achieve the same flow cross section in all ground types[78], for example terrain type III required an excavation 14.20 m wide and 14.20 m high (Figure 15) that after installation of the support provided a flow area of 14 m by 14 m.

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[75] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, section 5.1
[76] Unlabeled, Undated Report "Inspección Túneles de Desviación Originales" Page 1 of 4
[77] Ingetec-Sedic (2014) Informe Final de Obra de Contrato PHI-IFF-LC1-001-R0, Pages 76 and 77
[78] Integral (2012) Túneles Principales Secciones de Excavación y Revestimiento Secciones Típicas DC-PHI-031-TDV-SE-C-020-R2

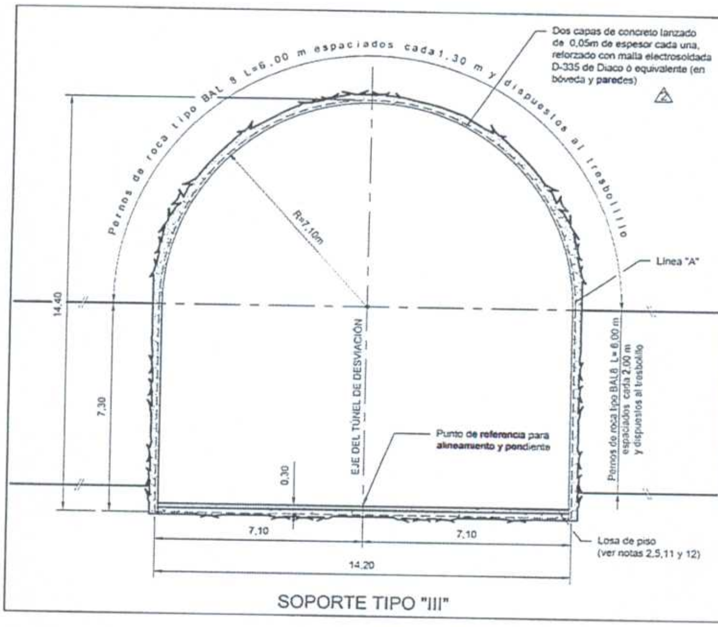

Figure 15. Left and Right diversion tunnels geometry

7.4. The configuration of the tunnel support on the various revisions of the design drawing for the LDT and RDT[79,80,81] matches the general description of the tunnel support in the geotechnical characterization report[82]. The support presented on the last revision of the design drawing of the LDT and RDT is essentially the same as the one described in the LDT and RDT design report[83] except for the longitudinal spacing of the rock bolts in Support Type IV which is 1.0 m. These reports stated that evaluation of the stability and the design of the tunnel support used the following methodologies[82].

- NGI Q empirical method to estimate the tunnel support
- Convergence-confinement method
- Mathematical modeling using finite elements method
- Kinematic wedge analysis

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[79] Integral (2011) Túneles Principales Secciones de Excavación y Revestimiento Secciones Típicas DC-PHI-031-TDV-SE-C-020-R0
[80] Integral (2012) Túneles Principales Secciones de Excavación y Revestimiento Secciones Típicas DC-PHI-031-TDV-SE-C-020-R1
[81] Integral (2012) Túneles Principales Secciones de Excavación y Revestimiento Secciones Típicas DC-PHI-031-TDV-SE-C-020-R2
[82] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3
[83] Integral (2014) Túneles de Desviación Memoria de Calculo Geotécnico, I-M-2194-031-GYG-01-R0

- Observational method[84]

7.5. The tunnel support was presented in Rev 0 of design drawing[85,86,87] that details the support for different Q-values and GSI ranges.

7.6. Additional tunnel support recommendations were provided during design meetings between EPM, Asesoria and Interventoria and revised design drawings were issued. The minutes of the meetings show that the Asesoria was discussing support recommendations that were not included in the revised drawings. For example, in the meeting[88] minutes of June 28, 2012 the use of steel fibers was approved as an alternative to the D-188 steel mesh for support type II, but this discussion was not reflected in the drawings.

7.7. The tunnel support was made stronger with Rev. 1[89] on July 19, 2012, which was before excavation of the main tunnels started[90]. Table 5 shows a summary of the support changes for the LDT and RDT on Revisions 0, 1 and 2.

Support type	From Revision 0 to Revision 1	From Revision 1 to Revision 2
I	No changes.	No changes.
II	Wall rockbolt length changed from 4.5m to 6.0m.	No changes.
III	Shotcrete total thickness changed from 0.1m to 0.2m. Reinforcement mesh changed from D-355 to D-188.	Shotcrete total thickness changed from 0.2m to 0.1m. Reinforcement mesh changed from D-188 to D-355.
IV	No changes.	Number of shotcrete layers changed from 4 to 2. Total thickness not changed.

Table 5. Support changes for the LDT and RDT design drawings

[84] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3. Section 5.5
[85] Integral (2011) Túneles Principales Secciones de Excavación y Revestimiento Secciones Típicas DC-PHI-031-TDV-SE-C-020-R0
[86] Integral (2012) Túneles Principales Secciones de Excavación y Revestimiento Secciones Típicas DC-PHI-031-TDV-SE-C-020-R1
[87] Integral (2012) Túneles Principales Secciones de Excavación y Revestimiento Secciones Típicas DC-PHI-031-TDV-SE-C-020-R2.
[88] Ingetec-Sedic (2012) Acta de Comité de Diseño y Construcción No 18 CITFS.
[89] Rev 1 07/19/11. Revision of support for terrain type II with anchor length and shotcrete extended to side walls; for type III shotcrete thickness in increased to 20cm and the steel mesh is changed to D-188
[90] Ingetec-Sedic (2014) Informe Final de Obra de Contrato PHI-IFF-LC1-001-R0. Page 342 Main tunnels, not including portals or galleries.

7.8. It is important to note that the February 2017 Asesoria report[91] states that the initial change permitted was from the use of D-335 mesh to two layers of D-188 mesh in the shotcrete in type III support. This was because the Contractor had difficulty bending the D-335 mesh to the irregular shape of the tunnel after blasting and the D-188 mesh was more flexible. The intention of the two layers was to achieve an equivalent amount of steel reinforcement in the shotcrete. Also, the Asesoria could instruct three layers of 0.05 m thick steel fiber reinforced shotcrete could be used.

7.9. The support types for the diversion tunnels were recorded in the annex of meeting[92] minutes of July 26, 2012. According to the annex, a new support type IIIA was created for ground type III with brittle behavior, affected by shear zones, generating overbreak in the tunnel crown, which required spiles[93]. The following two alternatives for support type IIIA were discussed but were not incorporated in the "For Construction" drawings.

- Two 5 cm layers of shotcrete reinforced by D-335 steel mesh and rock bolt type BAR 8
- Three 5 cm layers of shotcrete with steel fibers and rock bolts type BAR 8

7.10. The significant change is the type of rock bolt from a fully cement grouted bolt (BAL) to a fully resin grouted bolt (BAR), because the resin sets in minutes providing almost instantaneous reinforcement of the rock mass along the full length of the bolt, whereas the cement takes several hours to set, in which time the rock mass can relax and the tunnel face may have been advanced.

7.11. However, there is no evidence in the documents provided for the type IIIA support alternatives being used in a construction drawing or implemented in the tunnel.

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91 Integral Informe Memorias de Calculo Desviación Auxiliar I-I2194-2017_GAD

92 Ingetec-Sedic (2012) Acta de Comité de Diseño y Construcción No 21 CITFS

93 **SPILE** - A spile is a steel bar like a rock bolt that is installed sub horizontally ahead of the tunnel face to pre-support the ground

7.12. The same annex included a proposal to split the top heading for the different terrain types for rock mass types IIIA and IV as follows[94].

- A single or a two-face top heading for terrain type IIIA
- Three face top heading in terrain type IV

7.13. The observational method was employed in the LDT and RDT. The following are examples of the OM in use.

i) In the RDT at k 0+474[95] the kinematic wedge analysis was revised based on field observations, and additional activities were recommended by the designer to be implemented in the field.

ii) Review of the design for the RDT at Ramal 1 k 0+175[96] and various chainages on both RDT and LDT[97], when the designer used numerical methods and convergence measurements to verify the design and provided recommendations to be implemented in the field.

iii) The September 2012 correspondence include actions to be taken following review of convergence data, providing a threshold for convergence of 0.5% beyond which the issues corrective measures must be investigated. Furthermore, an area influenced by a fault zone with convergences greater than 0.5% was identified so procedures were required to control convergence but there was concern that the convergence readings only started 20 m from the face, which they state means that even these readings do not reflect the process of elastic relaxation of the ground.[98]

iv) The Asesoria also reviewed the design after receiving the regular instrumentation reports from the contractor in 2013.[99]

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[94] In the documents provided there is no record of subdividing the excavation heading being used in a construction drawing or implemented in the tunnel
[95] Integral (2012) Letter D-PHI-COP-130-2012
[96] Integral (2013) Letter D-PHI-COP-015-2013
[97] Integral (2013) Letter D-PHI-COP-183-2013
[98] Integral (2012) Letter D-PHI-COP-058-2012
[99] Integral (2013) Letter D-PHI-COP-200-2013

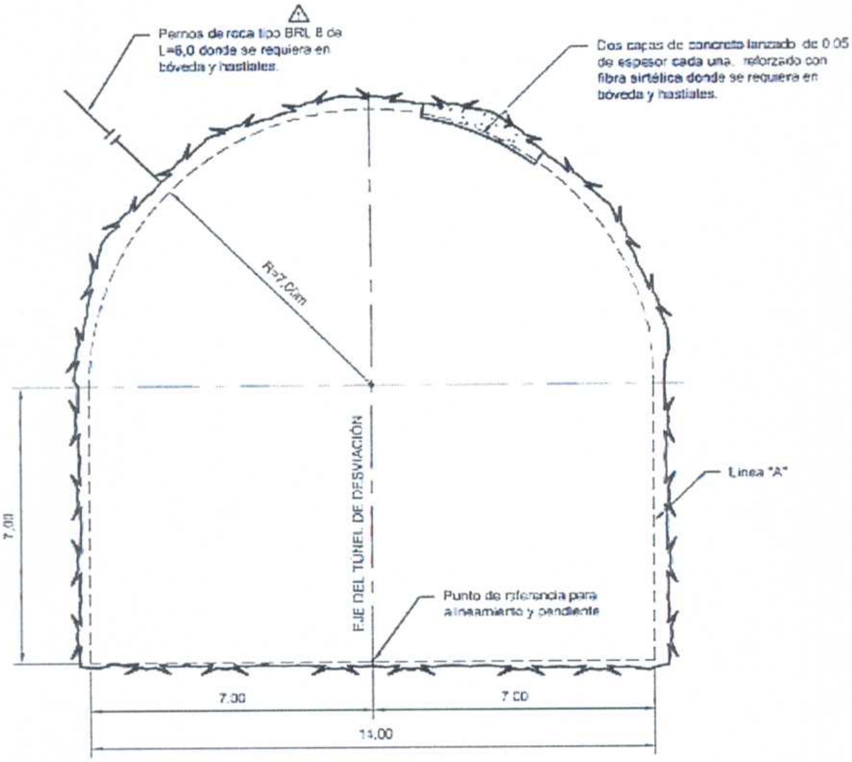
v) Prior to commissioning the LDT and RDT, the Designer presented a report on the analysis of the behaviour of the excavation support as the date for river diversion was approaching[100].

7.14. The sequence of discussions and changes demonstrated that the Project recognized the need for heavier tunnel support for poorer ground, including shear zones, and a demonstration of the effective use of the observational method in the LDT and RDT.

Design of the Auxiliary Diversion Tunnel - ADT

7.15. The ADT in Sector 2 had a D-shape cross section, with a nominal internal base of 14 m and total internal height of 14 m, and the same flow cross section as the LDT and RDT (Figure 16). The ADT in Sector 4 had a varying D-shape section, which modified the area of flow[101]. The excavation line depended on the terrain type and was 14.0 m for terrain type I, 14.2 m for types II and III, and 14.6 m for terrain type IV[102].

[100] Integral (2013) Letter D-PHI-COP-491-2013
[101] Integral (2017) Futuro túnel de descarga No. 4. Excavaciones Secciones Típicas D-PHI-034-TUN-EX-C-030-R4, Note 3.
[102] Integral (2016) Túnel Excavaciones Secciones Típicas D-PHI-034-TUN-EX-C-010-R1



SOPORTE PARA TERRENO TIPO "I"
ESCALA 1:100

Figure 16. Auxiliary Diversion Tunnel geometry for Sector 2

7.16. The design of the ADT included information from the underground works of the Project that had already been excavated and geologically mapped. The structural geology records and the monitoring results from adjacent completed underground excavations were compiled and analyzed to design the ADT.

7.17. It is our opinion that in general, recognized international procedures were used to characterize the rock mass and its discontinuities for the ADT, which are summarized as follows.

- The rock mass was characterized using the NGI Q system[103]

- Geomechanical shear strength parameters of the rock mass necessary for design were derived following the Hoek and Brown criteria, postulated from the Geological Strength Index, GSI, the unconfined compressive strength, UCS, the deformability

103 Using the Q-system – Rock Mass Classification and Support Design, NGI, 2015. The method was originally proposed by Barton and Grimstad.

modulus, E and factors related to the rock mass such as mi, mb, a, s (all defined by Hoek)[104]

- Shear strength parameters of the rock mass discontinuities were postulated following Barton's propositions

- The damaged zone around the tunnel when excavating by the drill and blast method was considered

- Recognized bibliographic references were used

7.18. The Q value ranges were used for defining support, classifying ground types, verification of the support type, and estimating geomechanical parameters for mathematical analyses. We note that the Q ranges for defining support were different to those used to estimate the geomechanical parameters and the mathematical analyses to verify the support in Sector 2 of the ADT (Table 6). The consequence of this difference was that the verification of support requirements using mathematical methods would apply different geomechanical parameters to those automatically associated to the Q-ranges used to define support types, particularly at the lower end of the ranges[105].

"Tipos de Terreno" / Terrain Type	Q-value ranges defined in the ADT design report	Q-value ranges used for Geomechanical parameters in the ADT design	Q-value ranges presented in the design drawing for the ADT
	I-M-2194-034-GYG-01, Table 8	I-M-2194-034-GYG-01, Table 12	D-PHI-034-TUN-EX-C-010-R1
I	Q>20	Q>25	Q>20
II	3<Q<20	3<Q<25	3<Q<20
III	0.3<Q<3	0.1<Q<3	0.3<Q<3
IV	Q<0.3	Q<0.1	Q<0.3

Table 6. Different Q value ranges for Terrain Types for the ADT design

[104] Using commercially available RockLab software.
[105] Prepared from Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0.

7.19. The consequence of this difference is illustrated below for terrain type III which shows that the Q value ranges could produce two possible support recommendations when applied to the Q method chart, depending which of the sources from Table 6 is used.

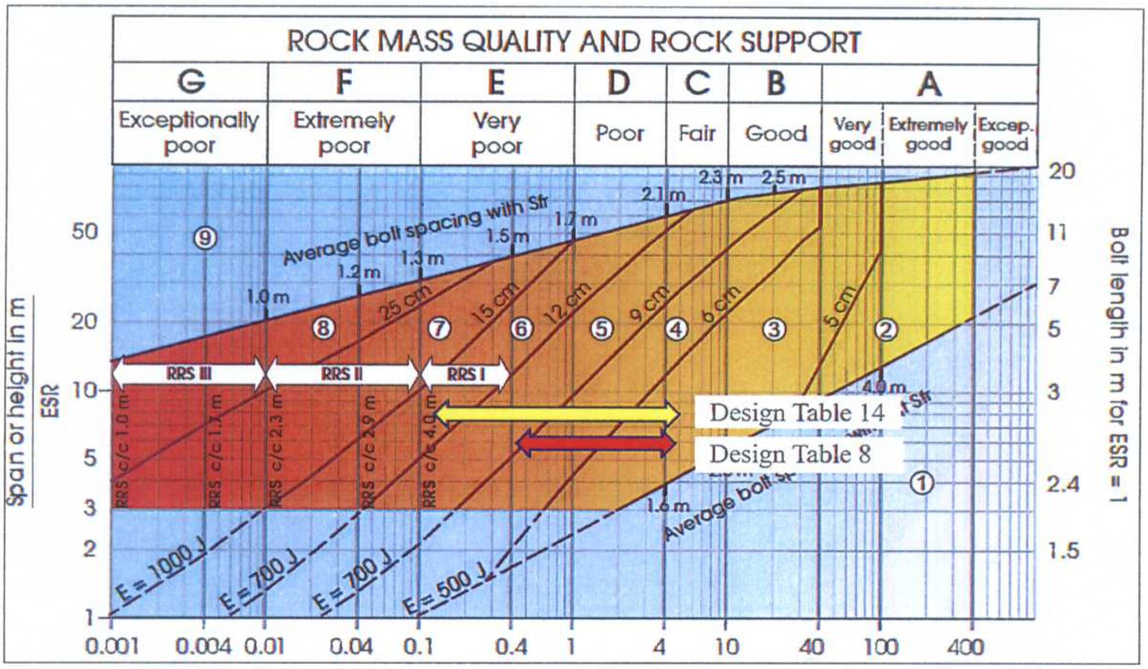


Figure 17. Q-Method design chart – range of values for terrain type III Sector 2 ADT.

Geomechanical Parameters

7.20. The shear strength parameters mb, s, a and deformability parameter, Em for the ADT design were estimated using Hoek and Brown methodology, with input data GSI, Ei[106], mi[107], and the blast damage D[108], derived from the design data bank[109] using Rocklab software. The values were converted to those of the rock mass following Hoek and Brown methodology for each of the terrain types. The Q ranges in Table 7 for terrain types were taken from the compilation of design calculations.

Terrain type	Q (a)	GSI (b)	mb (c)	s (d)	a (e)	Em [GPa] (f)	D (g)
I	>25	>70	8.002	0.0204	0.5020	12.63	0.7
II	3-25	55-70	5.613	0.0067	0.5040	8.17	0.4

[106] Ei is the intact rock deformability. Design values range is 15 GPa to 20 GPa.
[107] m_i is the material constant, adopted equal 28±3 in design.
[108] As used in the geological drawings
[109] Integral (2010) Caracterización Geológica y Geotécnica D-PHI-CCE-ADM-C0082 Rev. 3

Terrain type	**Q (a)**	**GSI (b)**	**mb (c)**	**s (d)**	**a (e)**	**Em [GPa] (f)**	**D (g)**
III	0.1-3	30-55	3.19	0.00127	0.5114	3.19	0.1
IV	<0.1	<30	1.63	0.00041	0.5223	1.63	0

Table 7. Rock mass properties used for the design of the ADT[110]

7.21. The Experts carried out an exercise to correlate the Q and GSI values (column a and b) with the design shear strength parameters (columns c to f) as used in a normal design process. Appendix E presents our verification of the compatibility between Q, GSI and shear strength parameters used in the ADT design. In this verification there were incompatibilities between the Q-value ranges, the equivalent GSI ranges and the calculated shear strength design values, which means that there are inconsistencies in the design process.

7.22. Rock mass classification Q values from the tunnels and caverns constructed before the ADT were analyzed statistically[111], leading to rock quality ranging from very poor to very good, with the mean value at the upper boundary of fair rock mass quality. The Q method was used as a first estimate of the tunnel support for the ADT using the Q-ranges, followed by the convergence-confinement method, elasto-plastic numerical analyses and kinematic wedge analysis. The wedge analysis was based on the discontinuities mapped in other underground works.

7.23. A detailed longitudinal geological-geomechanical profile of the tunnel indicating predicted terrain types and significant geological features that may be hazards, was not in the documents provided, and which we understand does not exist.[112] Such a document is expected because it is a part of the Observational Method of tunneling and risk management for tunnels.

Empirical Analysis

[110] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, Table 12, Page 5.6
[111] A data bank of 1542 readings is associated with these underground excavations
[112] Integral (2019) Letter report D-PHI-CCE-ADM-1-C4490, dated 8th July 2019, Page 2 of 6.

7.24. The Q-method was used to make a preliminary estimate of the necessary support for the ADT. The Q-value ranges used in design are presented in Table 3. A value of ESR[113] of 1.3 was used[114], which is lower than the 1.6 suggested for "Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks), water supply tunnels, pilot tunnels, drifts and headings for large openings". This is a conservative value for this parameter.

7.25. However, the support shown on the ADT design drawings[115] was not based on the minimum Q value of the range for each terrain types. A consequence of this was that the preliminary selection of support was lighter than recommended by this method for the lower part of the range. Using the minimum Q value of a range is normal industry practice.

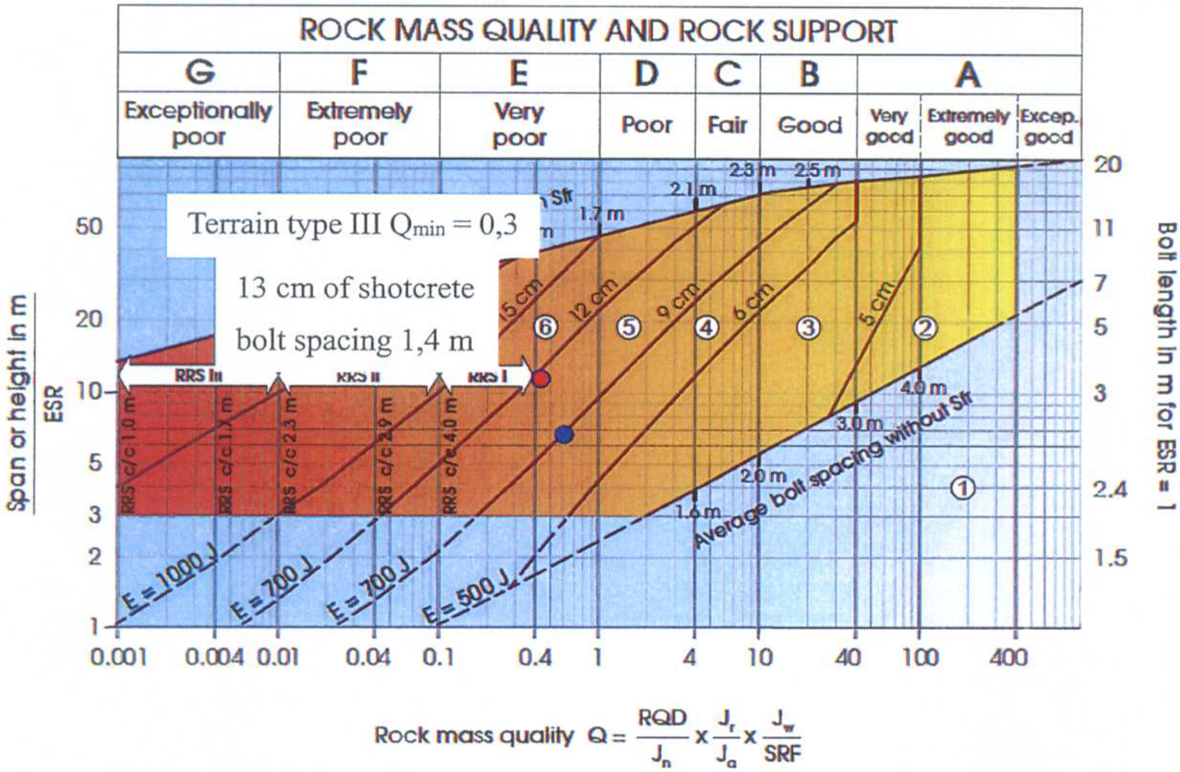


Figure 18. Diagram showing the different support selection for the design Q (blue dot) versus the minimum Q (red dot)

[113] Excavation Support Ratio, as defined in Using the Q-system Rock mass classification and support design, NGI 2015
[114] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0
[115] Integral (2016) Túnel Excavaciones Secciones Típicas D-PHI-034-TUN-EX-C-010-R1

7.26. If the Q method had been applied in accordance to normal practice the recommended support for terrain type III would have been heavier than the actual design drawing[116], Table 8.

Class / Type	Loc.	Design Drawing D-PHI-034-TUN-EX-C-010 Rev 1		Q-Method with minimum Q-values for ranges	
		Shotcrete thickness	**Rockbolt spacing**	**Shotcrete thickness**	**Rockbolt spacing**
I	Crown	10cm where required	BRL 8 length 6m where required	5 or 6cm	2.5m
	Wall	10 cm where required	BRL 8 length 6m where required	unsupported	where required
II	Crown	10cm reinforced with synthetic fibre	BRL 8 length 6m spaced 1.8m	7cm	2.0m
	Wall	10cm reinforced with synthetic fibre	BRL 8 length 6m spaced 2.5m	5 or 6cm	2.2m
III	Crown	10cm of shotcrete with synthetic fibre and steel mesh D-188 where required	BRL 8 length 6m spaced 1.5m	13cm	1.4m
	Wall	10cm of shotcrete with synthetic fibre	BRL 8 length 6m spaced 2.0m	8cm	1.6m
IV	Crown	10cm of shotcrete with synthetic fibre. Steel mesh D-188 where required in crown. HBE 160 steel profile spaced 1,0m. Floor slab		Different support propositions, different from designed support (see figure below, with support types applicable to this Q-Range)	
	Wall				

⑥ Fibre reinforced sprayed concrete and bolting, 12-15 cm + reinforced ribs of sprayed concrete and bolting. **Sfr (E700)+RRS I +B**

⑦ Fibre reinforced sprayed concrete >15 cm + reinforced ribs of sprayed concrete and bolting. **Sfr (E1000)+RRS II+B**

⑧ Cast concrete lining. **CCA** or **Sfr (E1000)+RRS III+B**

Table 8. Comparison between design support and support when using Q minimum showing 30% difference in shotcrete thickness and 25% difference in the number of bolts.

Convergence Confinement Analysis

7.27. The compilation of design calculations[117] show the design used a one-dimensional convergence-confinement method to estimate loads and displacements of the tunnel support and the surrounding ground, in an equivalent way as used in for the LDT and

[116] Integral (2016) Túnel Excavaciones Secciones Típicas D-PHI-034-TUN-EX-C-010-R1
[117] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0, I-I-2194-034-REV-01-R0, I-M-2194-034-EST-01-R0, and I-M-2194-034-HID-02-R0

RDT. Displacements predicted in the design report varied between about 8 mm and 53 mm, depending on the terrain type from I to IV (Table 9).

Support type / ground type	Convergence	Zone of plastification
Type I	7.9 mm / 0.16%	0.16 m
Type II	18.0 mm / 0.25%	0.22 m
Type III	26.4 mm / 0.37%	0.70 m
Type IV	53.5 mm / 0.75%	1.20 m

Table 9. Predicted convergences from the analysis for the ADT

Numerical Analysis

7.28. The numerical method used to analyse the tunnel excavation and simulate the behaviour of the surrounding rock mass was the finite element method using commercially available specialised software[118]. The analyses were performed using 2-D models, considering plane strain which is routine design procedure.

7.29. The displacements and the zones of rock mass relaxation and plasticisation around the periphery of the tunnel excavation were predicted for the four terrain types[119], as shown in Table 8. Diagrams of axial and shear forces and bending were used to verify acting stresses in the shotcrete.

Terrain type	Convergence [%]
I	0.19
II	0.31
III	0.20
IV	0.24

Table 10. Predicted displacements from the analysis for the ADT

7.30. It has not been possible to verify the numerical modelling because it includes too many inconsistencies and missing information. These are discussed in Appendix C. The consequence of these inconsistencies is that the analysis did not provide reliable convergence values for the tunnel for verification and validation of the support using the

[118] Rocscience Inc. (Undated), Phase2 Version Unknown - Finite Element Analysis for Excavations and Slopes.
[119] Integral (2018) Compilation of Design Calculations: I-M-2194-034-GYG-01-R0

observational method nor did it fulfil the design objective that it would be used to verify the support recommendations of the empirical methods.

7.31. We carried out numerical analysis that represented the design and the ground conditions at chainage 0+540 and produced the following convergence predictions, see hypothesis in Appendix E (Table 9).

Terrain type	Convergence [%]
I	-
II	-
III	0.46
IV	0.40

Table 11. Experts simulations predicted displacements for ADT design geometry

7.32. It is notable that our prediction is almost at the limit of convergence of 0.5%, which as discussed earlier (7.13.iii)) triggered a review and possible remedial measures when it occurred in the original diversion tunnels.[120]

7.33. The technical Specification for the design of concrete was to AASHTO and ACI which prescribes verification using reduction factors applied to each of the variables (loads and resistances). Whereas the verification presented by the Asesoria was developed using a single global factor of safety, and so not in accordance with the TS. However, the verification methodology used by the Asesoria was based on a recognized bibliography for initial verification.

7.34. Some of the input to the numerical analysis are different to the ADT design report[121]. For example, the crown rock bolt spacing for Type III used in the numerical simulation is more robust than the design because it was reduced from 1.5 m to 1.3 m.

Kinematic Wedge Analysis

7.35. The Asesoria used the discontinuity families mapped in the other tunnels to estimate the rock mass wedges that were kinematically possible to form in the tunnel cross section in terrain type II only; normal practice would be to verify ground types I, II and III. The

[120] Integral (2012), Letter Report D-PHII-COP-058-2012, page 8 of 8
[121] Integral (2018) Compilation of Design Calculations I-M-2194-034-GYG-01-R0

rock mass wedges were analysed using conventional static procedures implemented in commercially available software[122] to achieve a FS of 1.5[123]. Additional analysis dated after the event of April 28, 2018 was carried out for submerged conditions up to a pressure of 5 ton/m^2 (5 m of water head) which show the crown wedges fail at this water pressure[124].

7.36. The Asesoria also stated in its root cause analysis that a Kinematic Wedge Analysis using the discontinuities in the area yields a Factor of Safety for the support of 1.9, and it is capable of supporting a water pressure between 4 and 8 m of equivalent water head[125]. The Asesoria presented another Kinematic Wedge Analysis using the shear zones, discontinuities and foliations that yields a factor of safety of 1.5[126], and less than 1.0 for partially installed support[127]. This means that all the support had to be installed before advancing the tunnel for the wedges to be stable and this is discussed further in Section 8.

Observational Method

7.37. It is important to know that the instrumentation and monitoring for the LDT and RDT was by the CTIFS contractor, that used a specialist sub-contractor to take the readings and report on the data. These reports were of the frequency and scope that would be expected. The responsibility for taking the readings and interpretation of the data was with the Asesoria for the ADT and reports were not provided that had the detail and timeliness as when the CTIFS was responsible. A constant in this process was that the Asesoria was responsible for the interpretation of the data.

7.38. There are examples of the OM for the ADT[128] such as revision of the kinematic wedge analysis as was done for the LDT and RDT.

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[122] Rocscience Inc. (2003) Unwedge Version 3.0 - Underground Wedge Stability Analysis; Fine spol s.r.o. (2018) Geo5 Rock Stability and others.
[123] Integral (2018) Compilation of Design Calculations I-M-2194-034-GYG-01-R0, Page 6.33
[124] CITE
[125] Integral (2019) Taponamiento GAD - Estudio de Causa Raíz. I-I-2194-034-TAP-01, Pages 79 and 80
[126] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2, Pages 7.179 and 7.180
[127] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2, Figures 7-16 and 7-17, Page 7-184.
[128] Integral (2016) Letter D-PHI-COP-1011-2016

Comparison of LDT-RDT and ADT for Terrain Type III

7.39. The design of the diversion tunnels evolved over time. The LDT and RDT were strengthened at different points in time prior to and during the construction phase such that the as built tunnel was stronger than the tunnels envisaged at design stage (¶7.7). The opposite occurred with the ADT. In a design spreadsheet[129], presumed to date from earlier than the ADT basic design, the support in the crown and sidewalls was identical to the last revision of the LDT and RDT then reduced for the basic design of the ADT[130] and was further reduced for the construction phase. The following are the specific changes that ultimately led to this difference.

i) The rockbolt spacing in the crown was increased from 1.3 m to 1.5 m which meant that about 25% fewer rockbolts were installed in the same section of tunnel crown.

ii) The steel mesh for the ADT consisted only of one layer of D-188 mesh as opposed to the heavier D-355 mesh. The reinforcement of ADT's shotcrete was 70% of the steel reinforcement used on the LDT and RDT which meant less resistance to loads and displacements imposed by the ground.

iii) The zone of blast damage parameter D for the ADT was scaled according to the terrain type and varied from 0 for terrain type IV which is no blast damage because the tunnel was assumed to be mechanically excavated, to 0.7 for terrain type I which was assumed to suffer the most blast damage, and with a value of 0.3 for Type III indicative of a low degree of blast damage. For the LDT and RDT multiple references in the geotechnical characterization report and the LDT and RDT design report either indicate similar hypotheses to those of the ADT or that D was always assumed as 0.7, which would lead to more conservative design.

[129] Integral (Undated) Análisis cinemático de cunas - Galería de Desviación Auxiliar Filename Anexo 3-MC-GDA-CUÑAS-R1

[130] Integral (2015) Soportes de excavacion y revestimiento Secciones tipicas del tunel Basic Design D-PHI-034-TUN-EX-C-010

iv) The steel mesh for the ADT was "where required" and only in the crown, whereas it was systematic for crown and sidewalls for the LDT and RDT. An example of where required was a shear zone or fault, such as was encountered at around 0+540.

Analysis of the Need of Lining for Hydraulic Reasons

7.40. Design of the ADT included a specific document focusing on the "Analysis of the Need of Lining"[131] which stated that the average velocity of the water passing through the tunnel will be approximately 5 m/s, varying between 3.5 m/s and 17.5 m/s. These velocities are presented as compatible with a list of projects in the compilation of design calculations. The Asesoria relied on this to decide that no special lining was needed except a concrete floor to protect the tunnel from the effects of flowing water in both terrain type IV and shear zones[132]. In the event no concrete floor was installed in the shear zone around 0+540. This contrasts with the LDT and RDT where a concrete floor was installed for about 70% of the tunnels including all areas where shear zones were mapped[133].

7.41. The design required a means of preventing floating debris entering the ADT. We understand that the Technical Committee decided that this was not required at the intake portal. However, a barrier upstream of the intake portal was designed and installed. However, we have seen undated photographs that show what appears to be a netting type barrier across the lower half of the portal. We were not provided with the design or construction records of this barrier. These photographs show a substantial amount of floating debris against the portal and netting, indicating that the upstream barrier must have been breached at some time that we have not been able to determine from the records provided.

[131] Integral (2018) Compilation of Design Calculations: I-I-2194-034-REV-01-R0
[132] Integral (2010) Manuales de Caracteristicas for the Sistema de Desviacion del Rio, D-Phi-CCE-ADM-C0314, Section 3.1.3.3, Page 3.1.7
[133] Ingetec-Sedic (2014) Informe Final de Obra de Contrato PHI-IFF-LC1-001-R0

8. CONSTRUCTION OF THE ADT

Excavation Profile

8.1. The profile of the excavation defines the shape of the ADT. The design profile of the excavation is delineated and defined by Line A[134,135]. The technical specifications of the limit of the excavations for the underground works is not inside Line A, and should not excavate outside Line A, as translated below.

> EPM. La línea "A" de excavación que se muestra en los planos corresponde a aquella dentro de la cual no podrá quedar material alguno sin excavar. No se permitirá que
>
> Free translation: The Line "A" of the excavation shown in the drawings corresponds to the one within no material can be left unexcavated.
>
> El Contratista no deberá excavar por fuera de las líneas y pendientes mostradas en los planos u ordenadas por escrito por EPM. Serán por cuenta del Contratista las
>
> Free translation: The Contractor should not excavate outside the lines and slopes shown on the drawings or as ordered in writing by EPM.

Figure 19. Line "A" of Excavation defined in the Technical Specifications.

8.2. The tunnel was surveyed by the Contractor after blasting and scaling of loose rock, and the information given to the Supervision for record keeping. The survey records provided were dated after completion of the tunnel (Figure 20). The green line shows the achieved excavation profile, and the red line shows the design excavation profile (a complete set of survey records for Sector 2 is included in Appendix D).

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[134] Integral (2018) Tunel Excavaciones Secciones Tipicas D-PHI-034-TUN-EX-C-010-R2.
[135] EPM (2012) Especificaciones Técnicas de Construcción, Technical Specifications

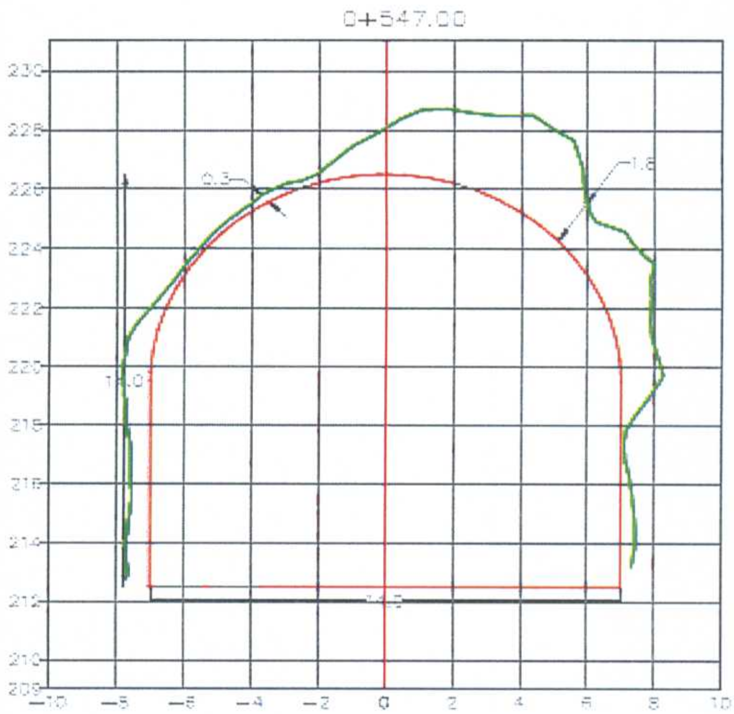

Figure 20. Comparison between Designed and As-Built cross section at 0+547

8.3. The magnitude and location of the overbreak meant that the designed vertical sidewalls of 7 m were actually as high as 16 m in places such as at 0+547 as shown in Figure 20, which is more than twice the design vertical dimension. The TS stated that if in EPM's opinion the overbreak must be backfilled with concrete, steel reinforced concrete or steel fiber reinforced shotcrete, then the Contractor must backfill the overbreak at their own cost[136]. The design would not perform as intended with the overbreak shown in Figure 20 and Figure 21, but this backfilling was not carried out. Additional examples of overbreak are presented in Appendix D.1. The Supervisor has not stated if the survey was done before or after the installation of the shotcrete. Thus, we understand it represents the tunnel after the installation of the shotcrete.

[136] EPM (2012) Especificaciones Técnicas de Construcción, Technical Specifications, section 3.1.1.3

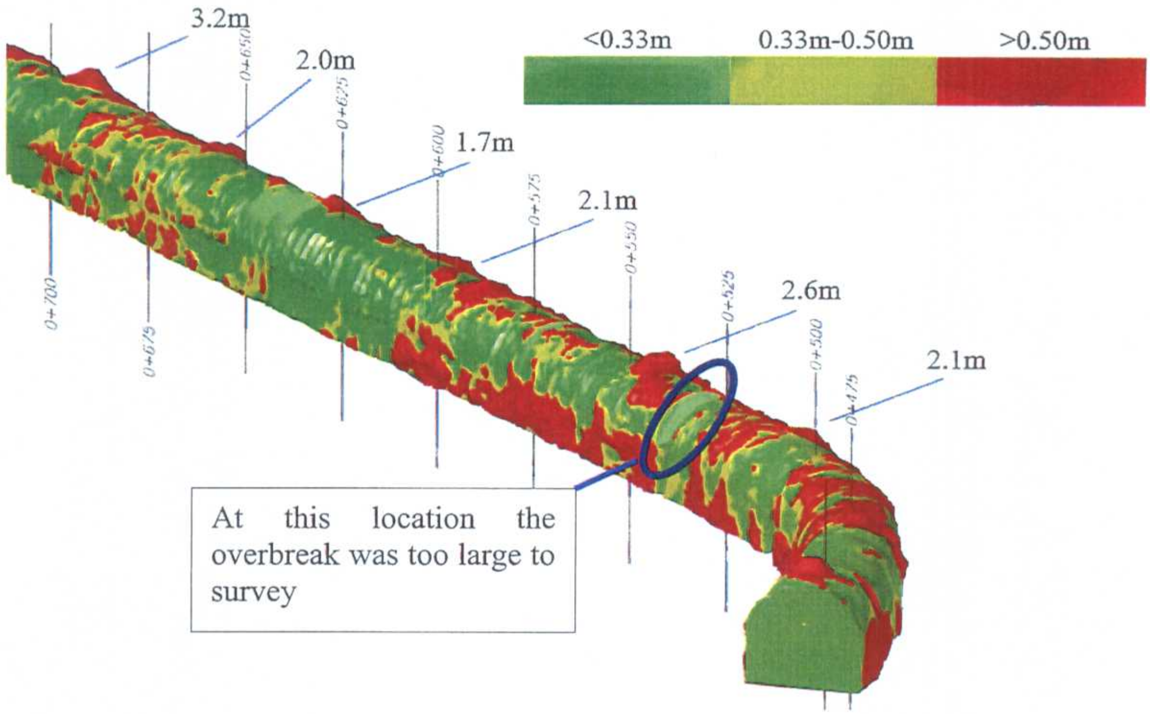

Figure 21. Isometric View of Overbreak on ADT from Upstream Portal to k 0+700

Rock Mass Classification and Support Selection during Construction

8.4. The Asesoria was responsible for the classification of the rock mass to select the supports. The Contract stated that the classification will be carried out by the Contractor and the Asesoria in a format to be signed no later than 24 hours following an advance. In the case of a dispute the details will be presented to EPM no more than 72 hours later. EPM may order the works required for the Contractor to execute and continue with their normal works. Any decision by EPM does not relieve the Contractor of its responsibilities for the excavation and support of the underground works.[137]

8.5. The exposed rock was classified into four terrain types (Types I to IV) following the design method which was the NGI Q-system, as indicated on the Drawings[138]. The records prepared by both the Asesoria[139] (Figure 22) and the Contractor (Figure 23) included a minimum, maximum and weighted average Q value and the latter was used

[137] EPM (2012) Especificaciones Técnicas de Construcción, Section 3.1.1.2
[138] Integral (2016) Túnel Excavaciones Secciones Típicas D-PHI-034-TUN-EX-C-010-R1
[139] Integral (2016-2017) Registro geológico - geotécnico de clasificación del macizo rocoso

to determine the required support. The face of the ADT in Sector 2 was not mapped after each blast. The records for each party are contained in Appendix D.2.

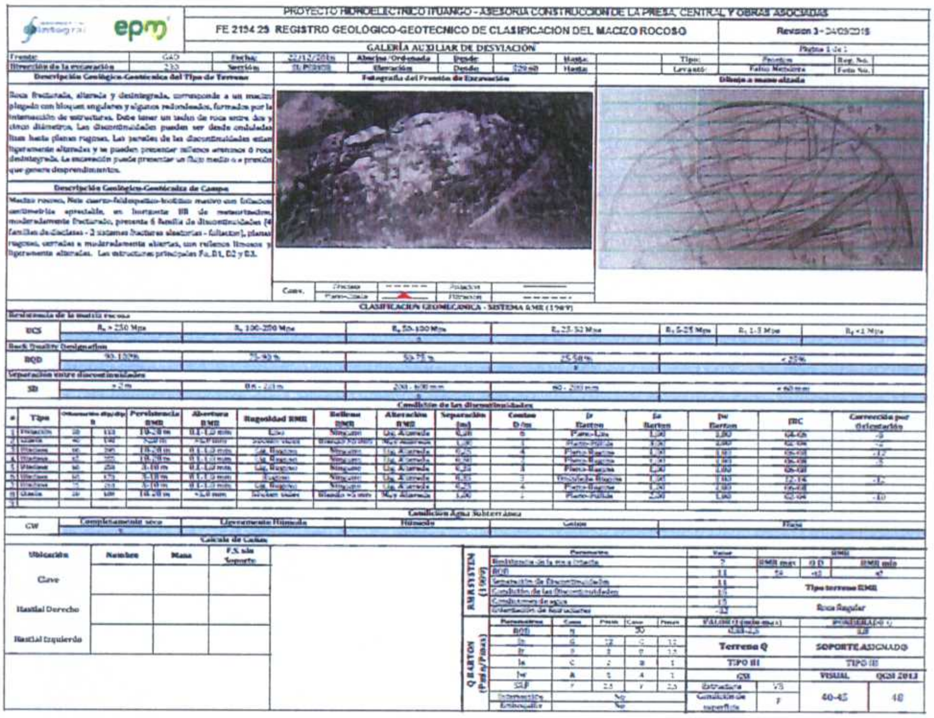

Figure 22. Face map record from the Asesoria on ADT at k 0+529.60

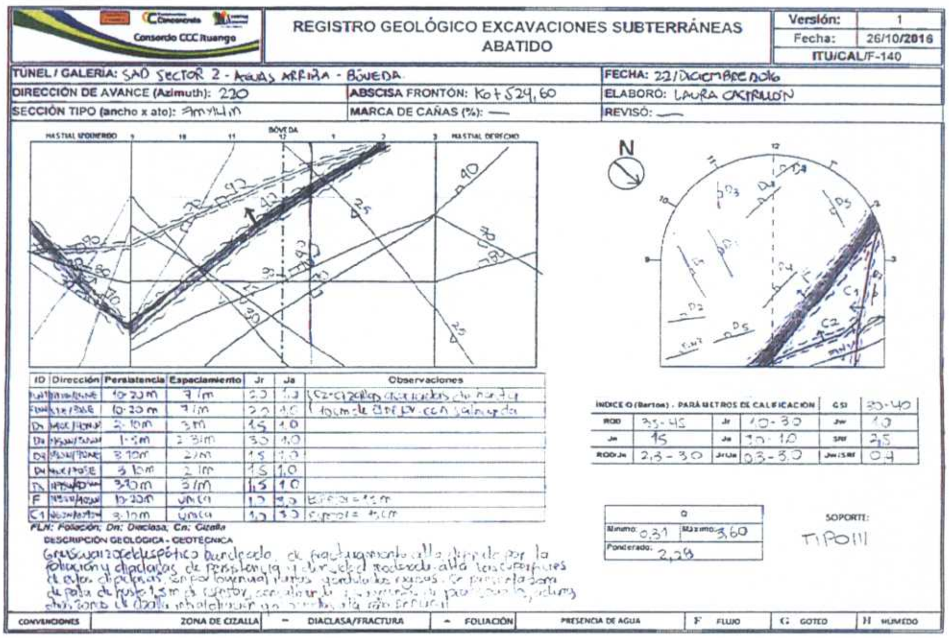

Figure 23. Face map record from the Contractor on ADT at k 0+529.60

8.6. The records were compiled as an as-built record by the Asesoria that included the geological and geotechnical records, the support measures installed, drainage, instrumentation and monitoring, for the whole of Sector 2 by interpolation between the face maps (Figure 24). These records are in Appendix D.

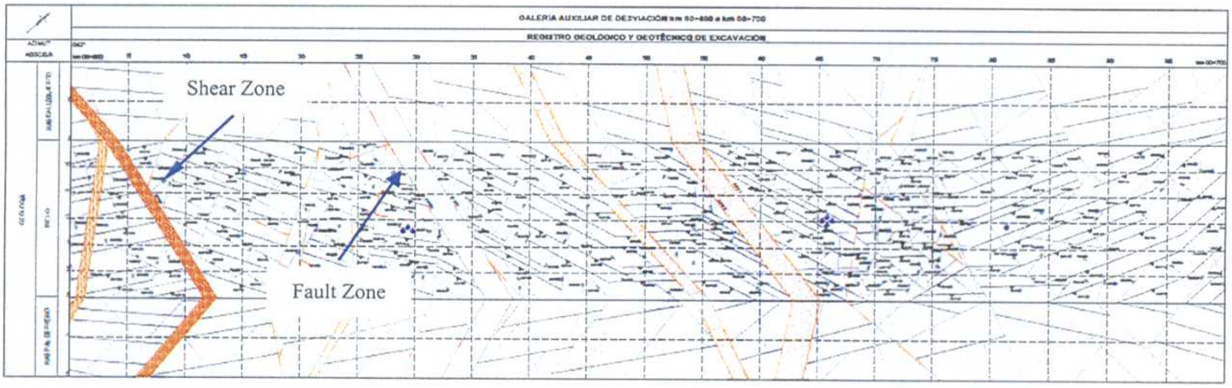

Figure 24. "As-built" Geological record between k 0+600 and k 0+700[140]

8.7. According to the final construction report by the Interventoria[141], Sector 2 of the ADT was excavated in terrain type II and type III in two stages, an 8 m high top heading and a 6 m high bench. No terrain type I or IV was mapped but parts of type IV support were installed in the ADT intake portal.

8.8. The occurrence of ground types encountered in the ADT Sector 2 was close to the predictions except there was no terrain type I and IV [142] (Table 12).

Ground type	Estimated [%]	Actual [%]	Length [m]
I	5	0	0
II	60	63	460.69
III	30	31	228.10
IV	5	0	0
Portal		6	42.73

Table 12. Actual versus estimated ground type for the ADT Sector 2

[140] Integral (2018) Excavaciones Mapeo Geológico D-PHI-034-TUN-MG-B-001_014A.pdf
[141] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, Section 5.6.7
[142] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, Table 7.19

8.9. Nine shear zones and faults were encountered during the excavation of Sector 2 of the ADT that the Experts estimate to be about 26% of this length (Table 13).

From	To	Terrain type	Number, thickness of shear zones
0+443	0+483	III	One, 15 m
0+498	0+676	III	Three, 18 m, 16 m, 41 m
0+698	0+733	III	One, 30 m
0+733	0+898	II	One, 27 m
0+902	0+905	III	One, 9 m
0+905	1+154	II	Two, 15 m, 14 m
Total			**185 m (26%)**

Table 13. Location and estimated length of shears and faults mapped in the ADT Sector 2

Advance Lengths and Rates

8.10. The advance rates for the terrain types for the top heading through Sector 2 of the ADT are in Table 14[143]. The average advance rates were close to the blast design in the Contractors methods statement[144]. However, the maximum advance lengths exceeded the design and the minimum advance lengths were unusually short.

Terrain Type	Blast design drilled length [m][144]	Advance length [m]		
		Minimum	Maximum	Average
II	3.0	1.8	4.2	3.0
III	2.5	1.5	4.0	2.8

Table 14. Advance lengths for the top heading of the ADT Sector 2

8.11. The tunnel Inspector's daily reports were reviewed, and a table prepared of the tunneling activities taking place in Sector 2 of the ADT in Type III ground between December 14 and 23, 2016 along with a description of the Inspector's comments recorded on these sheets and our observations of the significance of each comment for the ADT. In summary, these records confirm that there were issues with the construction cycle with delays, incomplete support installation and at k 0+543.2 lack of approval by the Inspector for a tunnel advance "due to geological conditions at the excavation front"[145]. The actual records are presented in Appendix D.

143 Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, Table 6.14, Table 6.15
144 CCCI (2016) Diagramas de Voladura ESQ-GAD-EXC-188 R1 - GAD - BOVEDA TIPO III.
145 *Sedic-Ingetec (2016) Daily inspector report* of December 16, 2016

Date	Time	Activity	Abscissa	Inspector's Comment	Observation
14th Dec. '16	06:15	**Blast No.102**: 3.2m	0+546.5 - 0+543.3	ZH 10-12, 2 No. Perimeter holes not charged. Over-excavation noted ZH 11-2	Even though the perimeter holes were not charged and 'spiles' had been installed an overbreak of circa 3m was recorded by the Inspector
	16:30	Finish Scaling		Delayed during testing of impermeable resin	This means that 10 hours has elapsed between the blast and the completion of scaling preventing the early installation of support in this unstable area
	21:15 – 22:00	Apply Primary Shotcrete	0+546.5 – 0+543.2	ZH 9 - 3	This means that there was a further delay of an additional 5 hours before any support is installed. *
15th Dec. '16	00:00 – 07:00	Drill & Install BRL8x6m bolts	0+547.58 – 0+546.26	ZH 9 - 3	6m long BRL rock bolts installed without grout, only max. 0.85m (3 cartridges) end anchor resin grout.
	00:00 – 07:00	Drill & Install BAR8 x 4.5m 'spiling' bolts	0+543.2	ZH 10.5 – 2.5	Only 4.5m long BAR rock bolts installed as 'spiles' providing only 1.6m of cover following blast of 2.9m (No.103).
	07:00 – 19:00	Complete installation and start injection of 39 No. BRL8x6m bolts	0+547.58 – 0+543.68	ZH 9 - 3	Grouting of BRL rock bolts not completed 19 hours after start of installation.
	20:30 – 00:00	Injection of 39 No. BRL8x6m bolts	0+547.58 – 0+543.68	ZH 9 -3	Grouting of BRL rock bolts only completed 24 hours after start of installation.
16th Dec. '16	01:00 – 02:00	Installation of 6 No. BAR8 x 6m localized	0+544.69 – 0+547.38		Geological conditions require the installation of localized, additional support measures.
	02:00 – 03:00	Installation of plates			Not clear which plates are being referenced but end anchor bolts require plates to function.
	16:45 – 17:45	Complete thickness Primary Shotcrete	0+546.5 – 0+543.2	ZH 9 – 3	Delay in application of complete primary shotcrete support until over 58 hours following blast. *
	16:45 – 17:45	Apply Secondary Shotcrete	0+548.86 – 0+544.78	ZH 9 - 3	Second layer of shotcrete being brought to within 1.5m of face 5 hours before blasting.
	21:40	**Blast No. 103**: 2.9m	0+543.2 – 0+540.3	ZH 9 – 12, 3 No. perimeter holes not charged. **Blast not approved due to geological conditions at the face of the excavation.**	Blast proceeds despite Inspector's warning not to advance due to geological conditions at the face.
17th Dec .'16	02:00 – 05:00	Scaling			3 hours of Scaling completed c. 7 hours following blast.
	05:30 – 08:30	Inactive, waiting for shotcrete			Further delay in completing application of first support measure until c. 11 hours following blast.
	08:30 – 09:00	Apply Primary Shotcrete	0+543.2 – 0+540.3	ZH 9 -3. **Insufficient thickness**, 10m3 approved, 7m3 applied	Shotcrete applied up to c.11 hours following blast insufficient to meet minimum requirements. *
	10:30 – 18:30	Drill, Install & Inject 26 No. BRL8x6m bolts	0+544.98 – 0+541.08	ZH 9 - 3	Within c. 21 hours following blast BRL rock bolts completely installed. * No BAR8 'spiling' type bolts installed prior to Blast No. 104.
	10:30 – 18:30	Install 2 No. BAR8 x 6m, localized	0+542.38 & 0+541.09		Geological conditions require the installation of localized, additional support measures.
	22:30	**Blast No. 104**: 2.8m	0+540.3 – 0+537.5	3 No. sets of perimeter holes not charged. Contractor advised not to charge the outer sets of holes between ZH 9 – 12 **due to unfavorable ground**	Due to unfavorable ground conditions the Inspector advises on reduction of charge in the blast holes to help prevent damage to rock mass.
18th Dec. '16	04:00 -05:30	Scaling			Scaling completed 7 hours following blasting.
	06:30 – 9:30	Inactive, waiting for shotcrete			Further delay in application of first support measures. *
	09:30 – 10:15	Additional Scaling		Requested by Inspector	Initial scaling insufficient requiring additional work and resulting in further delay in application of first support measures. *
	11:00 - 11:30	Apply Primary Shotcrete	0+540.3 – 0+537.5	ZH 9 - 3	Application of first support measure completed 13 hours following blast. *
	14:45 – 15:15	Apply Secondary Shotcrete, (Partial)	0+544.98 – 0+539.78	**Partial**, only where bolts installed and injected	Application of incomplete secondary shotcrete layer c. 17 hours following blast. *
	17:30 – 20:00	Drilling for bolts			Start of rock bolt installation process 19 hours following blast. *
19th Dec. '16	20:30 – 05:00	Installation of BAR8 x 4.5m 'spiling' bolts	0+537.5	ZH 10.5 – 2.5	Only 4.5m long BAR rock bolts installed as 'spiles' providing only 0.5m of cover following blast of 4.0m (No.105).
	20:30 – 05:00	Installation & Injection of 25 No. BRL8x6m bolts	0+543.68 – 0+539.68	ZH 9 - 3	BRL rock bolts completely installed only 3 hours before 4.0m advance.
	08:00	**Blast No. 105**: 4.0m	0+537.5 – 0+533.5	3 No. perimeter holes not charged between ZH 8 - 12	Some perimeter holes not charged to help avoid damage to rock mass by blasting.
	13:30 – 14:30	Scaling			Scaling completed 6.5 hours following blast.
	15:30 – 20:00	Inactive, waiting for shotcrete			Delay in application of first rock support measure up to 12 hours following 4.0m advance.
	21:00 – 22:00	Apply Primary Shotcrete	0+537.5 – 0+533.5	ZH 9 - 3	Application of first rock support measure completed 14 hours following 4.0m advance.
20th Dec. '16	23:30 – 00:30	Apply secondary shotcrete	0+539.98 – 0+537.5	ZH 9 - 3	Application of secondary rock support measure to within 4.0m from face completed 16.5 hours following 4.0m advance.
	03:00 – 07:00	Drilling for bolts			Start of rock bolt installation process 19 hours following blast. *
	07:00 – 17:30	Install 30 No. BRL8 x 6m bolts	0+539.78 – 0+537.18	ZH 9 - 3	Installation of BRL8 rock bolts with max. 0.85 resin end anchor to a distance of c. 4m from the face completed 33.5 hours following 4.0m advance. * BRL bolts not grouted before Blast No. 106.
	07:00 – 17:30	Install 15 No. BAR8 x 4.5m 'spiling' bolts	0+533.5	ZH 10.5 – 2.5	4.5m length BAR8 bolts installed for 1.6m advance providing 2.9m 'spile' cover after Blast No 106.
	20:00	**Blast No. 106**: 1.6m	0+533.5 – 0+531.9	2 rows of peripheral holes not charged between ZH 9 - 1	Some perimeter holes not charged to help avoid damage to rock mass by blasting.
21st Dec. '16	01:00- 02:00	Scaling			Scaling completed 6 hours following blast.
	03:30 – 07:00	Inactive, waiting for shotcrete			Delay in application of first rock support measure up to 11 hours following advance.

Date	Time	Activity	Abscissa	Inspector's Comment	Observation
	07:00 – 07:30	Apply Primary Shotcrete	0+533.5 – 0+531.9	ZH 9 - 3	Application of first support measure completed 11.5 hours following advance.
	07:00 – 07:30	Apply Secondary Shotcrete	0+537.5 – 0+538.48	ZH 9 - 3	Application of secondary shotcrete support to within c. 6m of face completed 11.5 hours following advance.
	11:30 – 19:00	Install 15 No. BRL8 x 6m bolts	0+537.18 – 0+535.88	ZH 9 - 3	Start of rock bolt installation process 15.5 hours following advance.
	11:30 – 19:00	Install 20 No. BAR8 x 4.5m 'spiling' bolts	0+531.9	ZH 10.5 – 2.5	4.5m length BAR8 bolts installed for 1.6m advance providing 1.5m 'spile' cover after Blast No 107.
	21:00 – 23:00	Grout 14 No. BRL8 x 6m bolts	0+539.18 – 0+535.88	ZH 9 - 3	Completion of BRL8 rock bolt installation 27 hours following advance.
22nd Dec. '16	06:45	**Blast No. 107**: 3.0m	0+531.9 – 0+528.9	3 rows of perimeter holes not charged between ZH 9 - 12	Some perimeter holes not charged to help avoid damage to rock mass by blasting.
	11:00 -12:30	Scaling			Scaling completed 6 hours following blast.
	13:30 – 15:30	Inactive, waiting for shotcrete			Delay in application of first rock support measure up to c. 9 hours following 3.0m advance.
	15:30 – 17:00	Apply primary shotcrete	0+531.9 - 0+529.6	ZH 9 - 3	Application of initial support measure completed c. 10 hours following 3.0m advance.
	15:30 – 17:00	Apply Secondary Shotcrete	0+541.08 – 0+538.48	ZH 9 - 3	Application of secondary shotcrete support to within c. 10m of face completed c.10 hours following advance.
23rd Dec. '16	03:00 – 06:30	Drilling for rock bolts		Inspector notes displacements continues on right side	Start of rock bolt installation process c.20 hours following advance.
	08:45 – 17:30	Installation and injection of 49 No. BRL 8 x 6m bolts	0+537.18 – 0+531.98	ZH 9 -3	Completion of some BRL 8 rock bolt installation c.36 hours following advance.
	08:45 – 17:30	Installation of 20 No. BAR 8 'Spiling' bolts	0+529.6	ZH 10.5 – 2.5	4.5m length BAR8 bolts installed for 3.0m advance providing 1.5m of cover after Blast No. 108
	20:00 – 21:50	Installation of missing plates for BRL8 bolts	0+533.28, 0+535.88, 0+531.98		Completion of BRL8 rock bolt installation c. 39 hours after advance.
	22:05	**Blast No. 108**: 3.0m	0+529.6 – 0+526.6	2 rows of periphery holes not charged between ZH 10 and 1	Some perimeter holes not charged to help avoid damage to rock mass by blasting.
24th Dec. '16	03:00 – 04:30	Scaling			Scaling completed c. 6 hours following blast
	07:00 – 11:45	Inactive, waiting for shotcrete			Delay in application of first support measure up to c. 13.5 hours following 3.0m advance.
	11:45 – 12:15	Apply primary shotcrete	0+526.6 – 0+529.6	ZH 9 -3	Application of initial support measure completed c.14 hours following 3.0m advance.
	12:30 – 15:00	Inactive, waiting for secondary shotcrete		Not dispatched from the plant due to priority of other fronts	Delay in application of secondary support due to priority of other fronts. Secondary support not applied prior to next 3.6m advance.
	20:00 – 06:30	Drilling and partial installation of bolts, systematic and 'pre-support'	0+530.68, 0+529.38 & 0+526.6	-6.92 to -6.92, -7.07 to 7.07 & ZH 10.5 – 2.5	Start of rock bolt installation process c. 22 hours after advance.
25th Dec. '16	07:00 – 11:15	Injection of 29 No. BRL 8 bolts	0+530.68 – 0+529.38		Completion of rock bolt installation c. 37 hours after 3.0m advance and 2 hours prior to next 3.6m advance.
	13:15	**Blast No. 109**: 3.6m	0+526.6 – 0+523.0	2 rows of periphery holes not charged between ZH 10 and 1 due to geological conditions	Some perimeter holes not charged to help avoid damage to rock mass by blasting. Secondary shotcrete support now over 15m from face.

Table 15. Summary of the tunnel Inspector's daily reports in Sector 2 of the ADT between December 14 and 23, 2016

8.12. The * in the Observation column in Table 15 refers to two figures from the Asesoria that show that without support most of the wedges they analyzed fail and even with bolts only installed without shotcrete many of these wedges fail.[146] (¶7.35, ¶7.36)

Installation of Rockbolts

8.13. The TS define the use of two types of passive rock bolts[147] BAL (*Barra Anclada con Lechada)* which is an untensioned (passive) anchor bar filled with cement grout (Right on Figure 25) and BAR (*Barra Anclada con Resina)* which is an anchor bar filled with resin (Left on Figure 25). The number following the designation BAL or BAR refers to

[146] Integral (2019) Diagnostica Geológico Geotécnico Contingencia, I-I-2194-062018-01-R2 Figures 7.16 and 7.17
[147] EPM (2012) Especificaciones Técnicas de Construcción, Technical Specifications, Section 3.5.2, Pages 128 and 129

the diameter of the steel bar in the rock bolt, e.g. *BAL 8* refers to an anchor bar with a diameter of 8/8 of an inch (1 inch or 25.4 mm diameter).

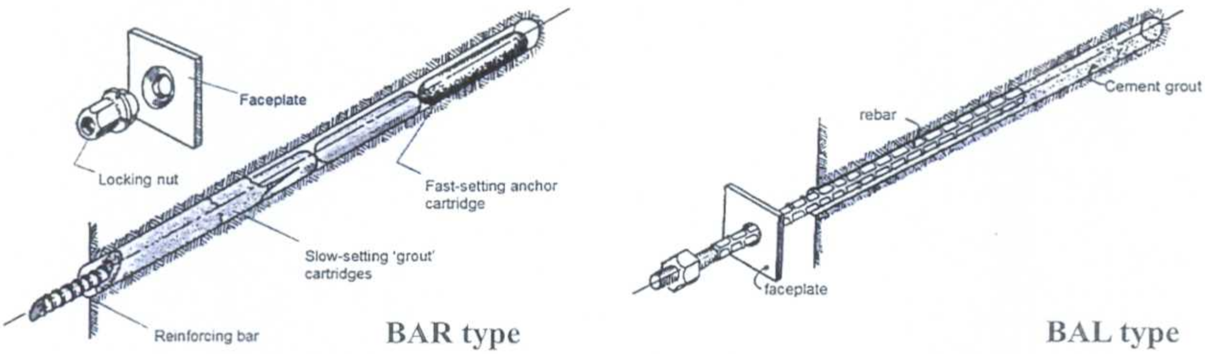

Figure 25. Comparison between BAR and BAL type rockbolts

8.14. The BAL rockbolts do not contribute to the support of the ground until after the cement grout has hardened, which can be as much as 24 hours, unlike the BAR rockbolts, where the resin hardens in minutes.

8.15. The BRL type rock bolts (*Barra Anclada con Resina y Lechada*) which is an untensioned bar with resin grout to anchor the far end of the bar and the rest of the annulus is filled with cement grout as for the BAL. This was a request from the Contractor to Supervision to excavate the tunnel faster and reduce the time required for the installation of support[148] in the discharge tunnels. This rockbolt is a combination of BAL and BAR rockbolts, where the steel bar is installed with resin anchoring nominally 1.2 m of the bar deep into the drilled hole, and the rest of the rockbolt is grouted with cement at a later stage. The deeply embedded resin of the BRL rockbolts sets within minutes, providing an almost instantaneous anchor able to pass the pull-out test, but without the beneficial friction along the steel bar, which is required in highly fractured ground. The cement grout portion is completed later in the excavation cycle and with the added delay of the hardening of the grout allows over-relaxation of the rock mass.

[148] CCCI (2015) Letter CI-04079

8.16. The Asesoria response was a request for pull-out tests and a resin grout length of 1.2 m[149]. After various communications between the parties[150],[151],[152], the Asesoria approved the BRL rockbolts for use in the discharge tunnels[153], and Supervision communicated this approval to the Contractor requiring the grouted length of 1.2 m of resin, and that the steel plate cannot be removed for the grouting of the BRL[154].

8.17. The Contractor stated that BRL type rockbolts were considered as a plan to meet the deadline for delivery of the ADT, but that this bolt was not included in the design drawings by the Asesoria because they still require BAL type rockbolts, so CCCI requested approval to use BRL rockbolts in the ADT and updated drawings[155]. The Asesoria authorized the use of BRL rockbolts in the ADT[156], and the Supervision notified the Contractor of the authorization[157]. Finally, the Asesoria expanded the authorization to use BRL rock bolts in all underground works of the ADT[158], and issued updated drawings including BRL rockbolts[159] to EPM whilst Supervision instructed the Contractor to use the updated drawings[160].

8.18. This is a fundamental change in how the ground is controlled in the field by a rockbolt because the 1.2 m resin grout classifies this as an "End anchored" rock bolt until the cement portion has set. In sound rock, end anchored bolts work completely different, as they will transfer the load acting on the tunnel surface via the steel plate to deep into the ground without any crumbling of the ground in between due to the characteristics of the rock mass. But end anchored bolts allow movement of the rock mass between the grout and the wall of the tunnel. Thus, their efficiency is reduced and the shotcrete thickness

149 Integral (2015) Letter D-PHI-COP-546-2015
150 Ingetec-Sedic (2015) Letter INT-OC-CCCI-781-15
151 Integral (2015) Letter D-PHI-COP-600-2015
152 Ingetec-Sedic (2015) Letter INT-OC-EPM-357-15
153 Integral (2015) Letter D-PHI-COP-701-2015
154 Ingetec-Sedic (2015) Letter INT-OC-CCCI-1127-15
155 CCCI (2015) Letter CI-04880
156 Integral (2015) Letter D-PHI-COP-903-2015
157 Ingetec-Sedic (2015) Letter INT-OC-CCCI-1433-15
158 Integral (2015) Nota de Campo NC-567-15
159 Integral (2016) Letter D-PHI-CCE-ADM-1-C2766
160 Ingetec-Sedic (2016) Letter INT-OC-CCCI-100-16

must be defined accordingly. Its applicability is reduced greatly in highly fractured or sheared ground, because the support relies heavily on the shotcrete as structural support.

8.19. Fully grouted bolts on the other hand transfer load by friction along the bolts starting at the steel plate location, helping control movements in sheared ground starting at the tunnel surface.

8.20. There are two fundamental issues with this type of rockbolt which makes them inappropriate for fractured and weak rock.

i) The number of resin cartridges used for the end anchor was insufficient to achieve a 1.2 m grouted length. In reality it would be significantly less because of the size of the annulus and the number of cartridges used could not provide this amount of grout and a length of 0.75 m was more likely.

ii) The cement grouting stage of the BRL rockbolts was completed at different times after installation and before the face advance (Table 16). The cement grout would not have hardened fully in the time before advance of the face, consequently the resin end anchor and the face plate would have been the only reinforcement of the rock mass until the grout was fully set. Therefore, the rock mass would relax due to the advance of the face before the full support was active.

8.21. It is notable that even though the grouting was not going to cure completely before the face was advanced, the rock bolts installed between about 0+548 and 0+536 where the tunnel failure first manifested itself were grouted much sooner, than in much of Sector 2 of the ADT, and all bolts were grouted before the face was advanced, unlike the remainder of the tunnel where the face was advanced with only the resin end anchor. This indicates heightened concern about the ground from one or other party, but we have not been provided with records to confirm what concerns or which party.

From	To	Time to cement grouting after installation	Time of cement grouting prior to advance
0+547.58	0+543.68	24 hours	22 hours
0+544.98	0+541.08	0 hours	4 hours

From	To	Time to cement grouting after installation	Time of cement grouting prior to advance
0+543.68	0+539.68	0 hours	3 hours
0+537.18	0+535.88	4 hours	4 hours

Table 16. Time of cement grouting of BRL rockbolts at the event

Installation of Convergence Pins

8.22. The TS state that the convergence pins are to be installed at a distance no greater than 5 advances of the face of the tunnel. Table 17 [161,162,163,164] compiles the following dates regarding the ADT:

i) Excavation of the top head of the tunnel

ii) Installation of the convergence pins

iii) Baseline reading of the convergence pins

iv) First reading for movement of the ground using convergence stations

8.23. The table also includes the associated delay to install, take the baseline reading, and take the first monitoring reading. Finally, the time taken to install the convergence pins is then compared to the average time for 5 face advances for the ADT, as required by the TS.

[161] Consorcio Ingetec-Sedic (Undated) Sistema Auxiliar de Desviación (SAD) Instrumentación Geotécnica Instalada. Summary of installation of convergence pins
[162] Integral (2018) Seguimiento de Convergencias. Spreadsheets GAD-##-km#+### containing convergence readings
[163] Integral (2017-2018) Registro Geológico-Geotécnico de Clasificación del Macizo Rocoso. Face mapping records
[164] Consorcio Ingetec-Sedic (Daily) Informe Diario de excavación y soporte. Daily Supervisor Reports of Excavation and Support

Table 17. Summary of time of installation and reading of convergence stations in the ADT

Station	Date of Excavation	Top Heading			Time delay [days]			Approximate # of cycles to install (2.01 days/cycle)	Meets TS of 5 cycles	Comment
	Top Head	Installation	Baseline	1st. Reading	To install	To read the baseline	To 1st Reading			
440*	July 2, 2017	August 12, 2017	July 16, 2017	July 18, 2017	41	-27	2	20.4	No	Report baseline before installation
445*	June 24, 2017	August 12, 2017	July 16, 2017	July 18, 2017	49	-27	2	24.4	No	Report baseline before installation
450	April 14, 2017	June 28, 2017	July 16, 2017	July 18, 2017	75	18	2	37.3	No	
455	April 8, 2017	April 18, 2017	July 16, 2017	July 18, 2017	10	89	2	5.0	Yes	
460	March 30, 2017	April 18, 2017	July 16, 2017	July 18, 2017	19	89	2	9.5	No	
465	March 25, 2017	April 18, 2017	July 16, 2017	July 18, 2017	24	89	2	11.9	No	
477	January 24, 2017	February 18, 2017	July 27, 2017	August 15, 2017	25	159	19	12.4	No	
492	January 18, 2017	March 3, 2017	April 5, 2017	NA	44	33		21.9	No	
508	January 8, 2017	January 28, 2017	April 5, 2017	July 23, 2017	20	67	109	10.0	No	
523	December 25, 2016	April 3, 2017	April 5, 2017	July 23, 2017	99	2	109	49.3	No	Report bench instrumented before top head
537	December 19, 2016	January 4, 2017	January 17, 2017	April 5, 2017	16	13	78	8.0	No	
553	December 9, 2016	January 4, 2017	January 17, 2017	NA	26	13		12.9	No	
567	December 1, 2016	January 4, 2017	January 17, 2017	NA	34	13		16.9	No	
582	November 23, 2016	January 4, 2017	January 17, 2017	NA	42	13		20.9	No	
597	November 15, 2016	January 4, 2017	January 17, 2017	NA	50	13		24.9	No	
612	November 8, 2016	November 20, 2016	November 20, 2016	November 27, 2016	12	0	7	6.0	No	
627	October 27, 2016	November 20, 2016	November 20, 2016	November 27, 2016	24	0	7	11.9	No	
657	October 7, 2016	November 20, 2016	October 18, 2016	NA	44	-33		21.9	No	Report baseline before installation
672	September 30, 2016	October 9, 2016	October 18, 2016	November 3, 2016	9	9	16	4.5	Yes	
687	September 26, 2016	October 9, 2016	October 18, 2016	November 3, 2016	13	9	16	6.5	No	
701	September 22, 2016	October 9, 2016	October 18, 2016	November 3, 2016	17	9	16	8.5	No	
711	September 18, 2016	October 9, 2016	October 18, 2016	November 3, 2016	21	9	16	10.4	No	
741	September 3, 2016	September 10, 2016	November 3, 2016	NA	7	54		3.5	Yes	
776	August 22, 2016	August 31, 2016	October 5, 2016	October 18, 2016	9	35	13	4.5	Yes	
791	August 16, 2016	July 31, 2016	August 31, 2016	October 5, 2016	-16	31	35	-8.0	Yes	Report installation before excavation
814	August 8, 2016	July 31, 2016	August 31, 2016	October 5, 2016	-8	31	35	-4.0	Yes	Report installation before excavation
839	July 24, 2016	July 31, 2016	August 31, 2016	October 5, 2016	7	31	35	3.5	Yes	
865	July 14, 2016	July 31, 2016	August 3, 2016	August 31, 2016	17	3	28	8.5	No	
889**	June 24, 2016	June 16, 2016	August 3, 2016	August 31, 2016	-8	48	28	-4.0	Yes	Report installation before excavation
904	June 24, 2016	July 16, 2016	August 11, 2016	August 31, 2016	22	26	20	10.9	No	
928	July 9, 2016	July 16, 2016	August 11, 2016	August 31, 2016	7	26	20	3.5	Yes	
955	July 20, 2016	August 3, 2016	September 3, 2016	September 23, 2016	14	31	20	7.0	No	
970	July 26, 2016	August 18, 2016	February 14, 2017	NA	23	180		11.4	No	
980	August 4, 2016	August 26, 2016	September 3, 2016	September 23, 2016	22	8	20	10.9	No	
1004	August 14, 2016	August 26, 2016	September 3, 2016	September 23, 2016	12	8	20	6.0	No	
1029	August 23, 2016	August 31, 2016	September 3, 2016	September 23, 2016	8	3	20	4.0	Yes	
1055**	September 4, 2016	October 9, 2016	NA	NA	35	NA		17.4	No	
1079**	September 10, 2016	October 9, 2016	September 23, 2016	October 18, 2016	29	-16	25	14.4	No	Report baseline before installation
1104**	September 21, 2016	October 9, 2016	October 18, 2016	NA	18	9		9.0	No	
				Average	23	28	25	12	100%	

8.24. The records show the following:

i) 74% of the convergence stations do not meet the TS and were installed after more than 5 advances of the face of the tunnel.

ii) The convergence pins were installed on average 23 days after the excavation of the face. Which, based on the performance of the contractor in the ADT, should have been done within 2.01day/cycle x 5 cycles= 10.05 days.

iii) In addition to the delay to install the convergence pins, it took 28 days on average to take the baseline reading after the pins were installed.

iv) Furthermore, it took an additional 25 days on average to take the first reading to be compared with the baseline, which is the first actual monitoring of the convergences

8.25. The first monitoring of convergence values was carried out on average, 76 days after the excavation had taken place. This is important because the convergence was not measured in time and therefore, the design could not be adjusted for the encountered ground conditions as required by the OM.

Comparison of Designed and the As Built ADT Sector 2

8.26. The as built ADT Sector 2 was generally in accordance with the design for construction[165,166,167] except for the following issues:

i) D-188 mesh between the shotcrete layers for terrain type III and IV in a shear zone, fault or zone of highly fractured rock was not installed in all such areas.

ii) Instrumentation to monitor for convergence was not installed for more than five advance lengths, or about 20 m of the tunnel face, whereas in the original diversion tunnel system, the standard applied was that these should be installed within 4.0m of the face[168].

iii) The required support was not installed before each advance.

iv) Fault material was not removed and replaced by support elements[169].

8.27. The as-built record for Sector 2 of the ADT (PHI-IFF-LC1-011-R0) shows the same level of support as the final design drawing (D-PHI-034-TUN-EX-C-010, R1) except for the inclusion in the as-built record of 'mesh as required' in the walls for Type III ground.

Repairs and Defective Work

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165 Integral (2018) Compilation of Design Calculations I-M-2194-034-GYG-01-R0
166 Integral (2016) Tunel Excavaciones Secciones Tipicas D-PHI-034-TUN-EX-C-010-R1. Note 8.
167 Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0
168 Integral (2012) Letter Report, D-PHI-COP-058-2012, Page 8 of 8.
169 EPM (2012) Especificaciones Técnicas de Construcción, Section 3.2.2

8.28. Defective work is all work not meeting the technical specifications, the design, the standard of performance of the contractor or standard of care of the Asesoria and Supervisor of the project. Defective work was notified in three ways, as follows:

i) Notifications from the Contractor to the Supervisor regarding the works not performing as designed, such as fall outs of rock and support requiring repairs.

ii) Instructions from Supervisor to Contractor to perform repairs or additional work not included in the design, e.g. reinstall rock support, install support not included in design such as spiles, where the designed support is not able to support the ground.

iii) Instructions from Supervisor to Contractor to adhere to construction method statements, technical specifications and engineering practice, such as correct and timely installation of support because inopportune installation allows excessive relaxation of the rock mass reducing the safety of the design, or correct installation of instrumentation because inopportune installation prevents verification of the design.

8.29. The following summarizes the records of defective work:

i)	(35 records)	Support installed not stabilizing the section
ii)	(9 records)	Excavation requires work not included in design
iii)	(63 records)	Installation of support does not meet Contractor Method Statement and allows unsafe rock relaxation
iv)	(3 records)	Excavation beyond the specified limits of excavation negatively affecting the design
v)	(1 record)	Concrete does not meet Technical Specifications
vi)	(1 record)	Late installation of convergence stations prevents verification of design
vii)	(1 record)	Blasting does not meet the Technical Specifications

viii) (1 record) Repairs do not meet the Technical Specifications

8.30. The Asesoria claims that the Contractor caused excessive damage to the rock mass by poor blasting. The Contractor claims the blasting complied with the TS, but the damage was due to the poor ground conditions. We do not offer an opinion on this, but we recognize that the central issue is that the rock mass suffered damage and overbreak that exceeded expectations of the Asesoria and the Contractor.

Commission and Operation

8.31. The Asesoria issued the Deviation Manual Rev. 0 (Manual de Desviación del Rio) on July 18, 2017, with subsequent revisions Rev. 1 on July 27, 2017 and Rev. 2 on August 28, 2017, which was about four weeks before the Cauca River was diverted through the ADT. The report included a checklist of activities to be completed prior to deviation.

8.32. The project created the ADT Commissioning Committee (*Comité de Puesta en Operación del SAD*) to review a checklist of activities from the Deviation Manual[170] at weekly meetings[171]. One activity was to clean the tunnel floor to sound rock in Sector 2 of the ADT that did not have a concrete invert. However, this item was no longer included in the checklist after August 30, 2017 after about 15% of the task was completed. The location of the floor that was cleaned to sound rock was not provided. It is our understanding, based on our discussions with the Asesoria, that their intention was to inspect the floor and instruct a concrete floor where required on completion of this task, but this never happened.

8.33. There appears to be another checklist from the Deviation Manual for the ADT which had a different cleaning requirement which was to the standards for ANLA which we understand was for removing detritus. This checklist showed most of the activities were completed for commissioning of the ADT and it was signed off by the Supervisor and the Contractor on September 10, 2017[172], amended with signature only by the Contractor stating completion of pending activities on September 11, 2017 and September 12, 2017.

[170] Integral (2017) Manual de Desviacion del Rio I-I-2194-03-0006 Rev 2
[171] Ingetec-Sedic (2017) Acta de comité de puesta en operación del SAD No. 001
[172] CCCI (2017) Letter CI-09228

The finalized checklist was delivered to Supervision on September 13, 2017[173]. From this we conclude that the only cleaned to floor to sound rock for 15% of the tunnel and the rest was only cleaned of detritus to comply with ANLA.

8.34. On September 12, 2017 the Cauca River flooded during removal of the cofferdam in front of the ADT portal, stopping the works[174].

8.35. The river level records on the days prior to deviating the river through the ADT are shown in Figure 26. The rise in the river level stopped the works until September 15, 2017, which means the crest of the coffer dam was between Elev. 226 and Elev. 228 masl.

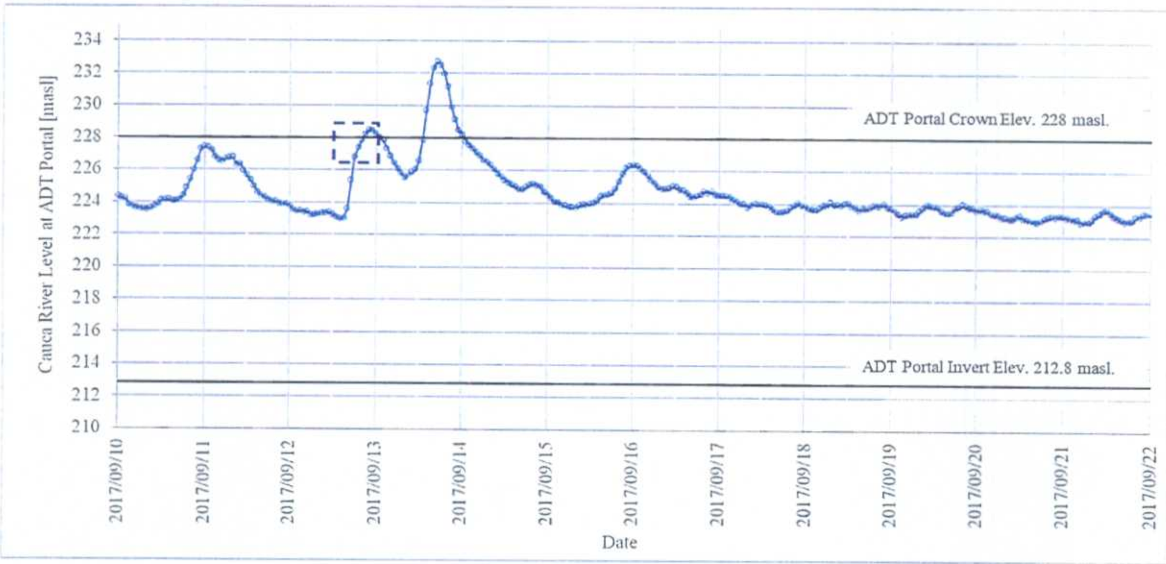


Figure 26. Cauca River Level at ADT Upstream Portal September 10, 2017 to September 22, 2017[175]

8.36. The Cauca River was diverted through the ADT on September 22, 2017 by lowering the crest of the coffer dam to Elev. 215 masl[176]. At this point the Cauca River was flowing through the LDT, RDT and ADT.

Operation of the ADT

[173] Integral (2017) Letter D-PHI-CCE-ADM-1-C3970
[174] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0.
[175] Integral (2018) Spreadsheet Desviaciones Cauca Velocidades
[176] Ingetec-Sedic (2017) Informe mensual detallado de septiembre 2017 PHI-INM-LC1-073-R0

8.37. The LDT and RDT were progressively closed during operation of the ADT (Table 18). The closure of each tunnel started with the construction of cofferdams to restrict the flow through each of the two intakes to each tunnel, followed by the construction of temporary concrete plugs on each intake, and a reinforced concrete plug in the main LDT[177]. Tapón No. 5 in Gallery No. 1 is included because this construction gallery is connected to the reservoir area and to the main section of the RDT.

Structure	**Start**	**Finish**
LDT - Coffer dams	2017.09.22	2017.10.06
Temp. concrete plug LDT branch 1, Tapón 16	2017.10.06	2017.10.12
Temp. concrete plug LDT branch 2, Tapón 17	2017.10.12	2017.10.20
Final Reinforced concrete plug LDT, Tapón 9	2017.11.11	2018.01.18
RDT - Coffer dams	2018.01.23	2018.02.14
Temp. concrete plug RDT branch 3, Tapón 49	2018.02.03	2018.02.28
Temp. concrete plug RDT branch 4, Tapón 50	2018.02.15	2018.03.18
Temp. concrete plug Gallery No. 1, Tapón 5[178]		2018.04.01

Table 18. Closure program for the LDT and RDT

8.38. The Asesoria requested bathymetry of the river channel and systematic river level measurements on October 4, 2017, to verify the status of the demolished cofferdams in front of the ADT portal, verify the calibration curve of the RDT and the ADT, and to validate the normal operation of the ADT[179]. This was the basis for flow data through the diversion system. It is our understanding that this bathymetric survey was not conducted.

8.39. The Asesoria notified EPM on November 28, 2017 that it was not viable to install a barrier for floating debris upstream of the ADT[180].

8.40. A concrete barrier to within about 2 m of the tunnel crown (Tapón 15A) was built in September 2017 near the intersection of Construction Gallery No. 4 and the ADT[181,182].

[177] Ingetec-Sedic (2019) Proceso de cierre de los túneles de desviación Izquierdo y derecho
[178] Ingetec-Sedic (2018) Informe mensual detallado de mayo 2018 PHI-INM-LC1-081-R0, Page 183
[179] Integral (2017) Letter D-PHI-CCE-ADM-1-C3889
[180] Integral (2017) Letter D-PHI-CCE-ADM-1-C3970
[181] Ingetec-Sedic (2017) Informe mensual detallado de septiembre 2017 PHI-INM-LC1-073-R0. Page 37
[182] Ingetec-Sedic (2017) As-Built Drawing Rojo-Verde D-PHI-034-GDC-GE-C-040 Rev 0

The location of Tapón 15A is shown in Figure 27. The as built record refers to this as a "perforated wall"[183] and the drawing shows drainage tubes passing through it[184].

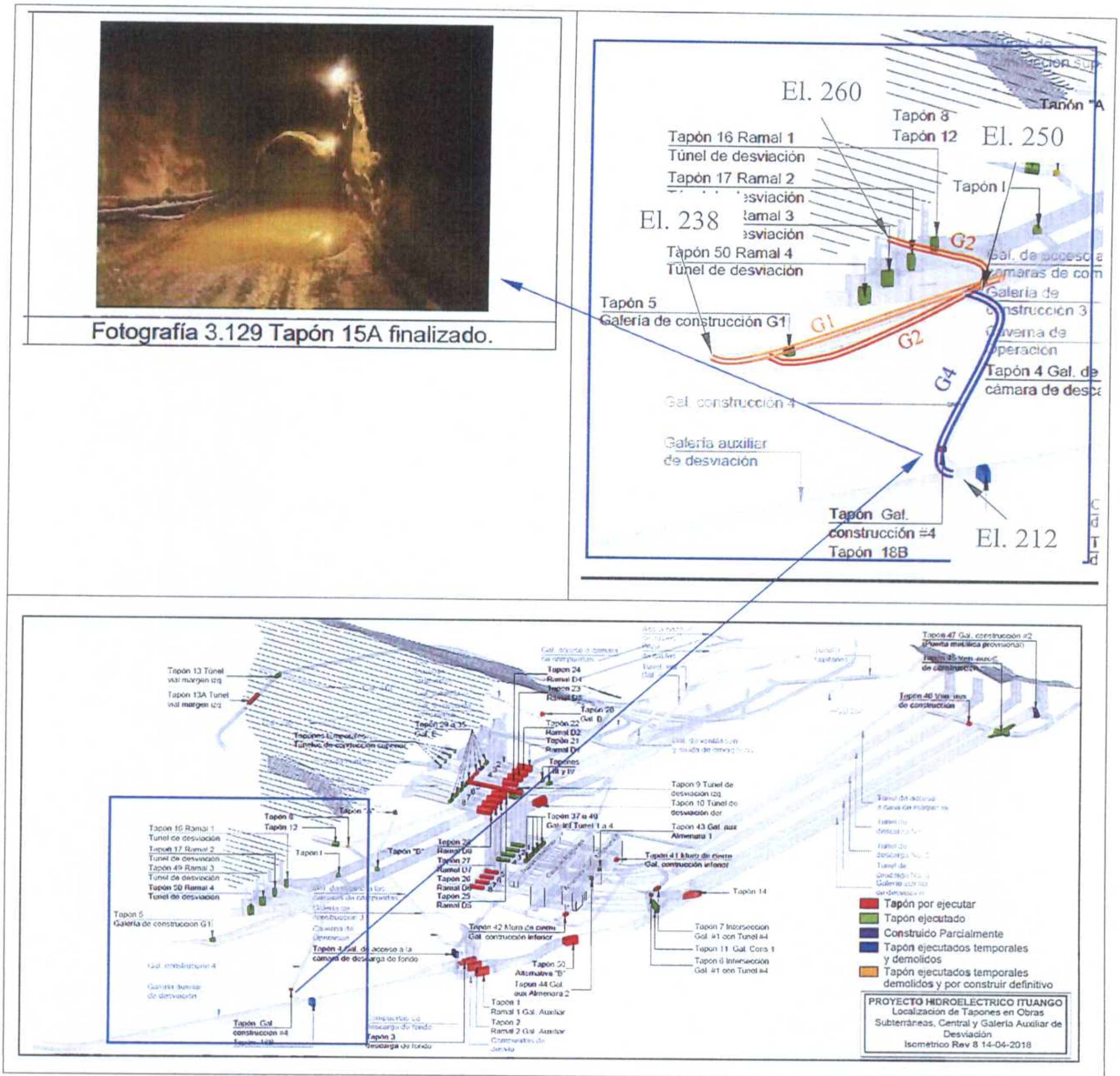


Figure 27. Location of concrete barrier (Tapón 15A)[185,186]. Elevations are at invert[187].

[183] D-PHI-O34-GEN-L0-C-010 REV 1.
[184] D-PHI-034-GDC-GE-C-040 REV 0
[185] Ingetec-Sedic (2017) Informe mensual detallado de Septiembre 2017 PHI-INM-LC1-073-R0
[186] Ingetec-Sedic (2018) OBRAS SUB. LOCA. TAPONES Rev. 8-Tapones
[187] Integral (2017) Letter INT-OC-CCCI-0569-17 with Drawing

8.41. River level at the ADT portal was measured at Puente Pescadero with varying frequency until April 5, 2018 and estimated from measurements at Station Olaya about 70 km upstream thereafter (Figure 28). The measurements were stopped due to a rise in the river evel inundating the station. The upstream portal of Construction Gallery No. 1 connects to the reservoir at elevation 238 masl, then connects to Construction Gallery No. 2 and connects to Construction Gallery No. 4 at elevation 250 masl, which then connects to the ADT at elevation 214 masl. The elevations of the Construction Galleries No. 1, No. 4 and the river level during operation of the ADT are shown in Figure 28. The river reached the ADT through construction galleries No. 1, No. 2 and No. 4 between April 12, 2018 and April 18, 2018 when the river lever surpassed elevation 250 masl. However, the design of the ADT was for water flow through the main tunnel only, and for a design flood with a 50-year return period, which corresponds to a river level 352.8 masl[188].

Figure 28. River Level at ADT upstream portal, measured and estimated[189]

8.42. Photographic records understood to date from prior to April 28, 2018 show accumulation of tree logs in the reservoir area near the ADT intake portal[190], which were retained by a mesh to prevent their ingress into the ADT. The Figure 29 shows a sketch comparing the

[188] Integral (2018) Compilation of Design Calculations: I-M-2194-034-HID-02-R0, Page 1.1.
[189] Integral (2018) Spreadsheet Desviaciones Cauca Velocidades
[190] Integral (2019) Taponamiento GAD - Estudio de Causa Raíz. I-I-2194-034-TAP-01, Anexo 4, Page 3.

design drawings of the ground reinforcement at the ADT portal[191] to the photographic record. The composite on Figure 29 indicates the reservoir level on the days prior to April 28, 2018 was about Elev. 234 - 235 masl, which coincides with the data from Figure 28, and is 6 to 7 m above the crown of the ADT portal. The floating tree logs at this minimum reservoir level are above the crown of the ADT portal, and the reservoir level did not drop below the crown (Figure 28) such that floating logs could enter the tunnel. Furthermore, there are no records provided of failure of the mesh at the time allowing tree logs to flow rapidly and in large quantities into the ADT.

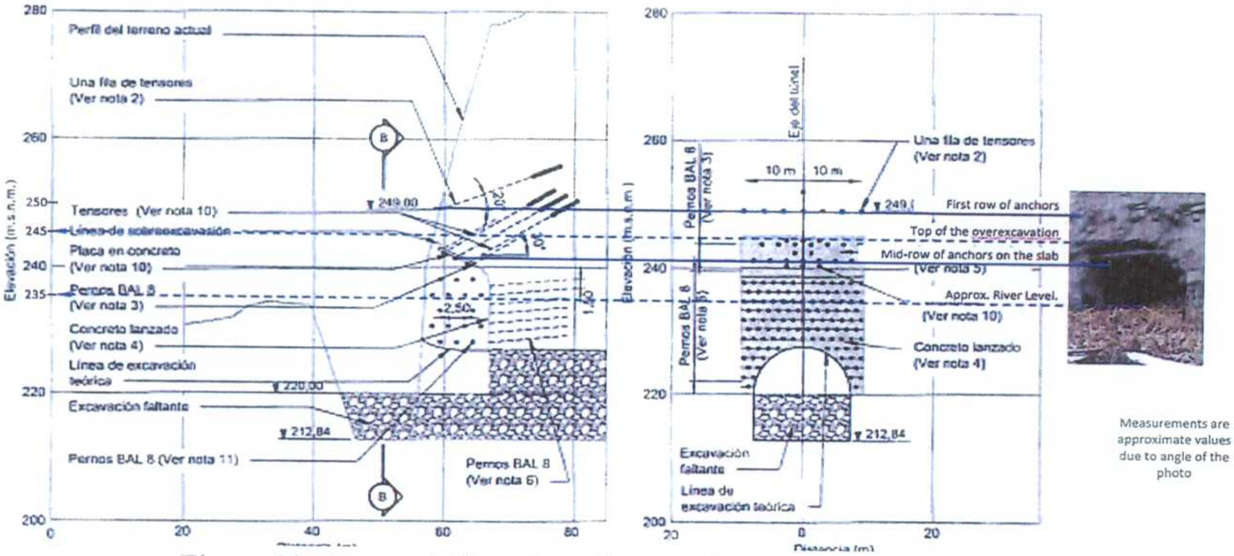


Figure 29. Accumulation of tree logs on days prior to April 28, 2018

8.43. It should be noted that as the river level increased the water could enter through G1 and G2. We are not aware from the information provided of any barrier or netting being installed at the portal of these tunnels but the partial concrete barrier in G4 (¶ 8.40) would prevent larger debris entering the ADT but some floating debris could still enter the tunnel from this point within the period the water level passes over the portal.

[191] Integral (2017) Planta y Secciones Drawing E-PHI-034-PDE-EX-C-001 Rev 1

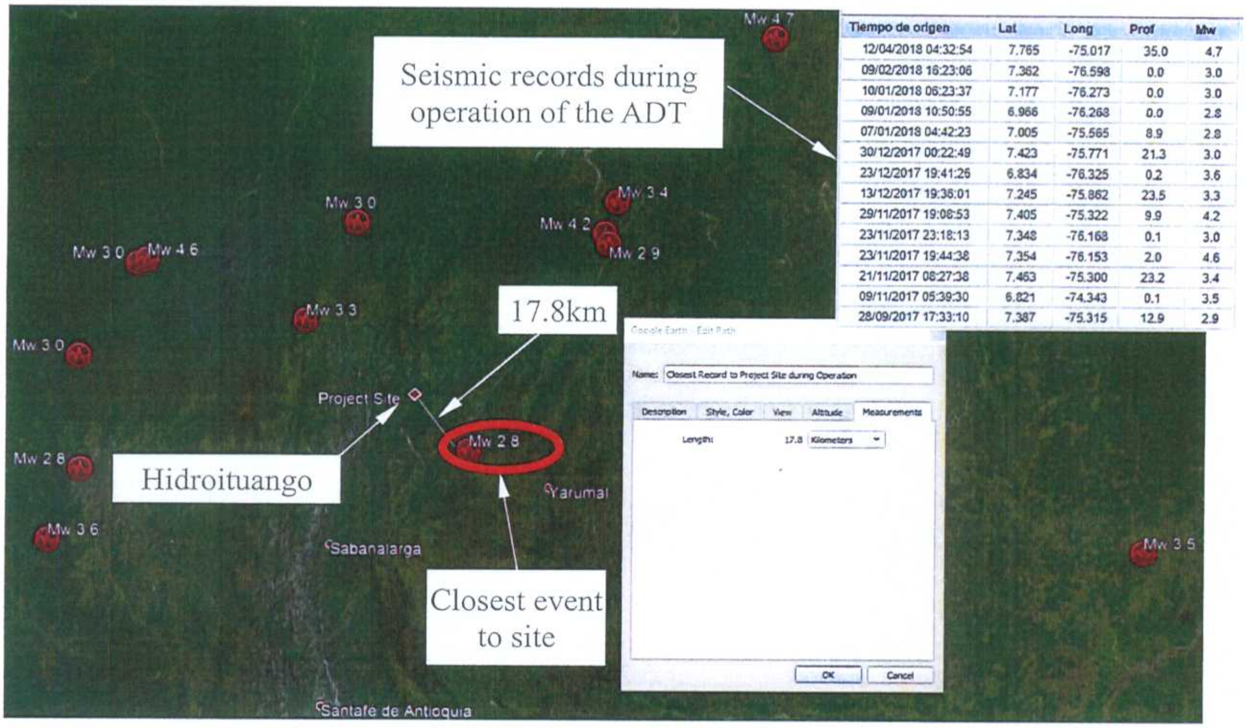
9. ANALYSIS OF THE COLLAPSE

Seismic records

9.1. Seismic events have been dismissed as causative because, although the area experiences earthquakes, no events were strong enough to cause damage.

9.2. The regional seismic system is shown in Figure 30 using information provided by the Colombian Geologic Service (*Servicio Geológico Colombiano* SGC). The Figure 30 shows the relation between the site and the epicenter of all seismic events with a magnitude in Mw greater than 2.0 (which is the about threshold felt by humans at rest) registered and reported by the SGC between operation of the ADT, September 22, 2017 and April 28, 2018.[192] The magnitude Mw of all seismic events registered during the operation of the ADT was lower than 5.0, which is about the threshold where some damage starts to occur. The closest event was recorded on January 7, 2018, as 2.8 magnitude Mw, 17.8 km south east from the project site, which is just above the level perceptible to some humans, and the highest level recorded was on the April 12, 2018 at a distance greater than 100 km from the site of 4.7 Mw, lower than the 5.0 threshold for the start of damage.

[192] Visor de Catálogo de Sismos. Servicio Geológico Colombiano, June 15 2019, http://catalogosismico.sgc.gov.co/VisorCatalogoSismos/mapa.html.



Tiempo de origen	Lat	Long	Prof	Mw
12/04/2018 04:32:54	7.765	-75.017	35.0	4.7
09/02/2018 16:23:06	7.362	-76.598	0.0	3.0
10/01/2018 06:23:37	7.177	-76.273	0.0	3.0
09/01/2018 10:50:55	6.966	-76.268	0.0	2.8
07/01/2018 04:42:23	7.005	-75.565	8.9	2.8
30/12/2017 00:22:49	7.423	-75.771	21.3	3.0
23/12/2017 19:41:26	6.834	-76.325	0.2	3.6
13/12/2017 19:36:01	7.245	-75.862	23.5	3.3
29/11/2017 19:08:53	7.405	-75.322	9.9	4.2
23/11/2017 23:18:13	7.348	-76.168	0.1	3.0
23/11/2017 19:44:38	7.354	-76.153	2.0	4.6
21/11/2017 08:27:38	7.463	-75.300	23.2	3.4
09/11/2017 05:39:30	6.821	-74.343	0.1	3.5
28/09/2017 17:33:10	7.387	-75.315	12.9	2.9

Figure 30. Seismic events in Antioquia during operation of the ADT between September 2017 and the end of April 29, 2018

Description of the Event of April 28, 2018

9.3. A summarized sequence of events (Figure 31), the changes in reservoir level (Figure 32) and the flow conditions (Figure 33) was presented in the Asesoria report of January 2019.[193]

[193] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2

Tabla 3-1. Descripción de los eventos mostrados en la Figura 3-1.

ID	Fecha y hora [local]	Evento	Nivel del embalse [m.s.n.m]	Señales registradas					
				Fecha y hora [local]	Hora [UTC]	Latitud [°]	Longitud [°]	Profundidad [km]	Magnitud [ML]
1	28/04/2018 21:00	Primer taponamiento GAD	239,2	28/04/2018 20:56:58*	01:56:58	7,114	-75,654	N/A	N/A
			239,2	28/04/2018 20:57:54	01:57:54	7,124	-75,669	2,3	1,1
2	29/04/2018 19:00	Destaponamiento natural de la GAD	277,92	-	-	-	-	-	-
3	30/04/2018 12:30	Taponamiento GAD (Chimenea)	268,6	30/04/2018 12:43:09	17:43:09	7,126	-75,662	2,3	1,3
4	07/05/2018 01:30	Taponamiento entrada túneles de desviación. Deslizamiento Zona B.	314,37	07/05/2018 01:29:17*	06:29:17	7,127	-75,664	N/A	N/A
5	09/05/2018 11:30	Aumento en el nivel de la descarga 4	340.46	09/05/2018 12:06:00*	17:06:00	N/A	N/A	N/A	N/A
6	10/05/2018 19:00	Apertura del flujo a través de CM	353,74	-	-	-	-	-	-
7	11/05/2018 02:00	Descarga del flujo a través de CM	356,42	-	-	-	-	-	-
8	12/05/2018 15:00	Destaponamiento natural del TDD (Emergencia Puerto Valdivia)	363,3	-	-	-	-	-	-
9	12/05/2018 18:30	Taponamiento natural del TDD. Deslizamiento Zona B.	361,4	12/05/2018 18:10:25*	23:10:25	7,133	-75,659	N/A	N/A
10	16/05/2018 12:15	Salida natural de flujo por la galería 284 (acceso C.M.)	368,51	16/05/2018 12:07:36*	17:07:36	7,138	-75,645	N/A	N/A
11	20/05/2018 02:00	Obstrucción en conducciones N°7 y N°8. Disminución en la descarga.	367,32	20/05/2018 03:13:00	08:13:00	N/A	N/A	N/A	N/A
12	26/05/2018 15:10	Cierre compuerta captación N°8	380,91	-	-	-	-	-	-
13	26/05/2018 15:31	Cierre compuerta captación N°7	380,91	-	-	-	-	-	-
14	26/05/2018 17:07	Deslizamiento parte superior plazoleta de compuertas. Zona Romerito.	380,99	26/05/2018 17:07:56*	22:07:56	7,141	-75,621	N/A	N/A
15	06/07/2018 17:30	Obstrucción. Disminución en caudal de descarga.	381,85	06/07/2018 17:30:00	22:30:00	N/A	N/A	-	-

*Evento asociado a desprendimiento de material. La localización fue realizada usando únicamente ondas P.
Notas: Hora UTC (Universal Time Coordinated): En Colombia la hora local oficial es UTC menos 5 horas.
GAD: Túnel auxiliar de desviación. TDD: Túnel de desviación derecho. CM: Casa de máquinas.
Fuente: Integral S.A. (2018).

Figure 31. Table extracted from I-I-2194-062018-01-R2 summarizing the sequence of events

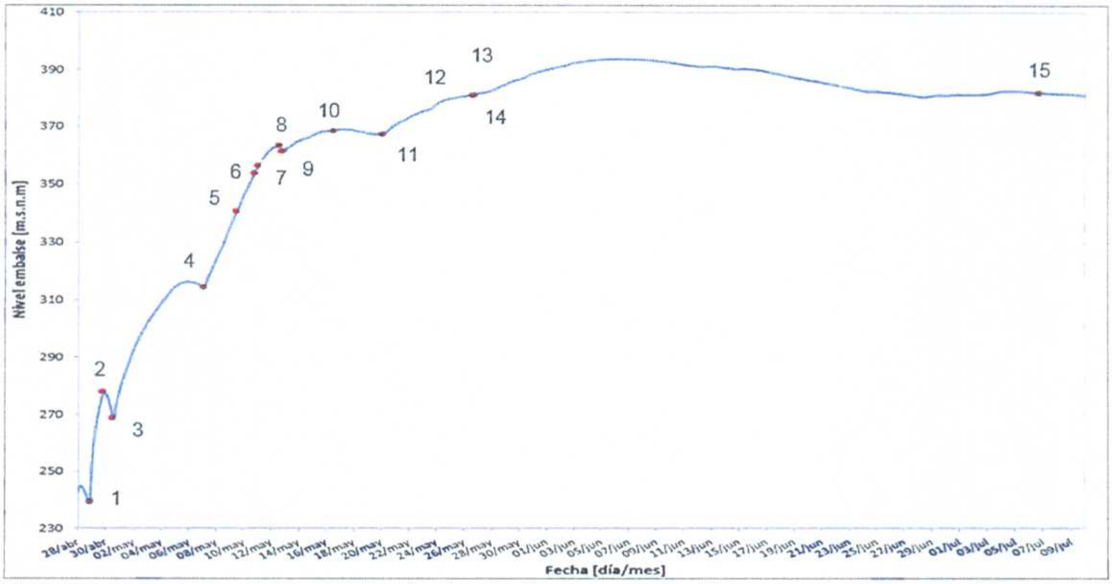


Figura 3-1. Secuencia de eventos

Figure 32. Graphical presentation of the sequence of events against the reservoir level

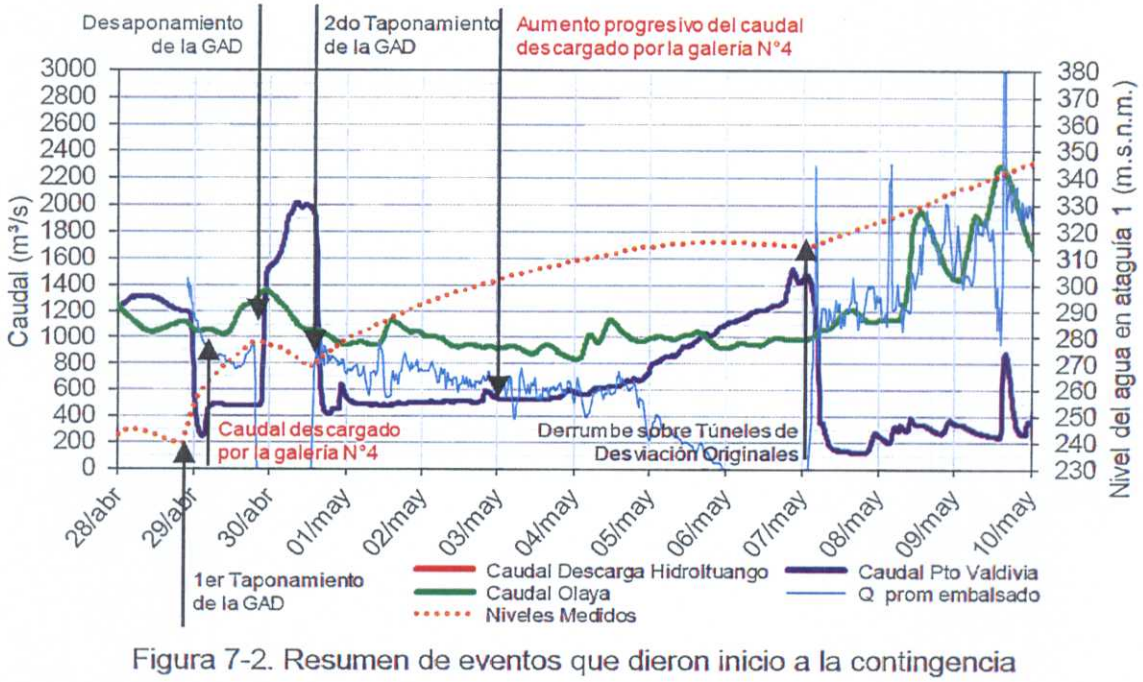


Figure 33. Graphical representation of the sequence of events against the estimated flow and river level at Ataguia 1 from April 28 to May 10, 2018

9.4. The description of the event of April 28, 2018 and the subsequent events is based on several sources and documents provided by EPM [194] [195] [196] [197] [198] [199] [200]. The description refers to the tunnels, portals and galleries for the Project. The layout is presented in Figure 3. Sectorization of the ADT, above for guidance.

9.5. On April 27, 2018, the reservoir level was about 242.6 masl, around 14.6 m above the crown of the tunnel and the average flow was estimated by the Asesoria as 1,198 m^3/s.

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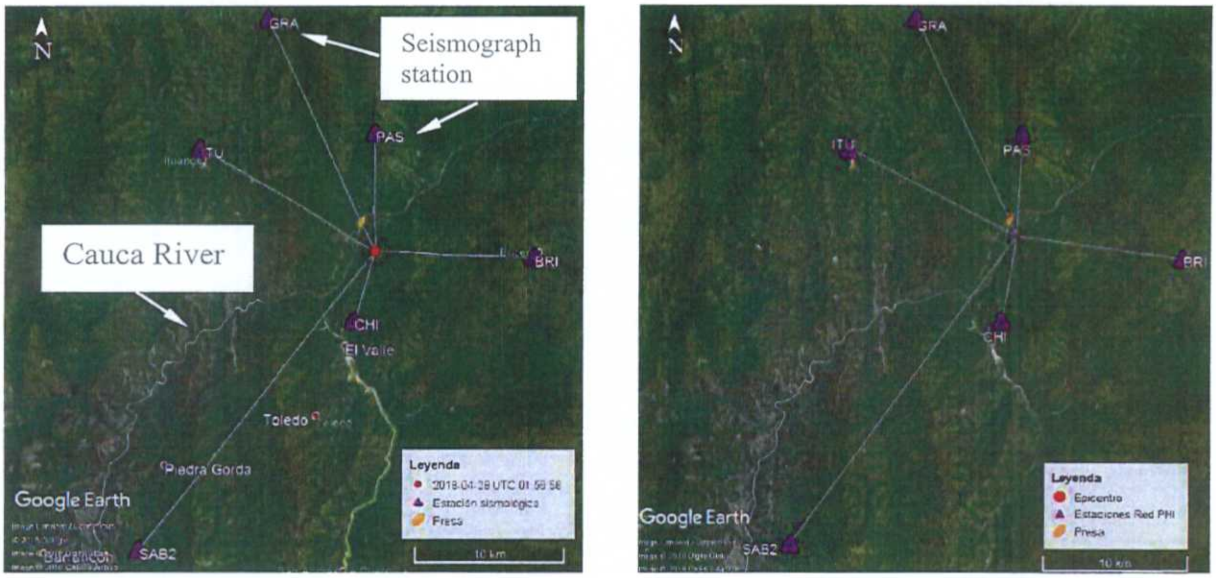
[194] Integral (2018) Análisis de Taponamiento Galería Auxiliar de Desviación I-I-2194-052018-01-R0
[195] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0
[196] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2.
[197] Skava (2019) Informe de estudio de causa raíz física
[198] Skava (2019) Informe complementario
[199] Integral (2019) Taponamiento GAD - Estudio de Causa Raíz. I-I-2194-034-TAP-01
[200] Integral (2019) Taponamiento GAD - Estudio de Causa Raíz. Informe Complementario. I-I-2194-034-TAP-02

9.6. The seismographs installed by the Project reacted to a release of energy at 20:56 and 20:57 on April 28, 2018. The signal was interpreted to be too weak for an earthquake and it was concluded by the Asesoria that it was related to a collapse of the ADT.



20h:56m:58s Magnitude unknown

20h:57m:54s ML 1.1

Figure 34. Local network and records near the ADT on April 28, 2018[201].

9.7. All the reports provided concurred that at around 21:00 hrs. on April 28, 2018 the flow through the ADT reduced. At this time personnel working on boats to remove floating vegetation in the area of the ADT portal saw a wave of water and heard a rush of air coming from the ADT portal[202]. There was a reasonable conclusion that there was a complete but temporary blockage of the tunnel at this time.

9.8. The reservoir level at the time of the event was 239.2 masl which was used by the Asesoria to estimate a flow through the ADT of 1,300 m^3/s at a velocity of 7.5 m/s[203].

9.9. At this time, the river flow measured at a downstream station (Valdivia) fell from about 1,190 m^3/s to about 230 m^3/s over three hours. The reservoir level rose from 239.2 masl

[201] Integral (2018) Red Sismológica Noviembre 2018, Pages 22 and 24
[202] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, Page 473
[203] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2, Page 7.169

to 277.98 masl at a rate of 1.8 m/hour with a simultaneous reduction in the level of the river downstream of the Discharge Tunnel 4 exit portal.[202] The locations of the measuring stations are shown in Figure 35.

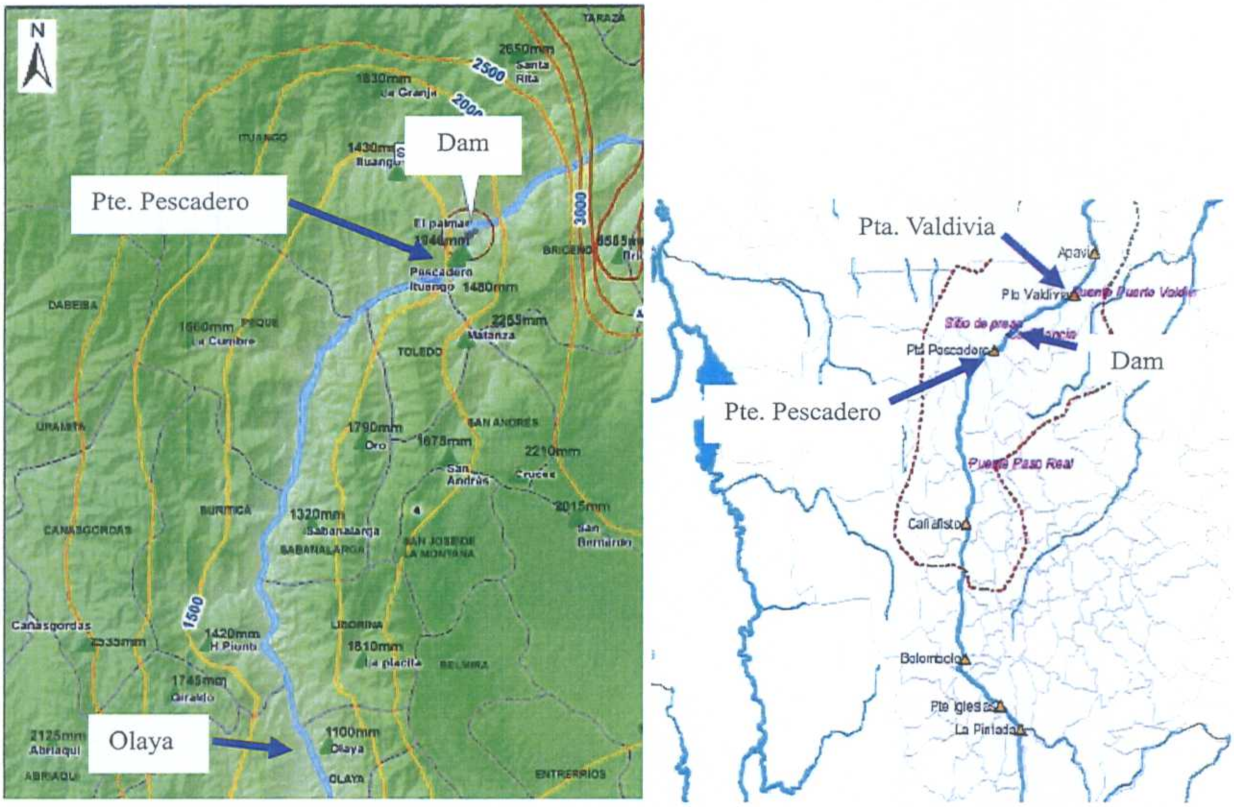


Figure 35. Location of water level and flow measuring stations.

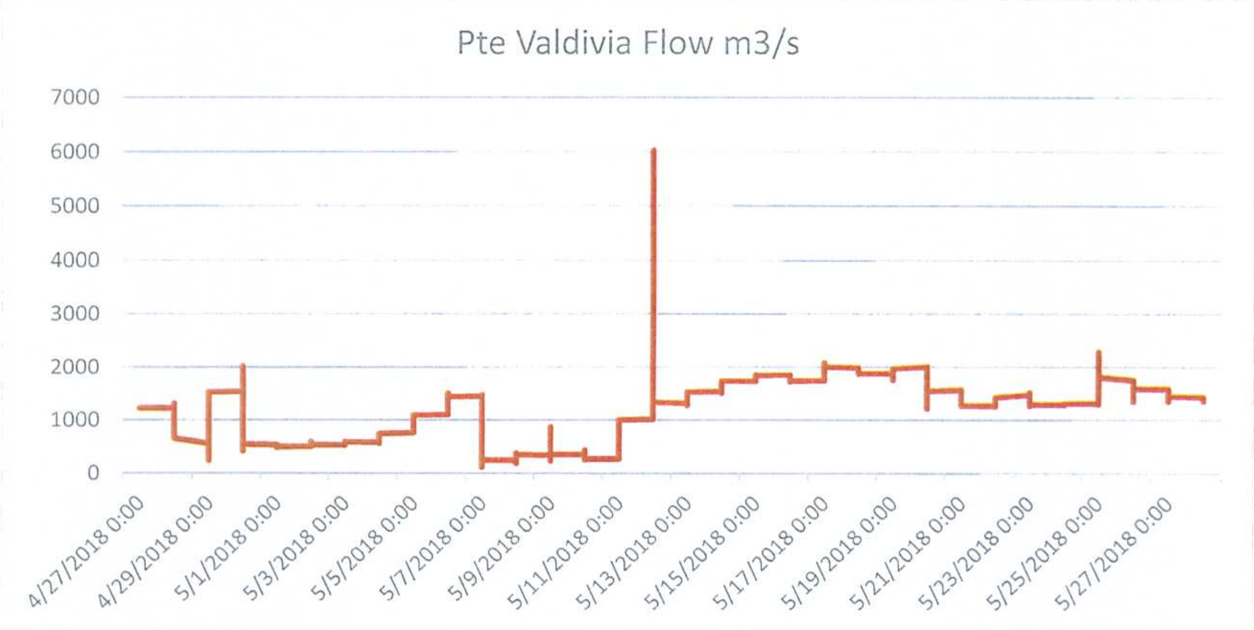


Figure 36. 15 minute interval flow records from Puente Valdivia

9.10. The discharge from the ADT gradually recovered on the night of the April 29, 2018[204], "presumably[205]" from flow through construction galleries 1 and 4[196] which is valid because flow was possible through the construction galleries G-1, G-2 and G-4 when the reservoir level was higher than 250 masl. (Figure 25).

9.11. The reports stated[206 207] that the ADT unblocked at 19:00 hrs. on April 29, 2018, and flow was restored downstream at Puerto Valdivia at 1,500 m^3/s, which then increased to 2,000 m^3/s on the morning of April 30, 2018 and the reservoir level fell to 268 masl with flow also possible through G1 and G2 to G4 and the ADT[208].

9.12. Another release of energy was recorded by the local seismographs at 12:43 hrs. on April 30, 2018[209] and at about 14:00 hrs. on that day a cone-shaped sinkhole approximately

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[204] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, pp. 473

[205] "Presuntamente" after Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2

[206] Consorcio Generación Ituango, May 2018, Análisis de Taponamiento Galería Auxiliar de Desviación, I-I-2194-052018-01-R0.

[207] Consorcio Generación Ituango, February 2019, Taponamiento GAD- Causa Raíz, I-I-2194-034-TAP-01-R0, Page 33

[208] Consorcio Generación Ituango, January 2019, Informe Geología y Geotecnia, I-I-2194-062018-01-R2

[209] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2, Table 3-1, Page 3.34

100 m in diameter narrowing to about 12m in its interior appeared at the ground surface above the ADT[210] (Figure 37, Figure 38, Figure 39[211]). Reservoir levels began to rise from 13:00 hrs. onwards.

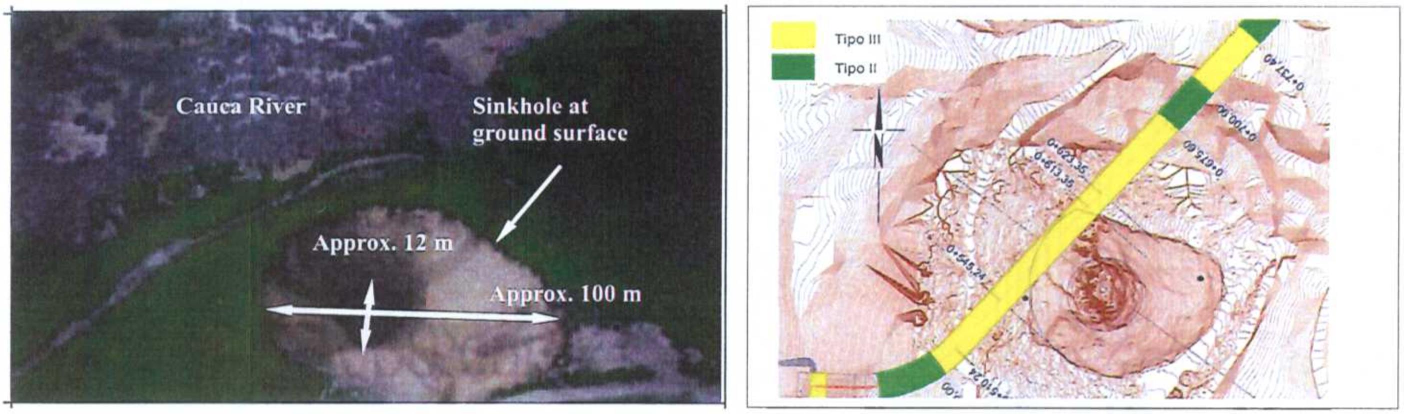


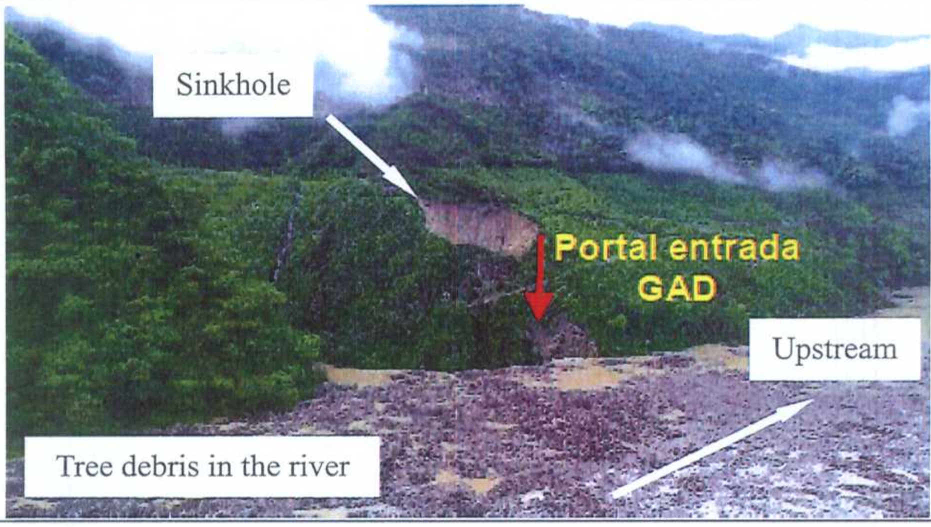
Figure 37. View of the sinkhole at the ground surface April 30, 2018

Fotografía 2. Vista interna del desprendimiento

Figure 38. Close up photograph of interior of sinkhole

[210] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, pp. 473

[211] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2



Fotografía 16.1 Vista del derrumbe en la ladera del portal de la Galería Auxiliar de Desviación 30 de abril de 2018

Figure 39. Photograph showing sinkhole and landslide above the ADT on April 30, 2018[212]

9.13. The tunnel was completely blocked coincident with the appearance of the sinkhole and the reservoir began to rise at a rate of 0.59 m/hour. Flow downstream at Puerto Valdivia reduced from around 2,000 m^3/s at 13:45 to around 400 m^3/s[213] by 18:00, presumably with flow continuing through the construction gallery system. The reservoir level exceeded 278 masl by midnight on the 30th April 2018.

9.14. On May 1, 2018 at 09:00 hrs. the reservoir level reached 284.3 masl[214] and the flow of water through Discharge Tunnel 4 increased gradually to approximately 600 m^3/s up to May 5, 2018[213] presumably due to continued flow through the construction galleries with an increasing head of water.

9.15. Work began on the 'Contingency Plan' on May 1, 2018 and actions included an attempt to blast the 22 m plug in the LDT but this was unsuccessful, and the plug remained in

[212] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0
[213] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2
[214] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0

place[215]. Also, holes were drilled through the plug of the IDT presumably in preparation for blasting of the plug to divert some of the flow[215], Note that the IDT would have a lower capacity than the deviation tunnels because it was smaller and the head of water lower for a given reservoir level.

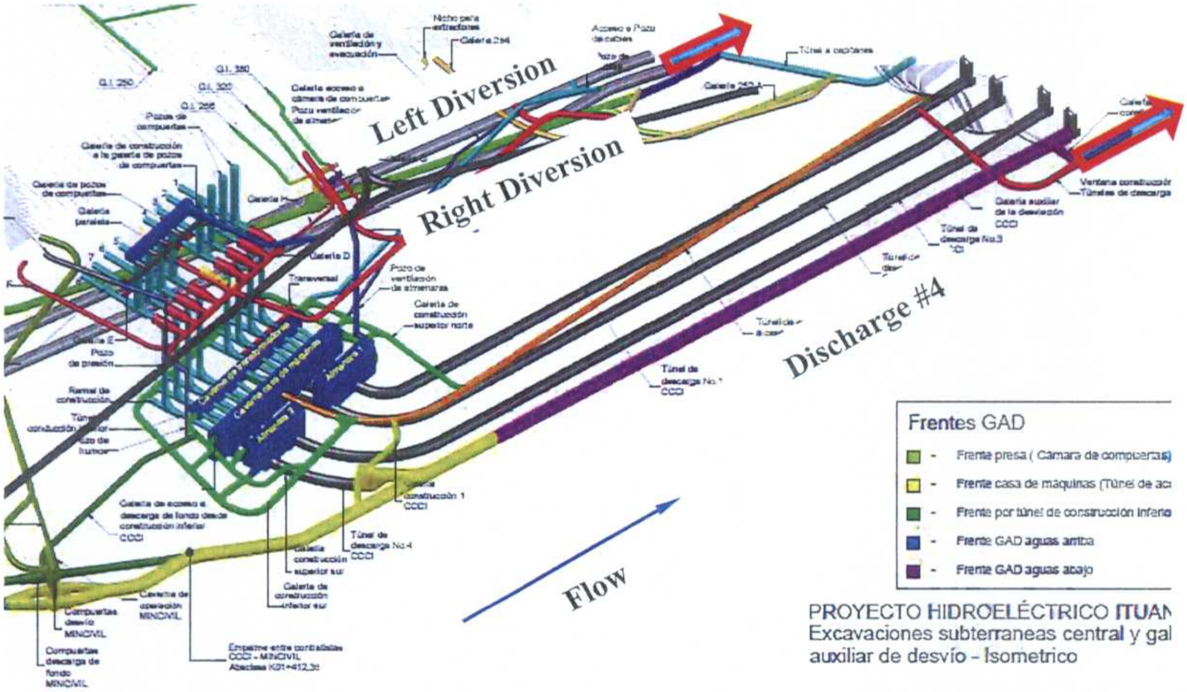


Figure 40. General Layout highlighting location of; Discharge No. 4, LDT and RDT.

9.16. Also, during the contingency works between May 4 and 6, 2018, the water level in the RDT increased and a surge of water at the tunnel discharge displaced the pumping system that had been installed there to dewater the RDT[215] since March 2018[216] when the RDT was plugged. It was reported that at the same time flow through Discharge Tunnel 4 was also increasing resulting in a slight reduction in reservoir levels until 14:00 hrs. on May 6, 2018 when flow through this tunnel reduced and the reservoir level started to increase again[215]. This indicates that the plugs for the RDT were either leaking or damaged shortly after being installed and the dramatic surge that displaced the pumping system on May 5, 2018 could have been either a collapse of the tunnel or actual failure of the plug(s). It

[215] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0

[216] Ingetec-Sedic (2018) Informe mensual detallado abril 2018. PHI-INM-LC1-080-R0, Section 3.8.1

should be noted that from the Pte. Valdivia data this actually seems to have occurred later on May 7, 2018.

9.17. On May 7, 2018 there was a landslide on the right bank of the river which further reduced discharge from Discharge Tunnel 4 exit portal and which also covered the two branches of the RDT (Figure 42).[217] The slope above the access portal to Galleria 1 was also affected, and there may have been a collapse of Galería No. 1 at the intersection with Galería No. 4.[218] A release of energy was recorded at 01:29 hrs.[218] Flow recorded downstream, at Puerto Valdivia reduced from a peak of 1,516 m^3/s at 20:45 on May 6, 2018 to a low of 120 m^3/s by 18:00 hrs. on May 7, 2018 before starting to rise again to 278 m^3/s by 23:00 hrs.

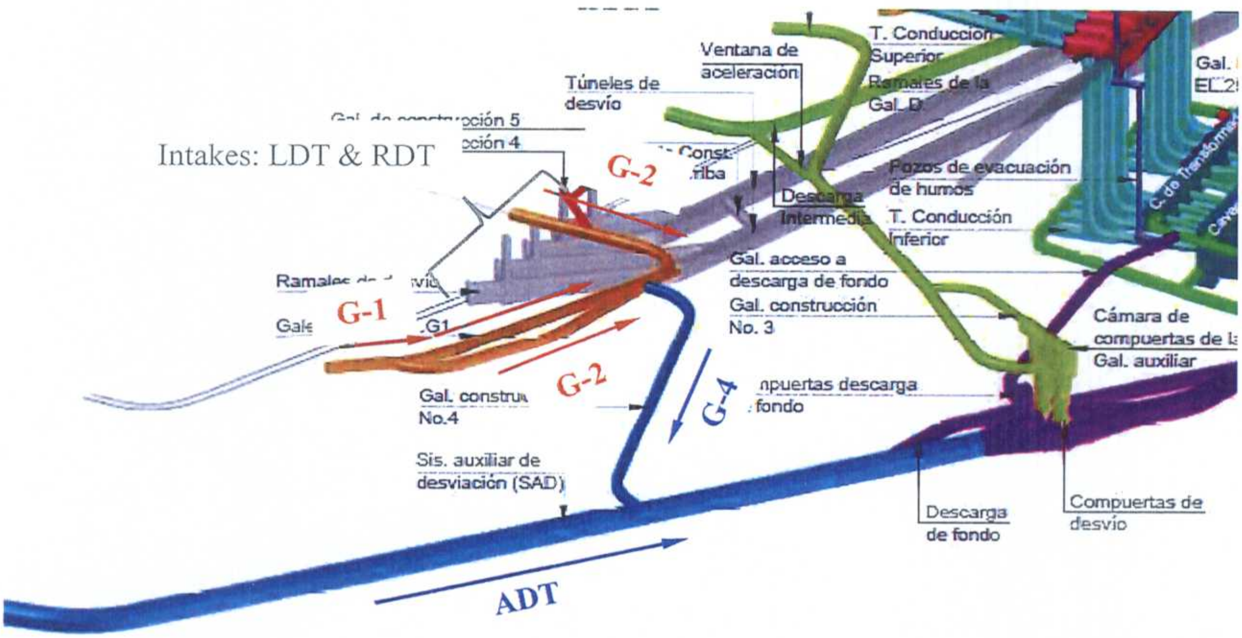


Figure 41. General Layout showing the location of LDT and RDT Intakes and Construction Galleries G-1, G-2 and G4

[217] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0
[218] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2

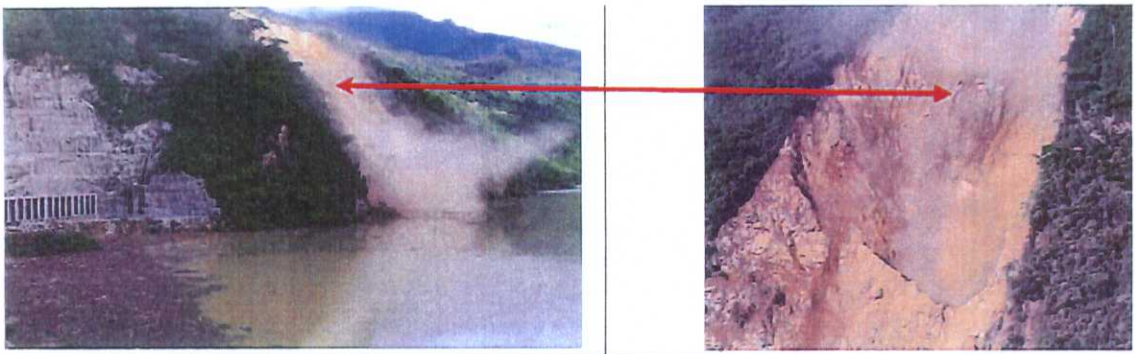
	
Fotografía 16.5 Segundo desprendimiento en la margen derecha, aguas abajo del depósito Tenche. 7 de mayo de 2018.	Fotografía 16.6 Segundo desprendimiento en la margen derecha, aguas abajo del depósito Tenche. 7 de mayo de 2018.

Figure 42. Landslide on the right bank of the river May 7, 2018 [219]

9.18. Following this event, the reservoir rose a rate of 0.42 m/hr. to 314.37 masl.[220]

9.19. On the morning of May 9, 2018, there was a considerable increase in flow from the exit portal of the RDT[220] and a release of energy was recorded at 12:06 hrs. At 16:30 hrs. there was another landslide on the right riverbank that blocked flow through that tunnel resulting in rising reservoir levels at a rate of 0.91 m/hour (Figure 43)[221]. A chimney appeared near the intersection of Galleries 2 (Asesoria refers to G1) and 4[221], (Figure 43 and Figure 44). Flow measured downstream at Pte. Valdivia increased from 236 m^3/s at 14:30 hrs. on May 9, 2018 to a peak of 875 m^3/s by 16:00 hrs., before falling again to 260 m^3/s by 21:30 hrs. of the same day.

[219] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, Photo 16.5 and 16.6
[220] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0
[221] Ingetec 2018 (PHI-IFF-LC1-011-R0, Page 475



Fotografía 16.7 Activación del derrumbe inicial generando un nuevo desprendimiento de material en la margen derecha. 9 de mayo de 2018.

Fotografía 16.8 Activación del derrumbe inicial generando un nuevo desprendimiento de material en la margen derecha. 9 de mayo de 2018.

Figure 43. Landslide and sinkhole above Gallerias 1 and 4 May 9, 2018[222] .

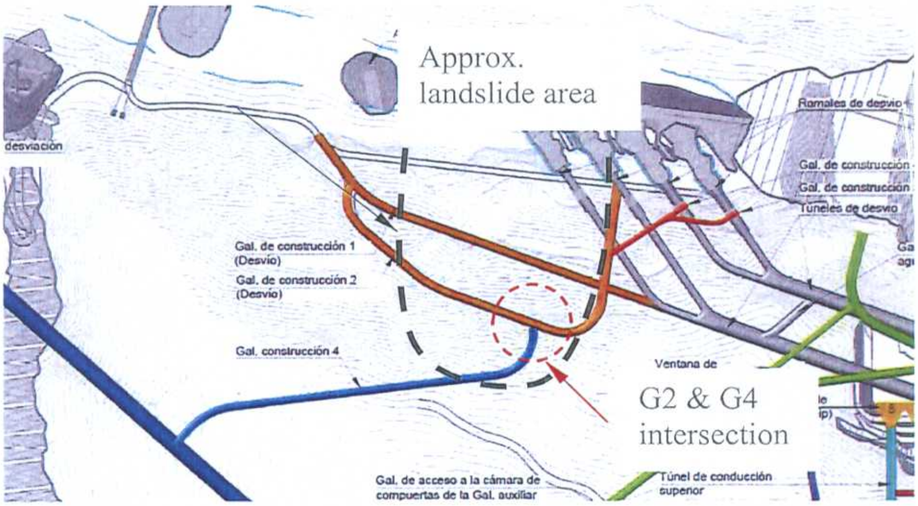


Figure 44. Area of landslide and sinkhole above Galleries 2 and 4.[223]

9.20. At this point, on May 9, 2018 EPM decided to divert reservoir water through the powerhouse complex and at 19:00 hrs. on May 10, 2018 water started to flow through intakes 1, 2, 7 and 8. When this was done water began to flow through the exit portals of Discharge tunnels 1, 2 and 3 and during the course of May 11, 2018 the flow through the

[222] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0 Photo 16.7 and 16.8 0

[223] Integral (Undated) Project Layout. Filename CENTRAL UNIFICADO V.14-PLANTA

three tunnels increased (Figure 45)[224]. The reservoir level at the time of this diversion was 353.74 masl continuing to rise to 356.42 masl at 02:00 hrs. on May 11, 2018.

	
Fotografía 16.9 Entrada del agua por las captaciones 1, 2, 7 y 8 a la Central Subterránea.	**Fotografía 16.10** Aumento de caudal en las Descargas.

Figure 45. Photographs showing the conditions on May 11, 2018[225]

9.21. At 15:30 hrs. on May 12, 2018, it was reported that there was an increase in flow from the exit portal of the RDT reaching a peak of about 5,000[226] or 6,000 m^3/sec[227] and the reservoir level which had reached 362.56 masl began to decrease rapidly to 361.36 masl (Figure 46). This flow was again blocked by a further movement of material from the previous landslide area and reservoir levels again began to increase[227]. Also, a release of energy was recorded by the local seismographs at 18:10 hrs. on May 12, 2018.[228] Flow measured downstream at Pte. Valdivia reached a peak of 6,025 m^3/s at 18:45hrs. on May 12, 2018.

[224] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0
[225] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, Photo16.9 and 16.10
[226] Skava (2019) Informe de estudio de causa raíz física and [228]
[227] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0
[228] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2

Fotografía 16.11 Aumento de caudal saliente del túnel de desviación derecho.

Fotografía 16.12 Aumento de caudal saliente del túnel de desviación derecho.

Figure 46. Photographs supporting observation of increase flow May 12, 2018[229] .

9.22. On the morning of May 16, 2018, damage was found at the exit canal of Discharge Tunnel 3 due to the force of water flow. At 11:10 hrs. water was recorded flowing from the Main Access Tunnel to the Powerhouse and at about mid-day from Galleria 284 (Figure 47) with air flow[230] as well as a reduction in flow from the outlet portal of Discharge Tunnel 3, from a blockage.[231] A release of energy was recorded at 12:07 hrs. on May 16, 2018.[232]

[229] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, Photo 16.11 and 16.12
[230] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2
[231] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0
[232] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2

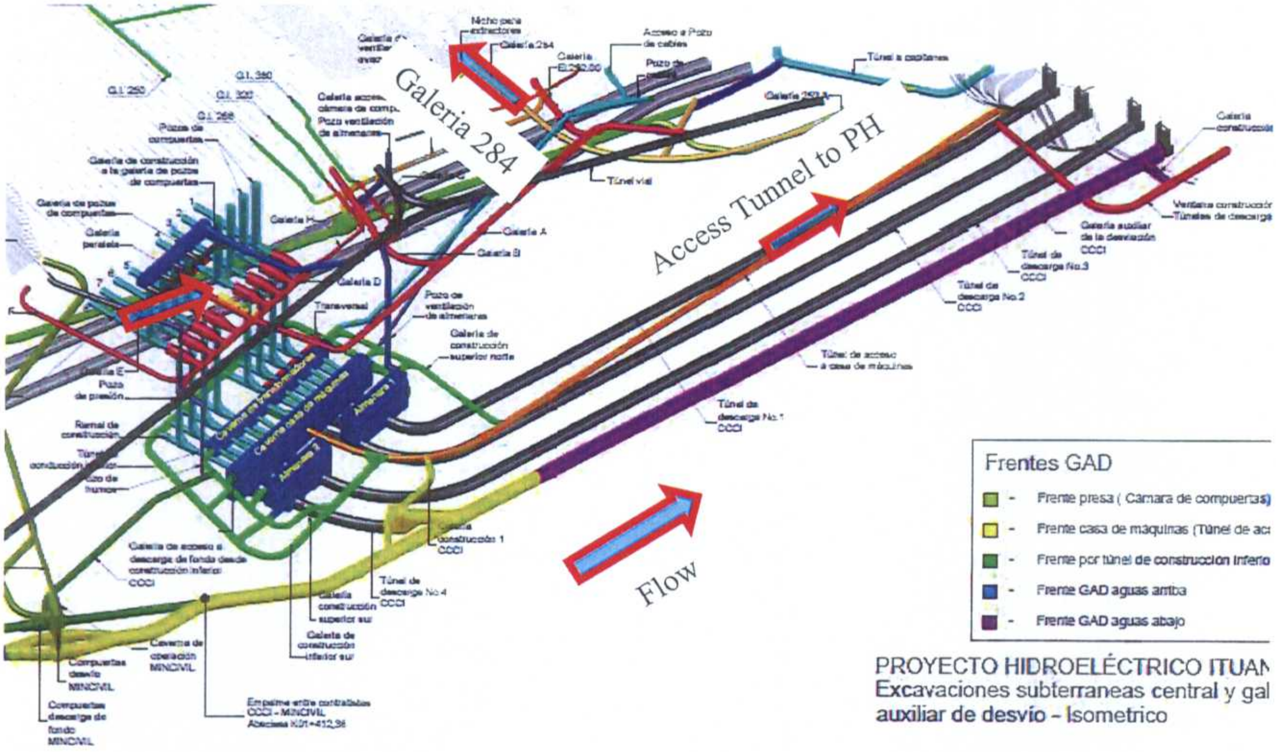


Figure 47. Location of the Flows recorded on May 16, 2018

Figure 48. Photographs of the events at Galleria 284 May 16, 2018[233]

[233] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, Photograph 16.15 - 16.18

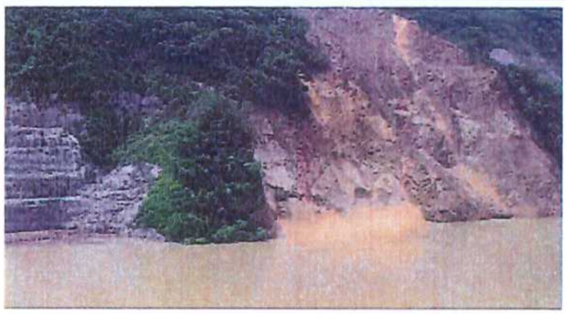
9.23. Flow continued through Galleria 284 and increased during the day on May 17, 2018, which then reduced on the night shift.

9.24. On May 18, 2018 at 12:20 hrs., there was a renewal of "moderate flow" which ceased at 13:30 hrs. on the same day[234]. Flow continued through Discharge Tunnels 1 and 2 and the reservoir level continued to decrease at a rate of 0.04 m/hour[234] with a rate of flow through the Powerhouse system constant through May 19 and 20, 2018 at between 1,700 and 1,800 m^3/sec[234].

9.25. The flow through the Powerhouse reduced on May 20, 2018, associated with a blockage of Intakes 7 and 8 and a release of energy was recorded at 03:13 hrs.[235]. At 19:00 on May 20, 2018 flow through the RDT restarted[234].

9.26. On the morning of May 21, 2018 the landslides above the diversion tunnel intakes and above the Gate Chamber Platform re-activated and cracks formed in the Gate Chamber (Gates 5 to 8) and the associated Construction Galleries (Figure 49)[234] and water continued to flow through the RDT outlet portal at about 50 m^3/sec[236].

Fotografía 16.19 Activación del derrumbe sobre los ramales de la desviación, margen derecha.



Fotografía 16.20 Activación del derrumbe sobre los ramales de la desviación, margen derecha.

Figure 49. Reactivation of landslide above the Diversion tunnel intakes May 21, 2018[237]

[234] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0
[235] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2
[236] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2
[237] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, Photograph 16.19 and 16.20

9.27. This situation continued through to May 26, 2018 with flow through Discharge Tunnels 1 and 2, but no flow through Gallería 284 or the PH Main Access Tunnel, although there was intermittent pulsing of air through the latter.

9.28. Gates 7 and 8 from the Gate Chamber were closed between 15:10 and 15:30 hrs. on May 26, 2018, taking about 12 minutes each, in sequence[238]. At 17:00 hrs. on May 26, 2018 there was a landslide referred to as the "Romerito" in the slope above the same gates 7 and 8 (Figure 50, Figure 51). This landslide was associated with distortion and cracking of the support of the Tunnel Vial. There is a record of a release of energy from the local seismographs at 17:07 hrs. on May 26, 2018[239].

	
Fotografía 16.21 Derrumbe sobre la plazoleta de pozos de compuertas	**Fotografía 16.22 Derrumbe sobre la plazoleta de pozos de compuertas**

Figure 50. The "Romerito" landslide above the Gate Chamber on May 26, 2018[240]

[238] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2 page 3.38
[239] Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2
[240] Ingetec-Sedic (2018) Informe final de obras principales parte 3 – Sistema auxiliar de desviación, PHI-IFF-LC1-011-R0, Photo 16.21 and 16.22

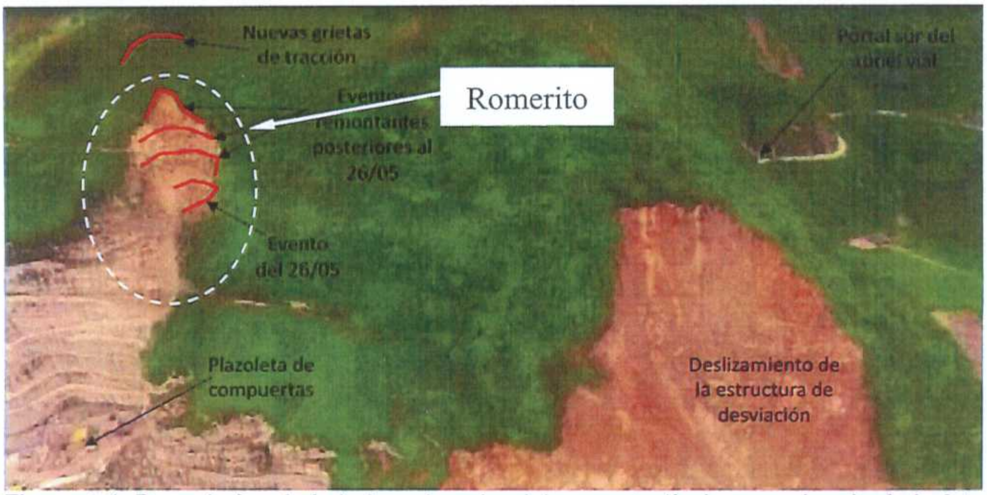


Figura 4-1. Panorámica de la ladera derecha del proyecto (fecha aproximada de la foto 28/05/2018)

Figure 51. Photograph showing the Romerito and the landslide above the deviation tunnel intakes[241]

Fotografía 4-1. Parte alta de la Plazoleta de Compuertas (junio 10/2018)

Figure 52. Close up photograph June 10, 2018 of the landslide above the Gate Chamber[242]

241 Integral (2019) Diagnostico Geologico Geotecnico Contingencia I-I-2194-062018-01-R2, Figure 4-1 showing the "Romerito"

242 Integral (2019) Diagnostico Geológico Geotécnico Contingencia I-I-2194-062018-01-R2

Analysis of the Collapse

9.29. The analysis of the collapse by the Experts is based on numerical methods for the initial failure of the tunnel support and kinematic wedge analysis for the collapse of the tunnel and progression of the collapse to the ground surface.

Numerical Analysis

9.30. The Experts developed numerical simulations to enrichen their understanding of the sensitivity of the ADT stability in the area of chainage 0+540 using geomechanical parameters derived using the face mapping information and derived from borehole P-GAD-02.

9.31. The purpose of the numerical analyses was to develop a model for the initial failure of the supported tunnel. The analyses were parametric, varying and testing the sensitivity of the following[243].

- Geomechanical properties of the rock in the region of the tunnel.
- Groundwater level conditions.
- Water level fluctuations inside the tunnel linked to the reservoir level.
- The as built tunnel profile.
- Erosion of the unlined floor and progression to the sidewalls.

9.32. The numerical simulations developed by the Experts show that the ADT at around chainage 0+540, suffered significant reductions in its safety associated with the large overbreak or fallouts that happened during excavation and significant level of deformation is predicted in the area of the shear zone.

[243] The numerical analyses were run using software Plaxis 2D, version 2018, a commercial software that models the ground and different types of structures using the finite element method. Different material models for the ground, shotcrete and rockbolts are available, simulating elasto-plastic behavior.

9.33. The ADT geometry, especially when the overbreak is incorporated, is sensitive to differences in water pressure from that in the rock mass and the water level inside the tunnel. Erosion that develop in the ADT floor, progressing to the rock mass in the tunnel's right side also leads to tunnel failure.

9.34. A full description of these analyses and the results of the numerical simulations are presented in Appendix E.

Kinematic Wedge Analysis

9.35. Standard analyses used that same method (UNWEDGE) as the Asesoria using the orientation, spacing and persistence of planes in the rock mass from the geological face maps. The purpose of this analysis was to investigate if the existing discontinuities form failure mechanisms that model the progress of the collapse from the shear zone in the tunnel towards the position of the sinkhole at the ground surface[244].

9.36. The structural geology as mapped in the region can be used for kinematic wedge analysis, and which was developed by the Experts only to verify possible mechanism of instability. These analysis show existence of families of discontinuities that form wedges which are compatible with the retrogressive collapse of material and proportionally larger volumes, blockage of the tunnel and appearance of the sink hole in the surface.

9.37. The analyses and the results are presented Appendix E.

[244] The software Unwedge, by Rocscience was used for these analyses. Its use is widespread as a tool to evaluate stability of rock wedges in tunnels in rock. The same software was used by the Asesoria for the ADT design.

10. CONCLUSIONS

10.1. The various parties involved in the project had defined responsibilities as stated in contracts. The Asesoria was responsible for the design process for the tunnels, the classification of the rock mass and the selection of the support. The Interventoria was responsible for ensuring the project was built according to the contract.

10.2. The Project operated a risk management programme at a high level, but it did not comply with the Tunnel Code of Practice because it did not have lower level construction stage risk management and did not control events using a risk mitigation program.

10.3. The design of the diversion tunnels was generally in accordance with the nominated codes, standards and normal industry practice. The observational method was a significant component of the design and construction process. Construction of the LDT and RDT followed the OM, for example the design was verified during construction using the convergence data. However, the OM was not correctly applied to the ADT; for example, the convergence was not measured soon enough to be used to verify the design and adjust the support.

10.4. The LDT and RDT was claimed to have functioned successfully during four years of operation, so the Asesoria used the experience and data from this and other tunnels to optimize the ADT support design. The ADT support was, however, lightened in comparison with the LDT and RDT such that it was vulnerable to a highly irregular shape, erosion and water pressure changes. Furthermore, the Asesoria's analysis showed that the support had to be fully installed and working before the face was advanced.

10.5. The rock mass in the ADT, was weaker around the shear zones than the design assumptions for Type III ground and it was allowed to relax more than expected by the design because the face was advanced before the grout was cured. Consequently, the excavated rock was not controlled by the support installed, and the support was not enhanced to compensate for over-relaxation of the rock, such as use of active support, to eliminate these weaknesses, as would have been done if the OM had been implemented correctly, as was done in the LDT and RDT. Furthermore, the steel mesh required for shear zones was not installed at the location of the collapse.

10.6. Also, the tunnel was not built in accordance with the design, nor the technical specification because the thickness of the shotcrete was not measured by depth pins, the blast lengths were excessive at times and a concrete floor was not installed to protect weak rock and shears zones from erosion by flowing water.

10.7. The tunnel showed clear signs of distress during construction because the Contractor made frequent use of additional support to advance the face (spiles), persistent overbreak, fall outs of rock and support from the crown and sidewalls, fissures and cracks in shotcrete. These signs of distress of the tunnel were not addressed by the Project as is required by the OM.

10.8. The parties advising EPM (Asesoria and Interventoria) and together with the Contractor were aware of where the work did not follow the design, where the construction did not produce the tunnel with the expected shape and support and where the tunnel collapsed during construction. Many of these issues were not remedied because those parties did not agree on the cause or whose responsibility it was to resolve them.

10.9. The support installed in the ADT was degraded by turbulent flow because it had a highly irregular shape and the floor was eroded because it did not have a concrete cover, which undermined the support from the start of operation. The parties had a critical divergence on cause of damage of the rock mass beyond the design assumptions in that the Asesoria considered the damage was poor blasting and the Contractor considered it to be pre-existing poor rock. This and other matters were not resolved, and the ADT was put into service with these vulnerabilities.

10.10. A flood event from April 1 to April 16 increased the pressure of the water in the rock mass of the shear zone at about k 0+540 by about 50 m equivalent head over invert level. On or about April 13, 2018, was the peak river level and these vulnerabilities manifested when the groundwater pressurized leading to an imbalance with the water pressure in the tunnel. The river level then dropped faster than the water could drain from the ground such that the pressure on the outside of the tunnel was greater than inside the tunnel, overloading the existing support causing it to fail, leading to fall out of key blocks of rock. This process of fall out of blocks and overloading of remaining support continued until there was a major collapse of the tunnel on April 28, 2018, that caused temporary blockage of the tunnel at around chainage k 0+540, that was breached on April 29, 2018, followed by a final major collapse that completely blocked the tunnel at k 0+540 and progressed to the surface on April 30, 2018.

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