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Stabilization of major soil masses using drainage tunnels

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Article

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Abstract
The destabilizing effect of groundwater is one of the major causes for landslides, which represent a major hazard to human life and the environment. Groundwater lowering is often the most efficient way to stabilize large unstable ground masses. Among groundwater lowering measures, drainage tunnels have several advantages, although construction costs may be proportionally high. This paper presents the concepts involved in the design and construction of drainage tunnels. Three case histories, two in Brazil and one in Argelia are presented, including geological background and monitoring results, where large landslides were stabilized using deep drainage through tunneling solutions.

1. Introduction

Slope failures have been a major hazard to human life and the environment, with a recorded average of around 4700 fatalities per year from 2004 to 2016 (Froude & Petley, 2018) and 3270 fatalities in 2019 (Petley, 2020), excluding seismic triggered slope failures. For this reason, slope stabilization works in urban environment and along transportation routes is, and has been, a main issue in geotechnical engineering.

Traditional slope stabilization measures include changes in slope geometry, construction of active or passive retaining structures and groundwater lowering measures used by itself or in different combinations.

Undoubtedly, the destabilizing effect of water plays a major role in triggering landslides and its control is one of the most effective tools for stabilization.

This paper is structured as follows:

- In item 2 the concepts of destabilizing effects of the groundwater are discussed.
- Item 3 presents the concepts of slope stabilization using drainage tunnels.
- Item 4 presents the importance of geology and a representative geological-geomechanical model on locating the drainage tunnels.
- Item 5 discusses briefly safety approaches for slopes.
- Item 6 presents important drainage tunnel design and construction issues.
- Item 7 presents 3 case histories, where drainage tunnels were used to stabilize slope failures that were affecting important infrastructure projects.
- Conclusions are presented in Item 8.

2. Effects of groundwater on slope stability

In Brazil, slope failures are concentrated in the rainy seasons, where superficial water infiltration and rise of groundwater table generate destabilizing forces and cause different types of slope failures. Similar conditions are encountered around the globe: Popescu (2002) and Highland & Brobowsky (2008), for example, describe water, seismic and volcanic activities as major triggering mechanisms of landslides. This paper focuses on failures caused by water, specifically, rising groundwater, one of the main causes of the destabilization of large soil masses. It implicitly considers that the soils involved, mainly colluvial deposits, residual soils and/or saprolites, are not very brittle, going through a large and sudden loss of their shear strength with very small displacements. In this specific failure mode, large excess pore pressures can be generated faster than their dissipation, and the soil mass can go into a flow type of landslide; additional analysis have to be performed in this case to define the stabilization concept to be used.

A simple way to show the effect of groundwater on slope stability is the so-called infinite slope model, were the soil-rock interface is considered impermeable and the flow lines are parallel to the surface.

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Considering the model presented in Fig. 1, with a clearly defined failure plane at the soil-rock interface, equilibrium can be evaluated by a *FoS* (Factor of Safety), defined as the ratio between the available shear strength and the acting shear stress. Groundwater reduces safety, because it reduces the effective stress along the failure plane, while the acting shear stress is almost not affected. The more frictional the soil is, the more safety is affected by the groundwater:

$$\sigma_z = \rho \times g \times z \times \cos\beta \tag{1}$$

$$\sigma_n = \rho \times g \times z \times \cos^2\beta \tag{2}$$

$$u = \rho_w \times g \times h \times \cos^2\beta \tag{3}$$

$$\tau_{app} = \rho \times g \times z \times \cos\beta \sin\beta \tag{4}$$

$$\tau_s = c' + (\sigma_n - u)\tan\varphi \tag{5}$$

$$FoS = \frac{\tau_s}{\tau_{app}} \tag{6}$$

where: σ_z = vertical stress; σ_n = normal stress; ρ = specific gravity of the soil; ρ_w = specific gravity of water; β = slope declivity; z = thickness of soil layer; h = water head measured from the soil rock interface; φ = friction angle of the soil; c' = cohesion of the soil.

For soils with no or negligible cohesion, the *FoS* can be written as:

$$FoS = \frac{\tan\varphi}{\tan\beta}\left(1 - \frac{\rho_w}{\rho}\frac{h}{z}\right) \tag{7}$$

For a dry slope ($h = 0$), the *FoS* becomes:

$$FoS = \frac{\tan\varphi}{\tan\beta} \tag{8}$$

If the soil has a specific gravity of around 20 kN/m^3, the *FoS* of the saturated slope is approximately 0.5 of the *FoS* of the dry slope, showing the high impact of the groundwater on safety. The groundwater level at the surface means $h = z$ and the *FoS* can be approached as being:

$$FoS = \frac{1}{2}\frac{\tan\varphi}{\tan\beta} \tag{9}$$

Analogous results are obtained using limit equilibrium or continuum modeling. Intuitively and as shown by several authors (for example, Patton & Hendron, 1974; Borges & Lacerda, 1986; Bastos, 2006), flow conditions tend to generate even more critical conditions at the base of slopes, where water flows in the direction of the external surface and, additionally, the existence of less permeable soil covers, like talus deposits, may lead to the generation of increase of pore pressures.

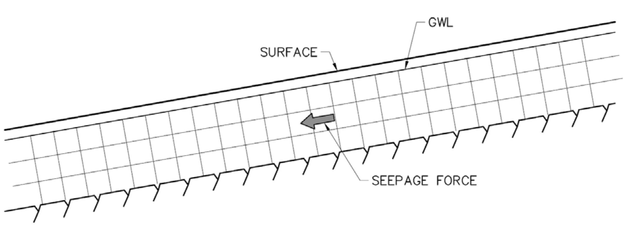

Figure 1. Simplified infinite slope model.

Popescu (2002) describes typical remedial measures for landslide stabilization:

- Modifications in slope geometry;
- Drainage;
- Retaining structures;
- Internal slope reinforcement.

Especially for large ground masses, the control of the groundwater is one of the most efficient ways to achieve stability. Typical means to reduce the destabilizing action of the groundwater are:

- Horizontal gravity drains;
- Deep pumping wells;
- Large diameter wells, possibly associated to horizontal gravity drains;
- Drainage tunnels.

Other ways to reduce the destabilizing action of water include well points, electro-osmosis and the use of vacuum to increase pumping wells efficiency, among others. However, these solutions are normally temporary and may not be feasible for large massifs; therefore, they will not be further discussed.

It is important to mention that these groundwater lowering measures have different efficiency, considering at least three aspects:

- Influence radius of individual elements: spacing of small diameter elements must be evaluated to guarantee efficient ground water lowering. The smaller the equivalent permeability, the smaller the influence radius of each individual element;
- Influence of the position of draining elements inside the unstable mass: drainage at the upper part of the unstable soil mass may look efficient to "cut" access of water, as an interceptor of the flow. But, depending on the problem, this location may not be efficient, due to complex water flow patterns. Drainage at the bottom, on the other hand, may be optimal from a constructive point of view, but not as efficient, because of reduced influence on pore pressures in the upper part of the unstable soil mass. In most of the published case histories, drainage, by different means (wells, horizontal drains, tunnels), is installed in different positions along the unstable soil mass, to achieve broader groundwater lowering;
- Influence on the direction of the destabilizing seepage forces:
    - Sub-horizontal drains tend to lower the groundwater, but seepage forces normally continue to act in the slope direction, *i.e.*, they continue, to a certain extent, contributing to destabilization (Fig. 2);

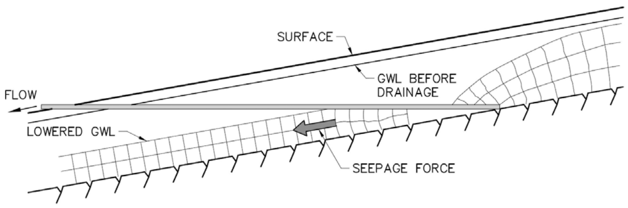

Figure 2. Schematic view of flow model for groundwater lowering using gravity drained sub-horizontal drains. Seepage force acts in the destabilizing direction.

- Deep wells when pumped often tend to invert seepage forces, *i.e.*, seepage forces may act as a stabilizing force, instead of destabilizing the soil mass (Fig. 3). The use of deep wells may be of interest to contribute as a temporary solution, helping to stabilize the soil massif while the long-term solution is implemented, as they have direct costs of electricity supply, require maintenance, backup pumps, etc.;
- Drainage tunnels may influence stability in different ways. The main goal is to obtain generalized groundwater lowering. The way the groundwater is lowered depends on the tunnel location (length, position with relation to the geological singularities), number, length and position of drains installed from the tunnel, position, among others. If vertical flow is achieved (Fig. 4), the destabilizing effect of water is practically eliminated.

In the case of vertical flow, the groundwater level cannot be interpreted as one of the boundaries of a “flow-channel”: vertical flow is gravitational, *i.e.*, the vertical gradient i equals 1 and pore pressures are zero in all points of the flow net.

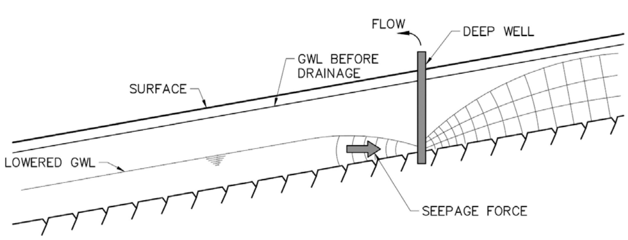

Figure 3. Schematic view of flow model for groundwater lowering using pumped deep wells. Seepage force acts partially as stabilizing force.

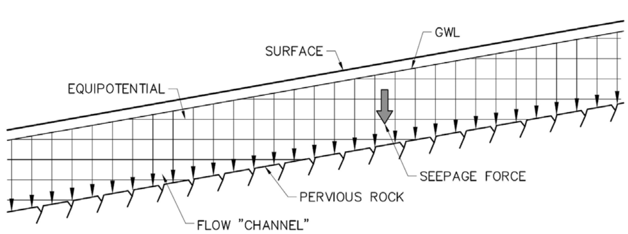

Figure 4. Simplified flow model for vertical flow, considering pervious rock.

3. Drainage tunnels

The broad concept of drainage through tunnels is to excavate them at depth, beneath the failure surface, in competent and stable material, serving as access for implementation of radial drainage, mainly upwards but also incorporating knowledge from the geological model to optimize the location of the drains. Main advantages of this drainage solution are:

- Groundwater lowering through predominantly vertical flow, conceptually eliminating / reducing significantly destabilizing seepage forces;
- Use of gravity flow, eliminating the necessity of energy supply and long-term maintenance of pumping systems;
- No need of any activity, including construction, inside the unstable soil mass;
- No impact on the surface and risk of damage to the stabilizing system through vandalism.

Stabilization of unstable or failed soil masses through drainage tunnels has been used in several locations around the world, with case histories presented, from different locations, in Europe (for example, Bardanis & Cavounidis, 2016; Bertola & Beatrizzotti, 1997; Eberhardt *et al.*, 2007; Marinos & Hoek, 2006; Futai *et al.*, 2009), in the Americas (Rico & Castillo, 1974; Vargas, 1966; Wolle *et al.*, 2004; Yassuda, 1988), Asia (JLS, 2002; Lin *et al.*, 2016; Sun *et al.*, 2009; Wang *et al.*, 2013; Wei *et al.*, 2019; Yan *et al.*, 2019), and Oceania (Gillon & Saul, 1996).

The use of drainage tunnels to reduce pore pressures is not restricted to the stabilization of landslides. Drainage galleries are often designed and built to reduce pore pressures and increase safety in the foundations and abutments of dams (de Mello, 2018).

Figure 5 presents an example of a typical drainage tunnel (Eberhardt *et al.*, 2007), installed inside the rock mass, below the landslide, with drains upwards drilled into the unstable soil, to reduce pore pressures:

The first published use of drainage tunnels to stabilize landslides in Brazil is described by Vargas (1966) and Guidicini & Nieble (1976). These authors describe a landslide that mobilized around 500.000 m^3 of material, triggered by a cut, built for the construction of the powerhouse

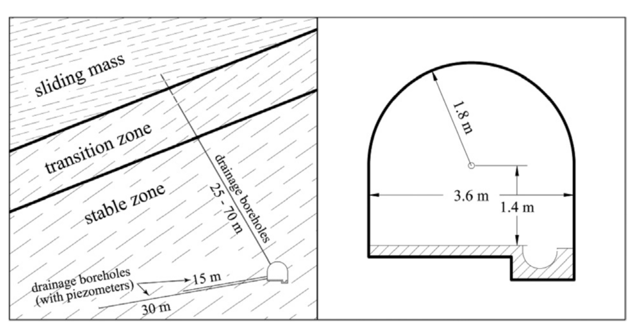

Figure 5. Typical cross section of drainage tunnel (Eberhardt *et al.*, 2007).

of Henry Borden hydro scheme in the foothill of Serra do Mar, Cubatão, Brazil in the past (around 1947). Following Terzaghi's (who acted as consultant) recommendations, drainage tunnels were excavated in the unstable soil mass and drains were drilled from inside the tunnels aiming at a specific geologic feature, said to be a quartzitic permeable high dip stratum (de Mello) that itself had a very broad influence in the slope. Drainage stabilized the soil mass completely, with a groundwater lowering of only around 3 m. Figure 6, reproduced from Vargas (1966) shows a plan view and cross section of the landslide. Figure 7 presents data published by Vargas (1966), showing the effectiveness of the groundwater lowering solution.

An important issue associated to drainage tunnels is its location under and outside the unstable soil mass. To optimize construction costs, the excavated tunnel length should be minimized. An adequate access must be found under unstable groundmass, where the tunnel portal can be located allowing gravity discharge flow, and excavation can start safely, but also minimizing tunnel length. Figure 6 above shows that for the stabilization of the landslide in Cubatão, several tunnels (galleries) were excavated and drains were drilled from these galleries. Figure 8 below shows the drainage tunnel and its adits used to stabilize the VA-19 landslide, published by Wolle *et al.* (2004). A single access tunnel was excavated from an adequately located position at the surface, under the unstable ground mass; adits were built from the tunnel alignment to optimize drainage and tunnel length.

Drainage tunnels are normally excavated in stable ground, but the drains perforated and installed from them into the unstable soil mass are often not conventional drains. Potential problems associated to the drains can be divided into installation problems and maintenance problems.

Installation of the drains in the tropical environment is often associated to drilling initially through rock, weathered rock and the rock-soil interfaces, with all its associated difficulties (Bilfinger, 2019), into soil and an unstable ground mass, often including blocky and unconsolidated

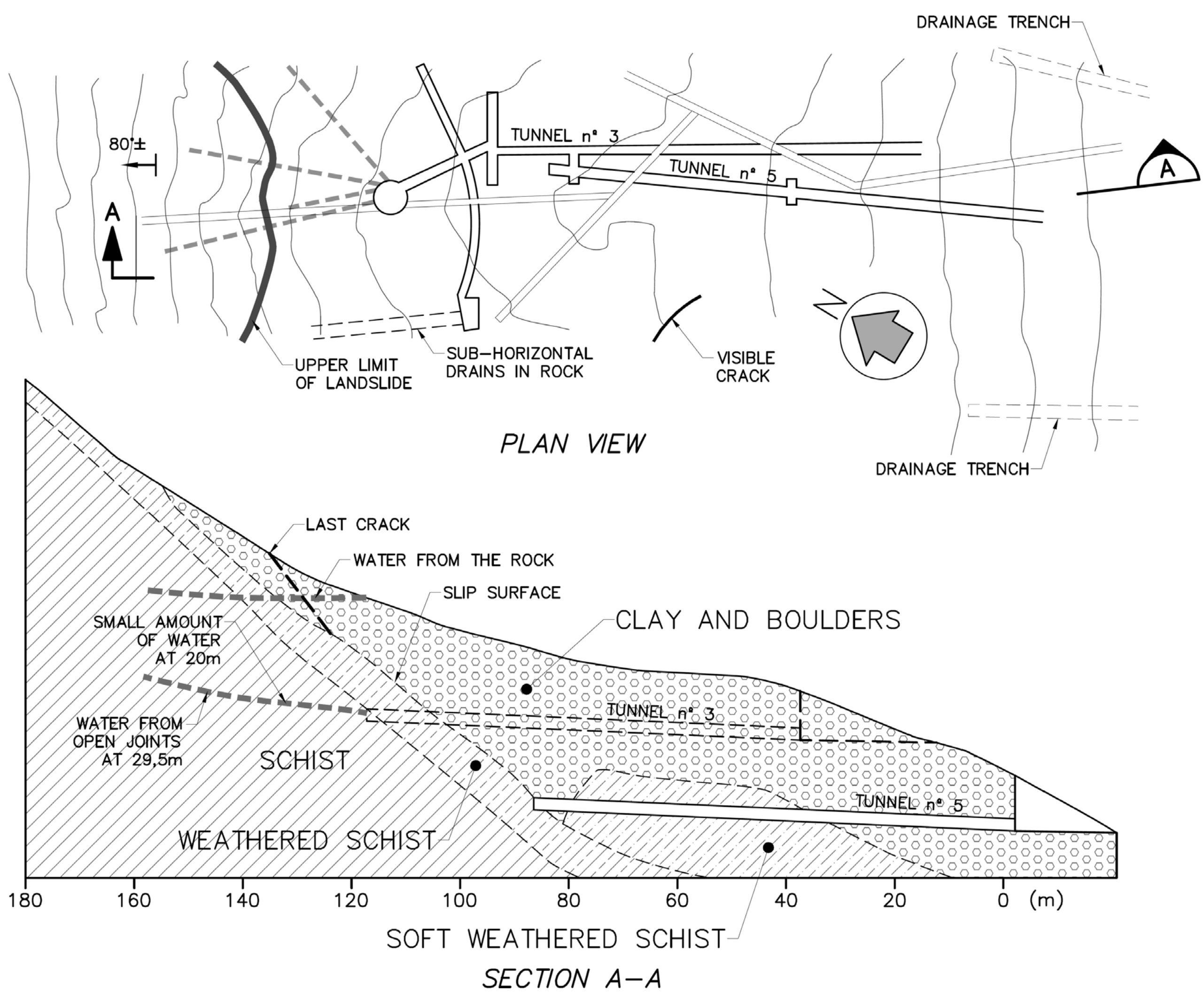

Figure 6. Landslide stabilized using drainage tunnels in Cubatão - Brazil (from Vargas, 1966).

material, with high groundwater level. Concentrated flow many times exists in the saprolite-weathered rock interface.

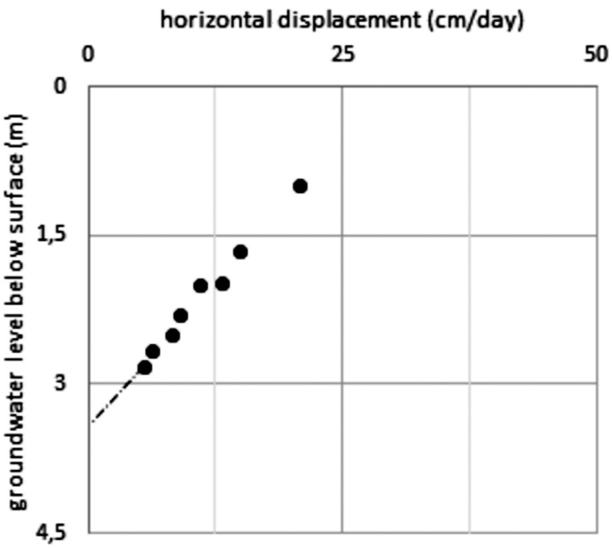

Figure 7. Ground mass movement as a function of the groundwater lowering for the stabilized landslide in Cubatão (from Vargas, 1966).

Sometimes, conventional drilling is not possible or is associated to risk of fines being washed through the drain or through the annular space between the drilled hole and the drain. In intensively fractured rock, there are concerns that in the procedure to retrieve the drilling tool and install the perforated drain, fragments of rock fall into the drilled void and the perforated drain pipe cannot be installed. Self-boring drains are often a more efficient and safe way to install drains in these situations.

Maintenance problems can be divided into short-term and long-term problems. Short-term problems are normally associated to a still non-stabilized ground mass, that may damage or shear the drains, reducing efficiency or even destroying them, sometimes releasing the water collected in the displacing massif in the shear zone. In the long-term, drains may be clogged with time by, for example, oxidation, which also reduces their efficiency. This problem is particularly relevant in tropical soils, with high iron oxide content.

Table 1 presents the main critical issues associated to the drains installed from drainage tunnels.

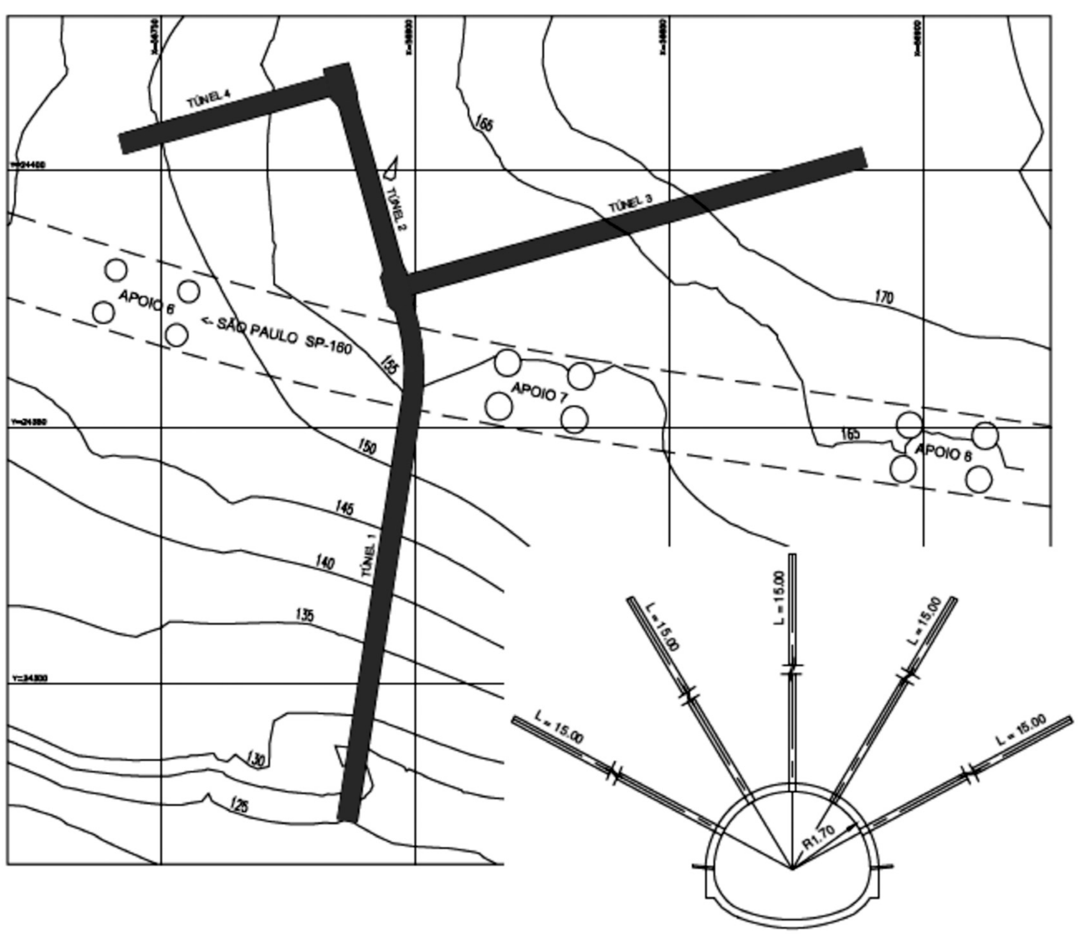

Figure 8. Plan view and tunnel cross section of the VA-19 drainage tunnel (Wolle *et al.*, 2004).

Table 1. Critical issues associated to drains installed from drainage tunnels.

Phase	Potential problem	Solution
Installation	Drilling through different materials (rock, blocky material, soil)	Cased drilling, selfboring drains
	High water table	Preventer
Maintenance	Unstable soil mass leading to shearing of drains	Water flow control and re-installation of drains
	Clogging (oxidation, fines)	Water flow control, washing and re-installation of drains

4. Importance of geology and geological-geomechanical model

Knowledge of the geology of a site and a solid and consistent geological model play a fundamental role in any engineering project. Fookes *et al.* (2000) present an interesting approach, starting from simple models, that evolve with time and help to plan the different steps of the geological and geotechnical project steps. What is particularly interesting in Fookes *et al.* (2000) approach, is its initial phase, where, to characterize a realistic model, the project site geology is identified as being one or more of the typical predefined models:

- Global scale tectonic models, based on plate tectonics;
- Local or site scale initial geological models;
- Local or site scale initial geomorphological models which characterize landforms.

Based on these models, an initial framework of the sites geological conditions can be established, reducing the risk of encountering not foreseen conditions.

A typical development of a geological model, which could include the interesting initial steps proposed by Fookes *et al.* (2000), follows the steps below:

- Desk studies, including aerial photo interpretation, bibliographic research, etc.;
- Walkover;
- Ground investigation;
- Supplementary investigation;
- Finally, during construction the model is updated with data from the site. In the case of drainage tunnel, the geological mapping of the excavation faces may be useful to optimize drain location or even for adjustments of tunnel alignment.

In the case of landslides, a geological-geomechanical model must be complemented by information about the extent and depth of the mobilized ground mass.

A slightly different site investigation approach, directly associated to the development of the geological model, is presented for tunnels by ITA (2015), but can be generalized:

- Feasibility studies, detailed in Table 2;
- Preliminary design, detailed in Table 3;
- Detailed design, detailed in Table 4.

Independently of the references, there is consensus that the development of an adequate geological model can be divided into 3 or 4 phases, initiating with desktop studies, followed by walkovers by experienced geologist(s). After these initial phases, investigation and studies include different types of geological-geomechanical investigations (boring, geophysical evaluations) in one or more phases.

When a landslide is being investigated, these steps are normally complemented by monitoring results, including:

- Surficial displacements, using conventional topography or more recently developed remote sensing techniques; (Zhao & Lu, 2018; Mantovani *et al.*, 2019);
- Displacements inside the ground mass, mainly through inclinometers;
- Pore-pressures, using piezometers - different types are available, for different conditions and soil types. In the case of failures in rock masses, the measurement of pore pressures is often more complicated, because normally water flows through discontinuities and pressures act on

Table 2. Site Investigations for feasibility studies, based on ITA recommendations (ITA, 2015).

Expected results	Investigation means
Geological and hydrogeological maps	Regional topographic, geological, hydrogeological / groundwater, seismic hazard map
Natural risk maps, when appropriate	Information from field surveys and/or adjacent similar projects
Longitudinal geological profile	Geophysics may provide useful information
Longitudinal geotechnical and geomechanical profile and identification of major hazards	Limited site investigations to confirm extremely critical geological or groundwater conditions
Preparation of risk register	

Table 3. Site Investigations for preliminary design, based on ITA recommendations (ITA, 2015).

Expected results	Investigation means
Longitudinal geological profile (1:5000 to 1:2000)	Geophysics and boreholes at portals and shafts
Longitudinal geotechnical-geomechanical profile (1:5000 to 1:2000) with ground behavior classes	Boreholes along the alignment
Geological and geotechnical cross sections at the portals (1:500 to 1:200)	Water sources and groundwater monitoring
Geological and geotechnical cross sections at access and ventilation shafts	Laboratory tests
Preliminary characterization of the hydrogeological regime	Outcrop and surface mapping
Update of risk register	*In situ* measurements and permeability tests, when appropriate
	Exploratory galleries / shafts, if needed

Table 4. Site Investigations for detailed design, based on ITA recommendations (ITA, 2015).

Expected results	Investigation means
Longitudinal geological profile (1:2000 to 1:1000)	Additional boreholes at portals and along alignment
Longitudinal geotechnical-geomechanical profile (1:2000 to 1:1000) with ground behavior classes	Laboratory and field tests
Geological and geotechnical cross sections at the portals and shafts (1:200 to 1:100)	In specific cases / locations, geophysics may provide useful information
Definition of detailed set of design parameters and their variability	Excavation of experimental sections along tunnel alignment, if needed
Detailed characterization of the hydrogeological regime	Continue the monitoring of water sources and groundwater
Update of risk register	

discontinuities. To measure the correct pore pressures, piezometers must be positioned adequately inside the water bearing discontinuities. Previously performed special Lugeon tests, manipulating the packers to properly determine the water bearing discontinuities by pinching in until they are located, will make this possible, reducing the risk of non-representative pore pressure measurements.

In the case of drainage tunnels to stabilize large unstable soil masses, geology must focus on some aspects, which may not be all that relevant in other types of projects or situations. A detailed and adequate geological-hydrogeological-geomechanical model of the unstable soil mass has to be conceived, characterizing geo-materials, flow patterns and pore-pressures. Additionally, the stable ground under the landslide, as well as regions outside the unstable soil mass, where an access tunnel is to be built, must also be characterized, because it is in these locations and materials that the tunnel will be built.

Table 5 below presents important aspects that should be part of the geological-geomechanical model.

Table 5. Important aspects associated to geological-geomechanical models for drainage tunnels.

	Important aspects
Landslide	Focus on longitudinal cross sections
	Main soil and rock layers inside and immediately under the unstable mass
	Geomechanical characterization of each layer
	Geohydrological model
	Position of “slip surface(s)” / shear zones
	Tridimensional landslide model for optimal tunnel location
Tunnel	Evaluation of access tunnel location (outside landslide area)
	Soil and rock layers under the unstable mass and their geomechanical properties
	Permeability of material and possible naturally draining features

5. Safety concepts

Slope stability is conventionally evaluated using the static limit equilibrium *FoS* approach, where available shear strength is compared to acting shear stress.

For conventional conditions, a design *FoS* is normally defined between 1.3 and 1.5. These *FoS* are compatible with the Brazilian slope stability standard, NBR 11682 (ABNT, 2009), where the minimum *FoS* of the “safety level” is related to human life and material and environmental losses. Table 6 presents the *FoS* proposed in the Brazilian Standard:

A *FoS* includes 3 types of uncertainties (Hachich, 1996):

- Intrinsic: the natural or fundamental uncertainty;
- Statistical: uncertainty associated to the parameters of the assumed model;
- Model: uncertainties associated to the model assumed to be representative of the phenomena.

When dealing with an unstable soil mass, the *FoS* can be considered as being around 1.0 prior to any intervention and some uncertainties tend to be nonexistent. Therefore, when stabilizing an unstable ground mass, the conventional approach of designing for a conventional *FoS* would be overconservative. Conceptually, a small increase in the *FoS* would be sufficient to maintain the ground mass stable. However, the limitation of the *FoS* approach should not be forgotten: in slope engineering, a *FoS* approach is associated normally to limit equilibrium calculations, whose use is often questionable, especially for large ground masses. Limit equilibrium analyses compare available shear strength with mobilized shear stresses and, theoretically, if the available shear strength is higher than the mobilized shear stress, the ground mass is stable. Ground behavior is far more complex: *FoS* close to 1are normally associated to creep and possibly even to progressive failure.

Therefore, the authors consider adequate that stabilizing measures could be dimensioned for an increase of *FoS* of 25 to 30 %, which would be seen as lower than conventional approaches, but sufficient to obtain a stable condition.

6. Design and construction

The design of drainage tunnels can be divided in two parts:

- The tunnel itself, to be built safely and economically.
- The tunnel as part of a drainage system.

The description of the tunnel design itself is not the scope of this paper. Several approaches and methods are available in the literature, like publications from ITA (1988, 2000, 2009, 2019), BTS (2004), NGI (2015) and several others.

The design of the tunnel as part of a drainage system should focus on the following aspects.

Table 6. *FoS* proposed in the Brazilian Standard (NBR 11682/2009).

Safety level against material and environmental damage	Safety level against human life		
	High	Intermediate	Low
High	1.5	1.5	1.4
Intermediate	1.5	1.4	1.3
Low	1.4	1.3	1.2

6.1 Tunnel cross section

The tunnel cross section should be minimized, sufficiently to allow drilling from inside to install drains and tunnel excavation itself. Knowledge of dimensions of available drilling equipment is fundamental, to optimize tunnel cross section and consequent construction costs. Variable cross section may be an alternative: access to the areas where the drains will be installed may have a smaller cross section than the regions where drilling is foreseen. A minimum tunnel diameter to allow manual drilling, in the experience of the authors, is around 2 m, although a case of 1.5 m high × 1.0 m width galleries is described by Moraes & Assis (2017). Table 7 below presents approximate equivalent tunnel diameters and corresponding constructive methods.

Tunnel length, according to available published data, is not significantly affected by tunnel dimensions: relatively small equivalent diameter tunnels (around 2 m) have been excavated with total tunnel length of more than 1 km. However, some aspects should be considered when defining the tunnel cross section:

- Construction time, as a function of equipment - conventional tunneling equipment tend to have higher production rates and probably will allow faster construction than manual excavation;
- If complex geological-geomechanical conditions are foreseen requiring special equipment, the cross section needs to be sufficiently large to allow operation and movement inside the tunnel.
- Tunnel length: movement of equipment inside the tunnel may be very complicated in narrow tunnel cross sections. This may be compensated by building enlarged tunnel sections at every 100 to 200 m;
- Ventilation during construction in future maintenance;
- Utilities to be used during construction and future maintenance.

6.2 Tunnel location

Tunnel location has to be chosen to minimize tunnel length, optimizing construction and operational costs. However, some aspects have to be considered when defining tunnel location:

- Tunnel portal in a location:
  - where gravity drainage is possible;
  - safe, not influenced by the landslide;

Table 7. Equivalent tunnel diameters and corresponding constructive methods.

Equivalent tunnel diameter	Constructive method
2 m	Manual excavation
3.5 m	Small equipment
5 m	Conventional tunneling equipment

- Vertical and horizontal alignment to:
  - excavate material that minimizes excavation cost (lining type, lining thickness, ground treatments, construction time);
  - minimize drain (drilled from inside the tunnel) length;
  - optimize geohydrological position;
  - optimize position to maximize groundwater lowering effect on the unstable ground mass.

It is often necessary to excavate more than a single tunnel to achieve efficient groundwater lowering. Landslides extend often over hundreds of meters and a single tunnel may not generate a regional groundwater lowering. Figure 9 below presents a cross section of the Hilane landslide (JLS, 2002) and it can intuitively be seen that a single tunnel would be much less efficient, than the two tunnels built to stabilize the landslide.

The plan view of the tunnel built to stabilize the VA-19 landslide (Wolle *et al.*, 2004), presented in Fig. 8, shows also that a single tunnel would be much less effective than the system of adits and tunnels built.

Some interesting details may be important during design and construction:

- Use of self-drilling drains. Self-drilling drains have the advantage to transform drain installation into a single operation;
- In some cases, preliminary stabilization must be implemented before the radial drains are perforated and installed, as ongoing displacements could shear through recently installed drains until stabilization of displacements is achieved;
- Cost-benefit analysis of the drainage tunnel solution has to consider long term costs, associated to maintenance, which with a tunnel as access can be higher as initial cost, but are minimized in the long term.

Drainage tunnels have been built using conventional tunneling method (ITA, 2009). The main advantages of this constructive methodology are:

- Excavation does not need special equipment, like TBMs, and therefore, construction can be started quickly;
- Use of variable, non-circular, cross sections;
- Flexibility during excavations, changing and adjusting tunnel alignment as a function of geological and geomechanical conditions encountered during excavation.

However, mechanized excavation method also presents important advantages:

- Excavation under difficult conditions using EPB or Slurry technology, without the need of complex and costly soil conditioning;
- High excavation velocities;
- Fixed circular cross sections, that may be used for special remotely controlled drain drilling equipment.

At least in Brazil, so far, no mechanized drainage tunnel has been built.

7. Case histories

7.1 Case 1: Stabilization of viaduct VA-19 of the Imigrantes Highway, Brazil

This case history is described in detail by Wolle *et al.* (2004) and is considered by the authors a landmark in Brazil. For this reason, in this item a summarized version is presented.

7.1.1 Description of the Landslide

Imigrantes Highway was built in the 1970's connecting São Paulo to the closely located coast, including the harbor city of Santos. The highway crosses "Serra do Mar" mountains, from the São Paulo metropolitan area at approximately elevation 750 m, to the coastline, below elevation 10 m. It includes several tunnels and viaducts, with up to 90 m support towers.

VA-19, one of the viaducts, had been suffering anomalous openings of floor slab joints in a specific stretch since the 1980's. At the end of that decade, in 1988, comprehensive geotechnical monitoring started and deep-seated movements were identified, in weathered biotite gneiss at substantial depths. Average velocity was around 10 mm/year, with higher velocities during the rainy seasons. The oblique direction of the movements generated differential movements of translation and rotation, and consequent not foreseen stresses in the viaduct structure. Monitoring showed also that the deep foundations of the viaduct were being intercepted by the interpreted failure plane. When all these conditions were clearly identified, the decision to implement stabilizing works was taken. Some unsuccessful attempts were tried previously, including the use of Jet Grouting.

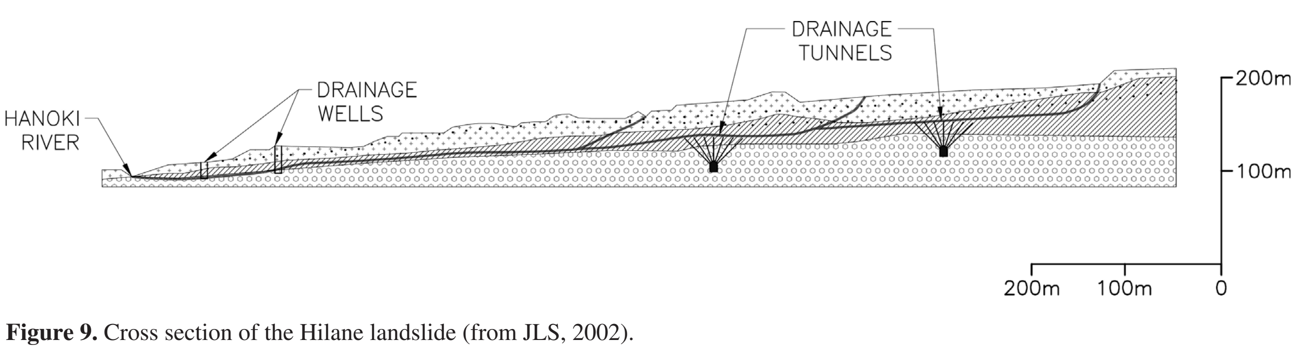

Figure 9. Cross section of the Hilane landslide (from JLS, 2002).

7.1.2 Geological model

Figure 10 presents a geological-geotechnical cross section through one of the viaducts supports. The figure includes a schematic representation of inclinometer readings, showing clear movements inside the residual soil. The residual soil - weathered rock was originated from foliated gneiss, with intercalations of quartzite and calcium silicate. The weathering profile allows a subdivision between highly weathered residual soils, with $N_{SPT} < 40$, overlaying a weathered layer (saprolite), with $N_{SPT} > 40$. At the basis, slightly weathered rock was encountered.

Groundwater level in the area was relatively high and increased even more during the rainy season.

7.1.3 Tunnel design

Several alternatives were evaluated, but, as frequently is the case with large landslides, the most effective way to improve stability was groundwater lowering. Stability analyses showed that a groundwater head reduction would be necessary to achieve an adequate safety increase. A drainage tunnel was chosen as solution. The location of the tunnel was optimized, taking into account geological and structural particularities of the ground mass.

Figure 11 and Fig. 12 present tunnel location and cross section. Total tunnel length is around 280 m and the cross section varied from 7 to 10 m^3/m, respectively in rock or soil.

Detailed geological mapping during the excavations was used to optimize drain locations: more fractured rocks or quartzitic veins concentrated water flow and the drains were concentrated, when possible, in these materials.

7.1.4 Monitoring results

Several instruments were installed during decades and, in part, lost due to vandalism or excessive horizontal displacements. This intensive monitoring led to the understanding of the mechanisms and, later, the control of the groundwater lowering measures results. Figure 13 presents the readings of one of the inclinometers, with readings between 1991 and 2002, including the tunnel construction period, that took place during the second semester of the year 2000. Accumulated horizontal displacements at approximately 30 m depth was 60 mm, but almost no further movement occurred during the next years.

Figure 14 presents water level readings of 3 piezometers, including a short period of about two years before tunnel construction, construction time and some three years of operation of the drainage tunnel. Significant groundwater lowering was measured, variable according to the position of the piezometers in respect to the tunnel location.

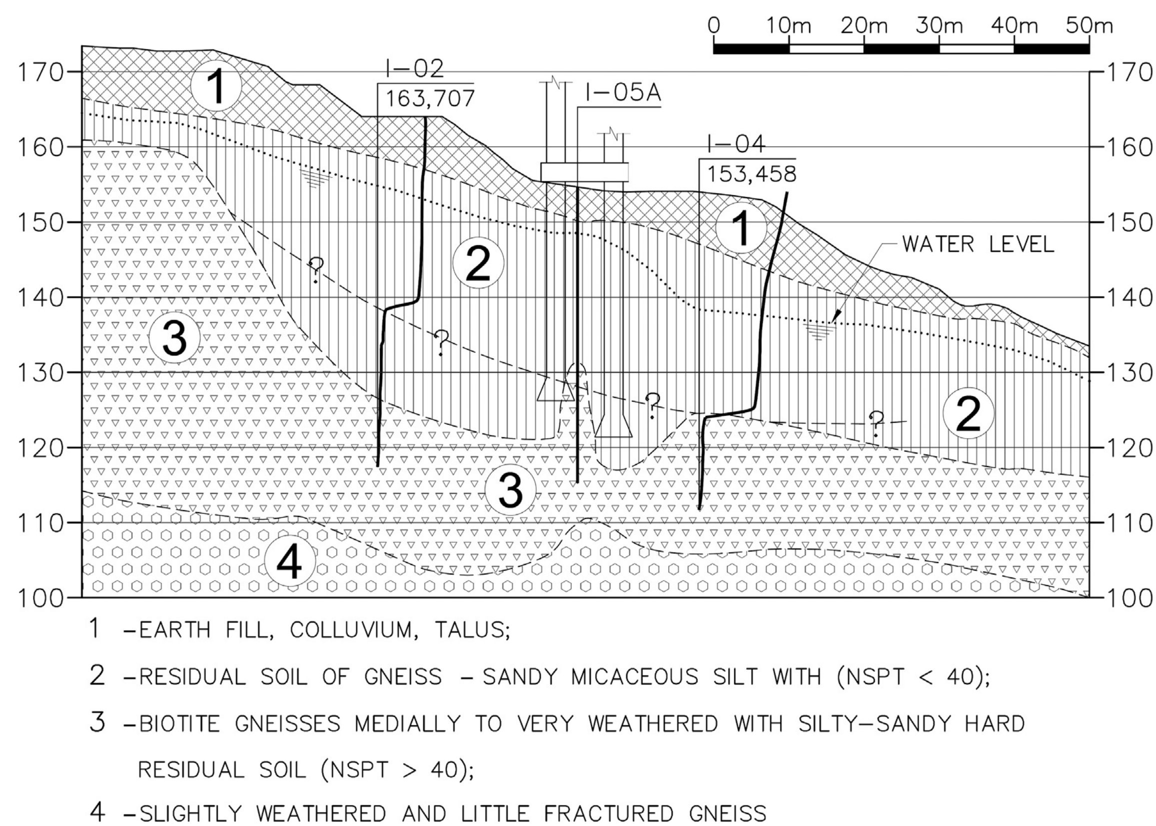

Figure 10. Transverse geological section with indication of shearing zones as detected by the inclinometers.

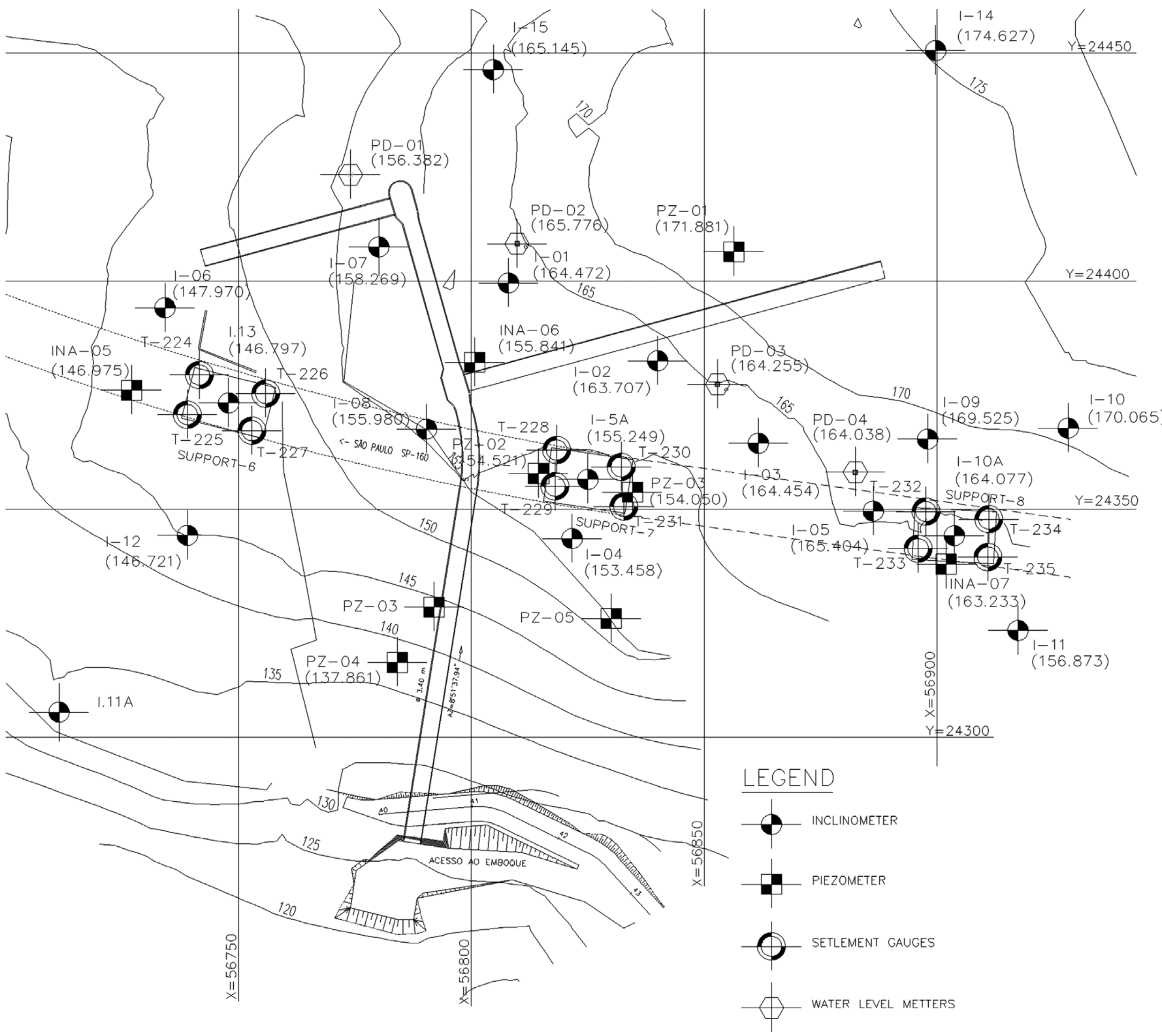

Figure 11. Plan view of tunnel, foundation of 3 pillars and monitoring instruments.

7.2 Case 2: Stabilization of km 376+400 of Candido Portinari Highway, Brazil

7.2.1 Description of the landslide

Candido Portinari Highway is in the state of São Paulo, Brazil, connecting Ribeirão Preto to Rifaina, close to the border with the state of Minas Gerais. It is a double lane highway with an average movement of around 12,800 vehicles per day (DER, 2020).

During the rainy season of 2006, at km 376+400 the north bound lanes suffered significant settlements and an emergency stabilizing berm was built, approximately at the toe of the existing embankment.

In early January 2007, during the rainy season, sudden vertical displacement of 2 to 3 m occurred, characterizing a geotechnical failure, with a mobilized ground mass of around 80.000 m^3. The failure was not limited to the existing embankment, but a significant part of its foundation was also involved in the unstable mass. Figure 15 shows a picture of the failure.

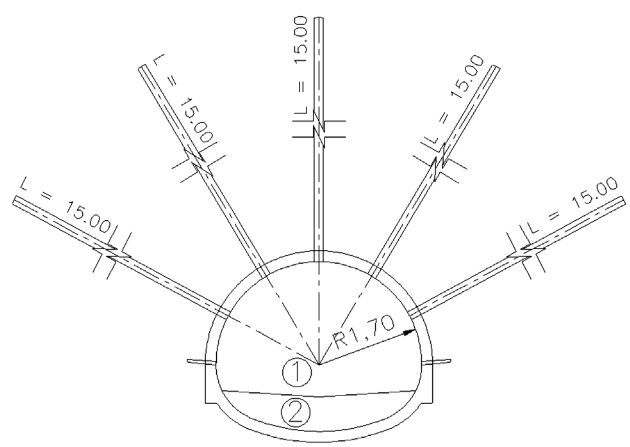

Figure 12. Tunnel cross section.

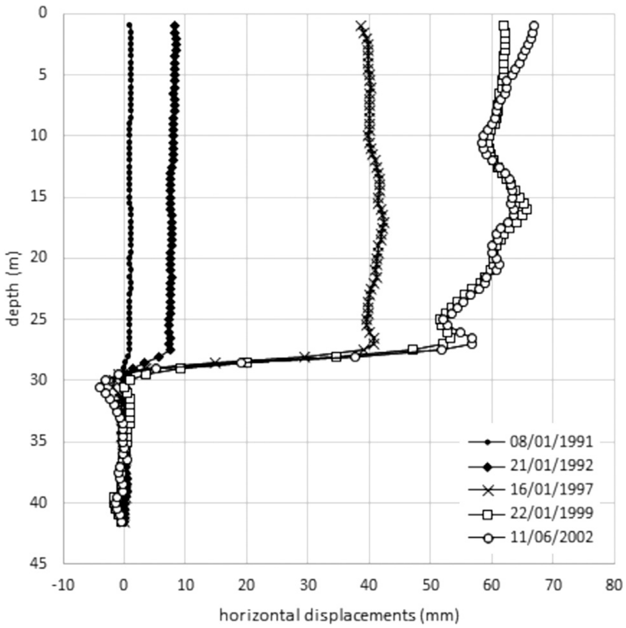

Figure 13. Inclinometer readings before and after tunnel construction.

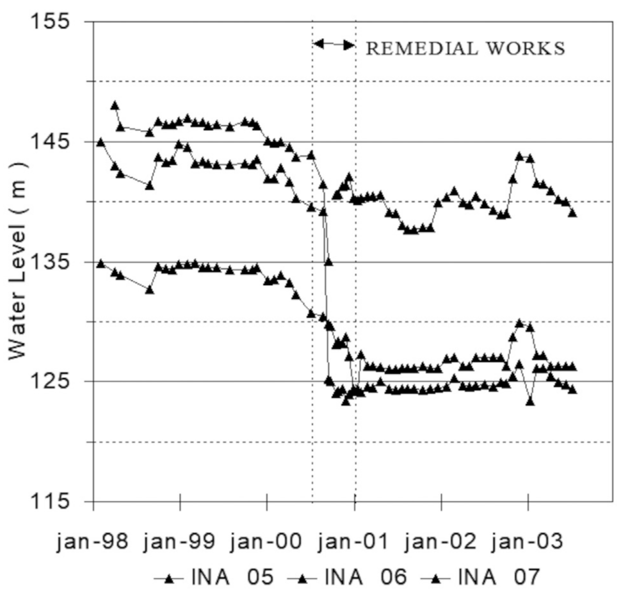

Figure 14. Piezometer readings before and after the construction of the drainage tunnel.

The average slope of the existing embankment was around 20°, and the average slope of the unstable area was around 13° to 15°.

7.2.2 Geological model

The region where the failure occurred is covered by reddish-purple soil ("terra roxa"), known in the past as excellent soil for coffee production. Geologically, the region is covered by basalts, and the products of its weathering, of

Figure 15. Geotechnical failure of the northbound lane of SP-334.

the Serra Geral Formation, over the sandstones of the Botucatu formation. Figure 16 below presents a geological cross section of the failed area.

The cross section shows relevant soil and rock layers, which explain the failure: the sound sandstone has low permeability and can be interpreted as an impermeable boundary. The fractured basalt, as shown by permeability tests, has high permeability. Borings showed also the presence of expansive clay minerals in part of the basalt fractures. The colluvial soils, as well as the road embankment, are mainly clayey soils, with low permeability. These low permeability layers generated a barrier, which led to a significant increase of pore pressure at the base of the unstable mass. The stabilizing berm built in the year prior to the main failure even increased the effectiveness of this water flow barrier. This pore pressure increase was interpreted as the main cause of the failure.

A few days after the failure, deep wells were installed between the highway lanes, immediately "upstream", temporarily stopping the movements of the failed ground mass.

7.2.3 Tunnel design

The drainage tunnel was designed to substitute the deep wells, which proved to be efficient to stabilize the unstable ground mass, in a permanent long-term stabilizing solution: the tunnel was excavated approximately 0.5 m from the bottom of the wells. After excavation, short horizontal drains were drilled from the tunnel to the wells, allowing drainage from the wells into the tunnel. 9 m long drains were also drilled into the fractured basalt, to improve the drainage of the fractured basalt. Figure 17 presents a schematic cross section of the solution.

Figure 18 presents a plan view of the tunnel location, with relation to the highway and the failed slope. Figure 19 presents a geological longitudinal section along the tunnel, including tunnel location.

The tunnel was built using steel corrugated plates as lining, with two different diameters. The initial stretch, that serves only as drain to allow gravitational flow, was built with a 1.2 m diameter. The stretch excavated close to the deep wells was excavated with a 2.2 m diameter, to allow the operation of a small drilling equipment, to perforate the holes and install the pipes between the tunnel and the wells and the drains into the soil massif.

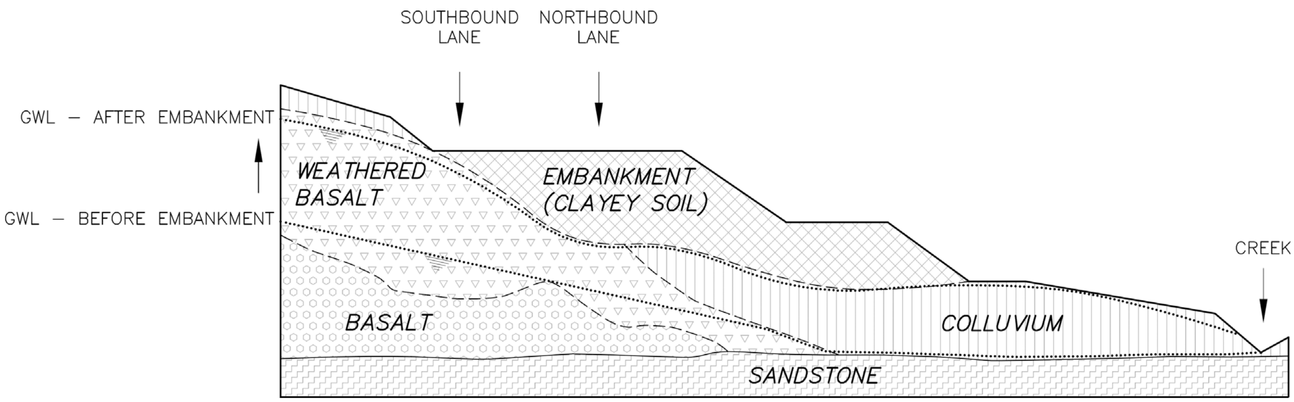

Figure 16. Cross section of the failure.

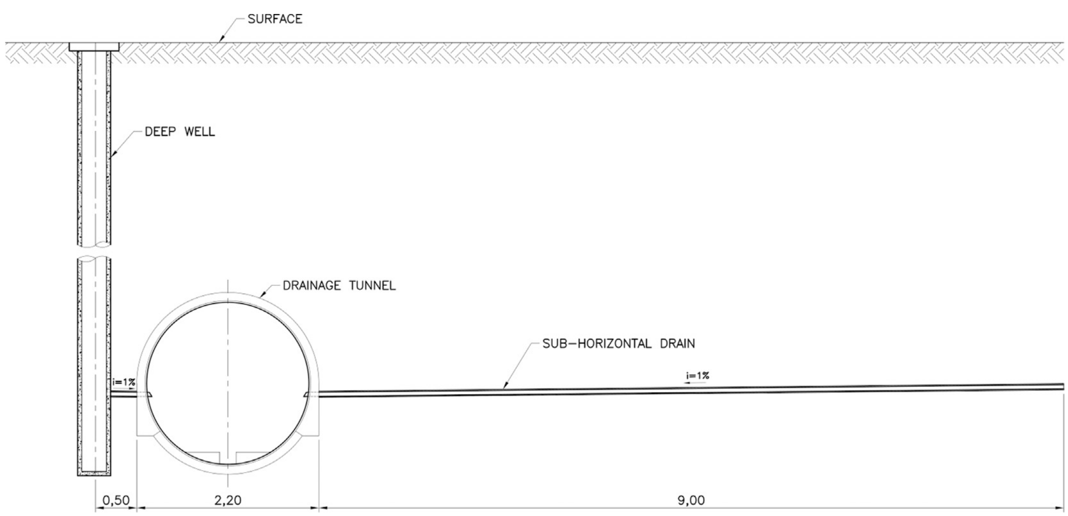

Figure 17. Schematic cross section of the tunnel and the deep wells.

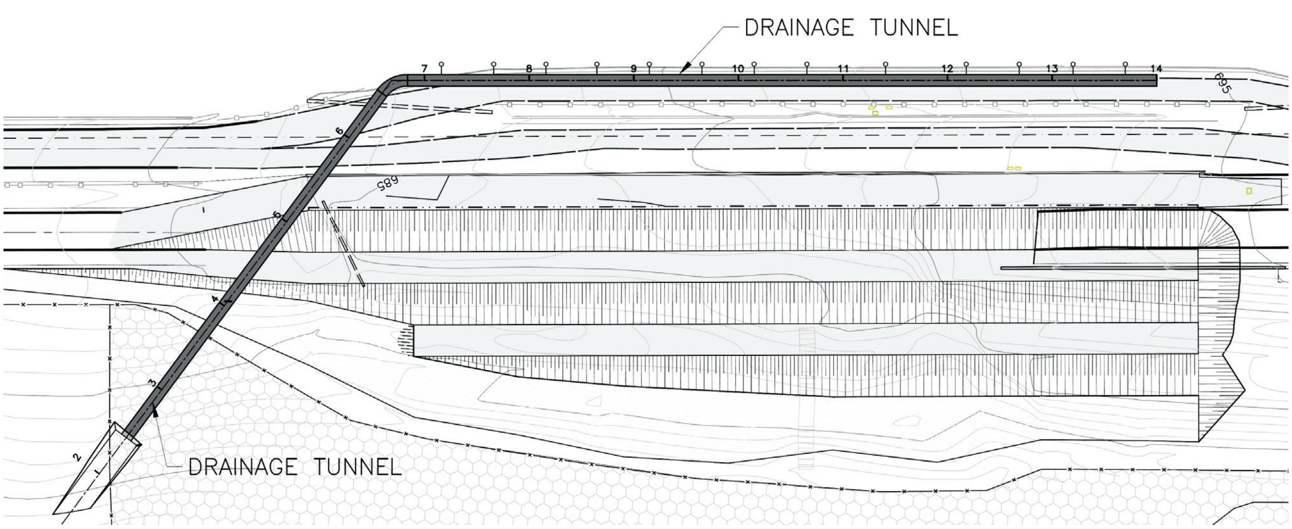

Figure 18. Plan view of the tunnel. The southern part of the tunnel was located in a position to allow gravitational drainage.

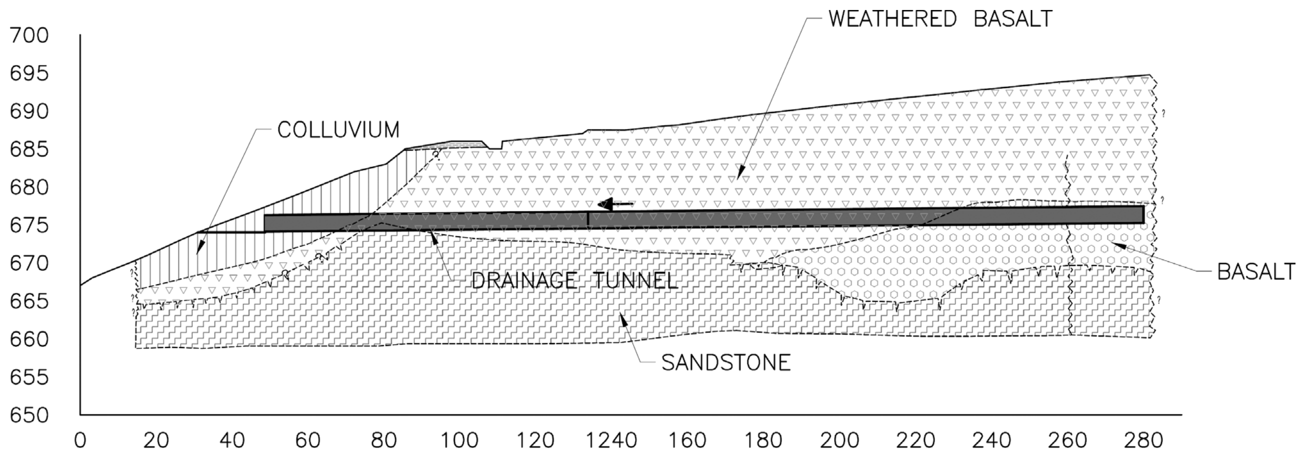

Figure 19. Longitudinal section along tunnel alignment.

7.2.4 Monitoring results

Unfortunately, displacement measurements, on the surface or inside the ground by inclinometers, were not made available until the deep wells were installed, and after their installation, movements ceased almost immediately.

The most relevant quantitative monitoring results is the information related to the groundwater level before and after the installation and operation of the deep wells. Figure 20 presents a longitudinal geological section, with highlighted position of pre and post pumping water levels.

After the groundwater lowering measures, the slope has been monitored until now and no significant movements have been registered.

7.3 Case 3: Stabilization of the Transrhumel Viaduct abutment in Constantine, Argelia

7.3.1 Description of the landslide

Constantine, in Algeria, north Africa, is known as the city of the Suspended Bridges. Founded in 300 b.C. it was reconstructed, renamed and chosen as the capital of the Roman Empire in North Africa by Constantino in 313 a.C.

Geological faults isolate the ancient town, facilitating its defense and imposing the need of bridges, at different heights since early times; technology and different cultures built impressive bridges of different materials, engineering concepts, and spans.

To commemorate the 21st century and to be ready for the election of the city as being the Arabic Culture Capital in 2015, a new cable stayed bridge was designed and built. The bridge is 756 m long, with a central span of 259 m and 60 m high pylons.

Constantine is also known for its active geological past, with many ancient landslides conditioning today's infrastructure.

A dormant ancient landslide previously known in the bridge's right abutment was remobilized by an earthquake linked to a particularly heavy rainy/snowy season when the bridge was near completion, about to close the main span. Emergency actions were taken to preserve the integrity of all foundations in the right abutment, as well as to design

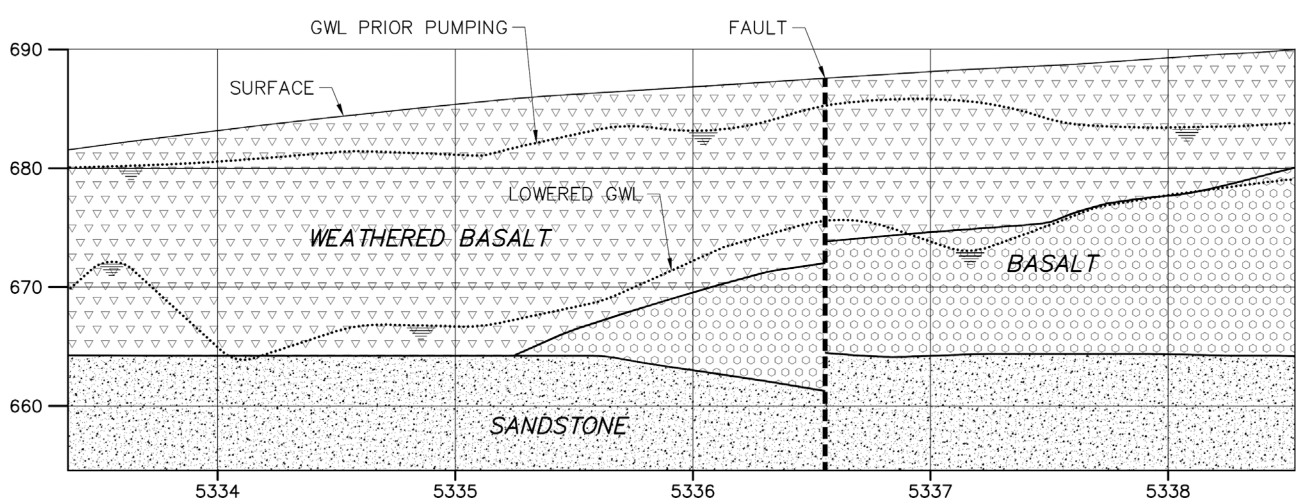

Figure 20. Longitudinal section with pre- and post-pumping water level.

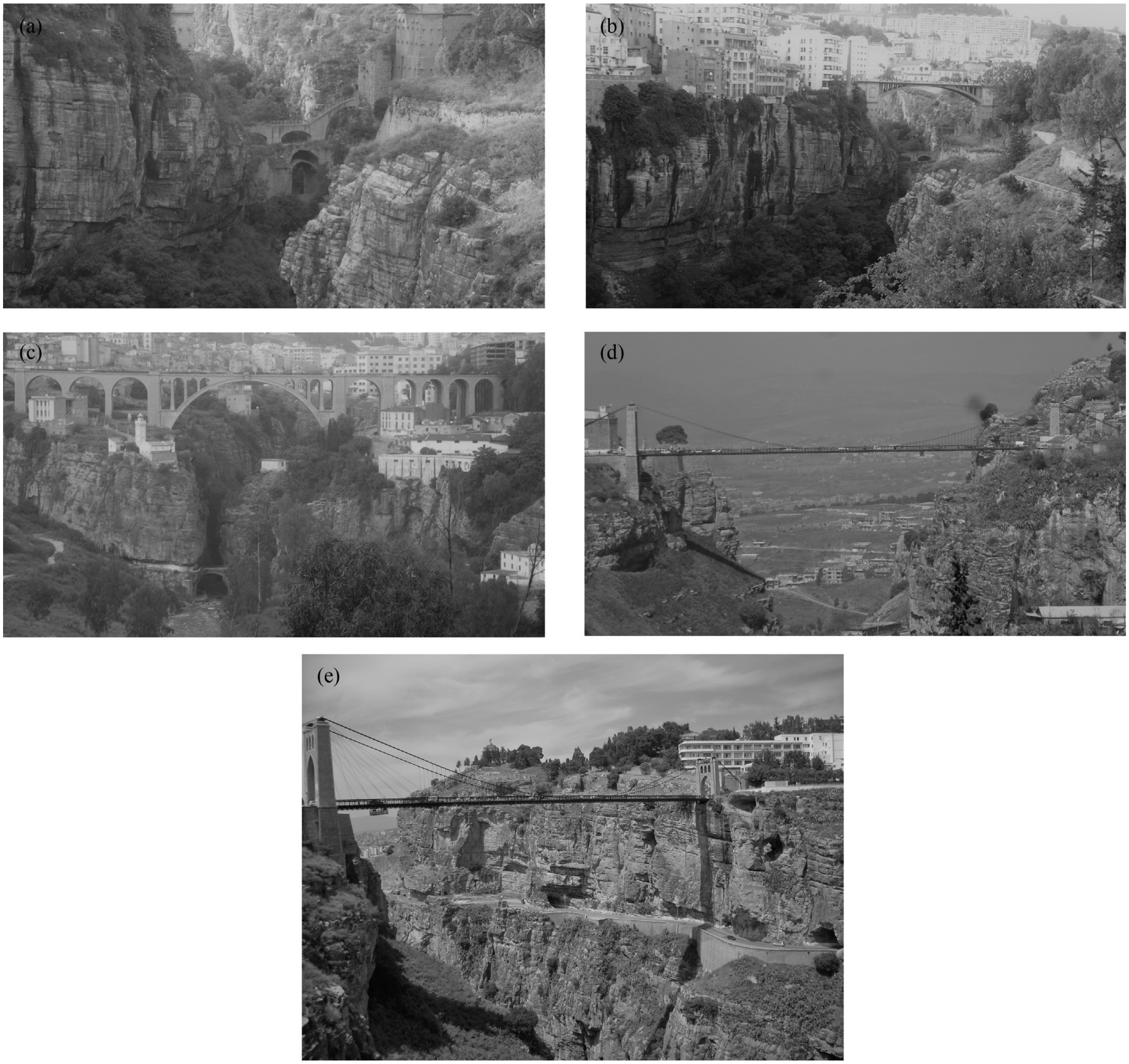

Figure 21. View of bridges built in different ages in Constantine.

and construct a definite solution to stabilize the whole slope. From the early discussions and considering the enormous mass involved, it was decided that the only solution was to lower the groundwater level through a tunnel.

7.3.2 Geological model

The Constantine region is located near the boundary between the African and Eurasian tectonic plates and the geological conditions of the region are complex with active seismicity prevalent in the area. The still ongoing collision introduces a compressional regime which is indicated by deformation of more recent Pliocene deposits. In the Constantine area outcrops range from Cretaceous, like marls and marlstones, to Quaternary deposits, like travertine conglomerates and top soils. The formation of the Rhumel river network and valley across which the Viaduct lies probably dates between 56 to 23 Ma.

The Constantine Viaduct is identified as being located in seismic zone IIa, with a range of peak ground acceleration from 1.6-2.4 m/s^2. Constantine has recorded three major earthquakes (Ms > 5) in the past century, in 1908, 1947, and 1984. The epicenters of all these earthquakes were located within 10 km of Constantine.

Many ancient landslides have been mapped in the Constantine region. The majority of the noted landslides are located in the region west of the Rhumel valley, while the landslide affecting the right abutment of the bridge is located in the east side of the valley.

Figure 22. View of the new bridge.

For the bridge foundations design an extensive site investigation campaign was pursued with many investigation boreholes as shown in Fig. 23, associated to geophysical methods and laboratory tests. The brittle and fragile characteristics of the marls and marlstones led to difficulties in retrieving quality samples for laboratory tests. The geological longitudinal profile along the bridge alignment shows superficial marls of different weathering degrees, followed by marlstones and limestones, as shown in Fig. 24.

The site investigation extended to the east, along the access road to the bridge, allowing knowledge of the extension of the stratigraphy.

Specifically, in the region comprising the stretch of bridge in its right abutment as well as the road system extending from it, numerous inclinometers, piezometers and

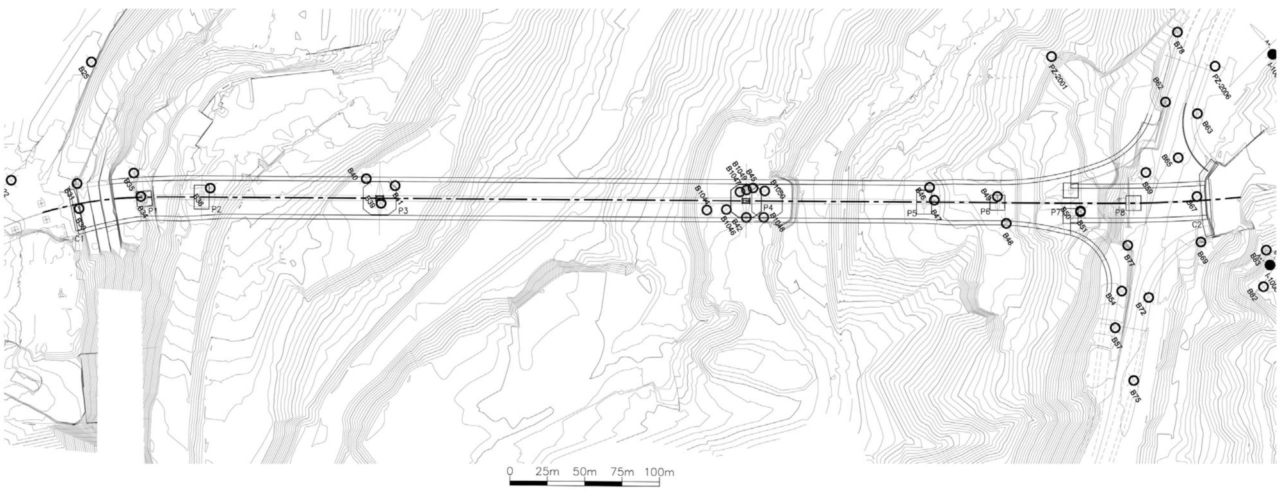

Figure 23. Location of Site Investigation boreholes.

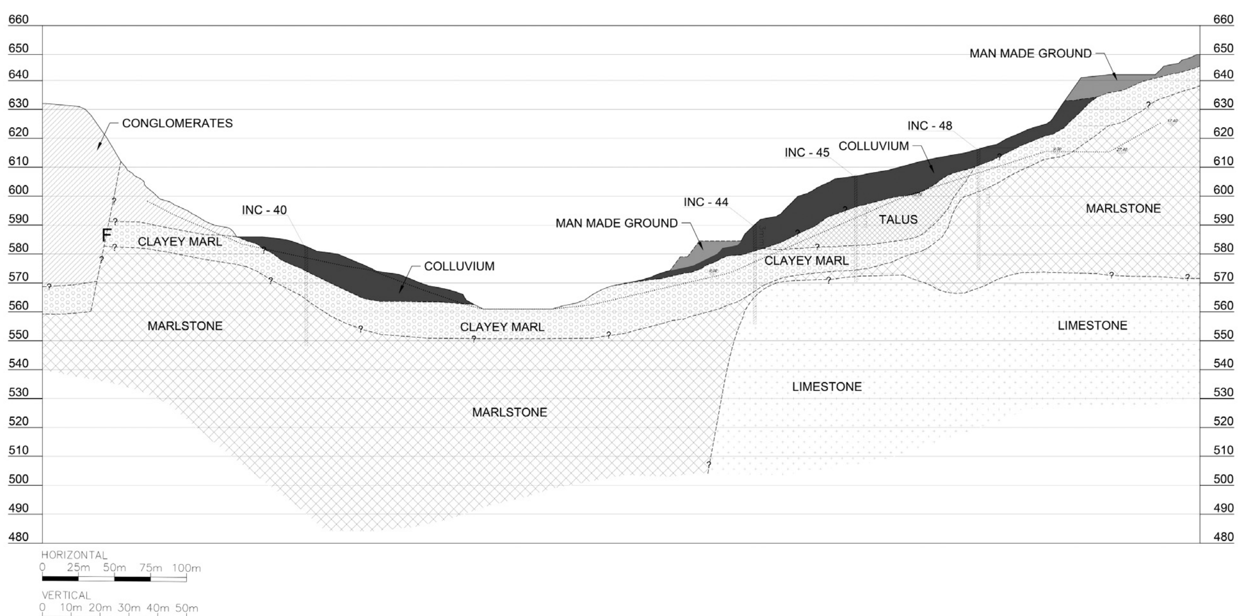

Figure 24. Geological profile along the alignment of the bridge.

drainage wells that also supply water level indication were installed. Figure 25 presents the location of the inclinometers, piezometers and wells/water level indicators used for interpretation of the slope behaviour. Deep wells and a displacement buffer were installed to temporarily stabilize the region and guarantee the structural integrity of the pylon's foundations; while the final solutions were conceived, designed and constructed, the bridge's pylons and pillars were preserved.

Inclinometer and piezometer data clearly show the existence of a slip surface at depth, and that a stable slope was reactivated by a sudden event in late January 2013, and enhanced during the spring of 2014 when melting snow generate substantial water infiltration.

The displacement pattern at different depths is seen as indicated in Fig. 26 and Fig. 27. Figure 28 shows the displacement direction as measured by several inclinometers and the effects at the ground surface is almost obvious, as Fig. 29 shows.

The Sidi Rached bridge, a masonry structure constructed in early 20th century, presented in Fig. 21c and in

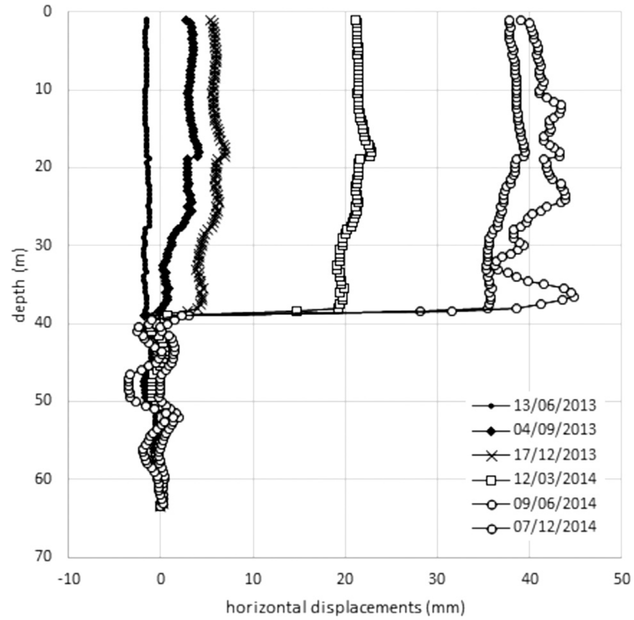

Figure 26. Typical inclinometer readings, showing clear development of slip surface at almost 40 m depth.

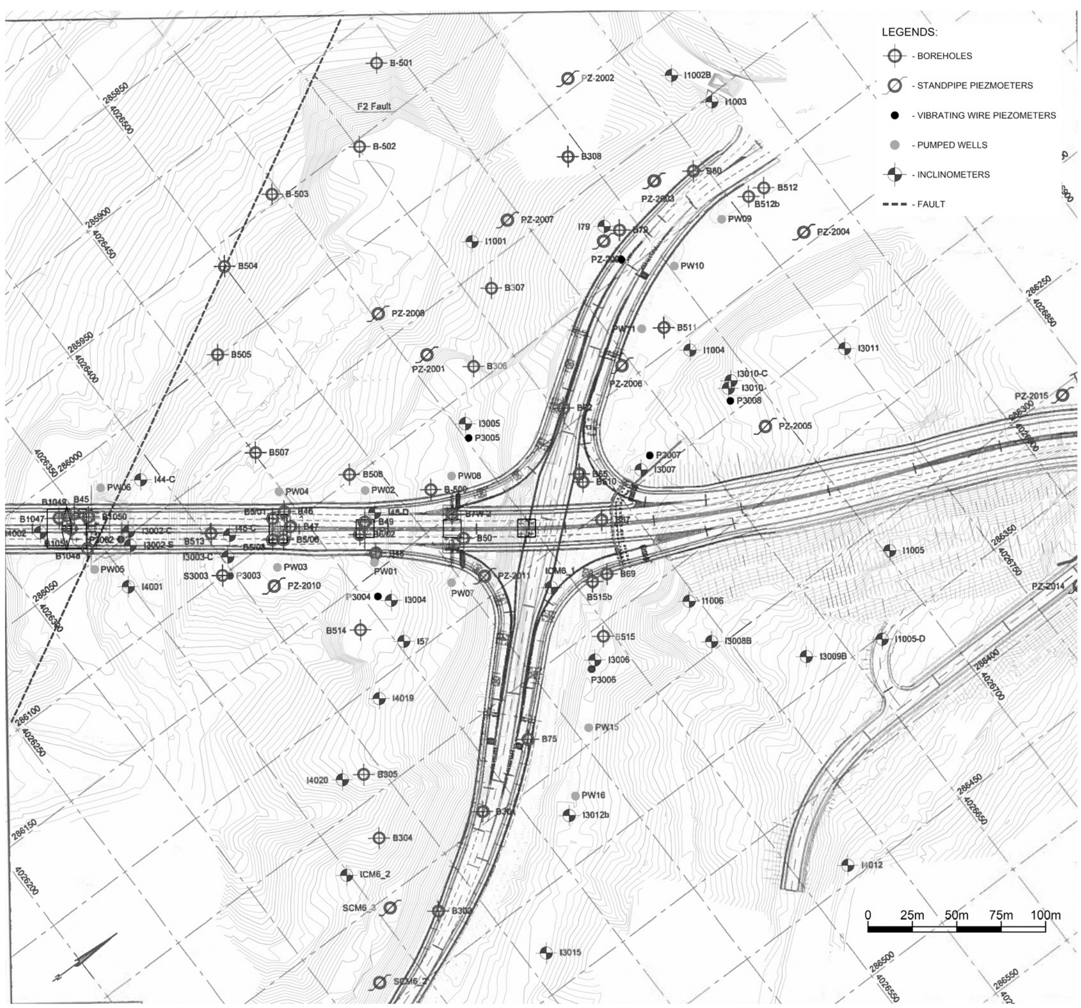

Figure 25. Location of boreholes, inclinometers, piezometers and deep wells. Data of highlighted inclinometer are presented in Fig. 26.

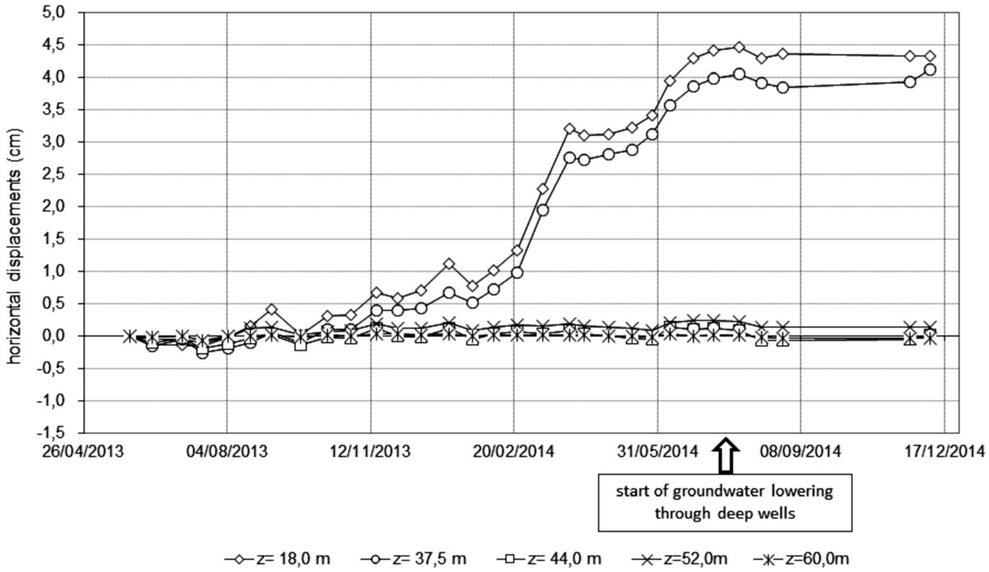

Figure 27. Shear displacements along depth for inclinometer of Fig. 26.

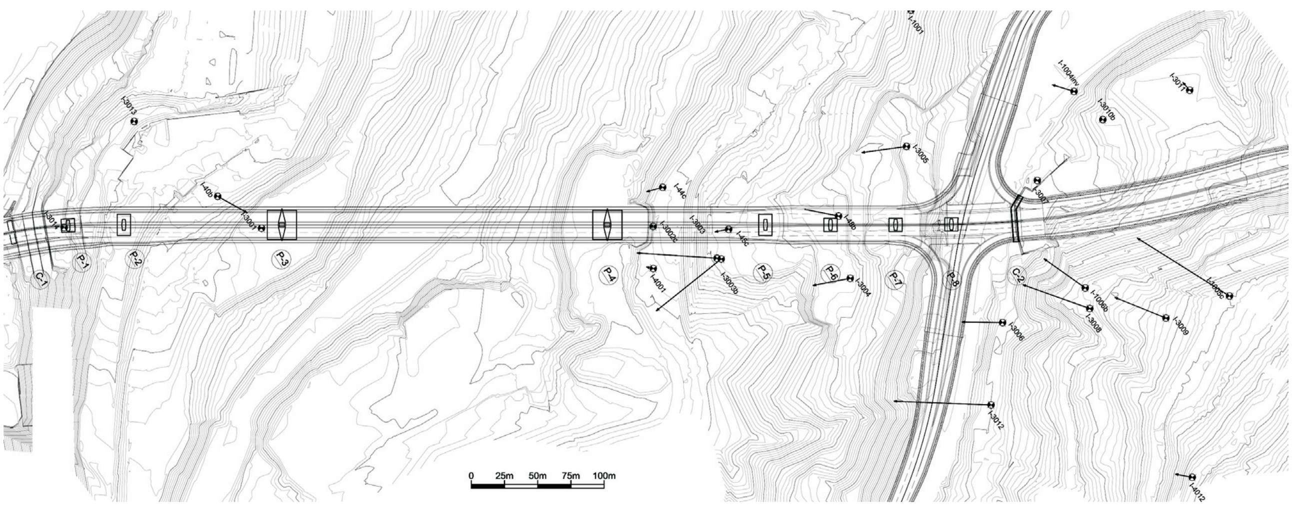

Figure 28. Displacement vectors as measured by inclinometers.

the background in Fig. 29, shows important signs of distress and is being reinforced and retrofitted so that it maintains its integrity and functionality.

The evaluation of all the inclinometer data leads to the interpretation of a deep seated landslide, retrogressing in its active part, shearing the foundations of one of the bridge's pylons while the foundations of all other pilar in this abutment would be "floating" in the sliding mass. The shear surface is located in the interface between the marls and the marlstones and limestones.

The emergency pumping to lower the groundwater showed almost immediate results, which were noticed by a significant displacement velocity reduction, as well as significant pore pressure reduction, as can be seen in Fig. 31.

A direct correlation between the rainfall data and the ground water response was also identified.

7.3.3 Tunnel design

The stabilization of the right abutment slope was conceived using an access tunnel and three adits excavated in the limestone beneath the slide surface and spreading laterally so that 3D limit equilibrium slope stability analysis numerical simulations associated to a hydrogeological model showed that the decrease of the water table would generate an increase in safety of 25 to 30 %.

Radial drains were perforated to reach and lower the pore pressures acting on the slip surface. Drains were located at constant intervals of the tunnel length; a very detailed geological mapping of the face of the excavation allowed perforation of additional drains at specific locations where geological features like open water bearing discontinuities would be intercepted and drained.

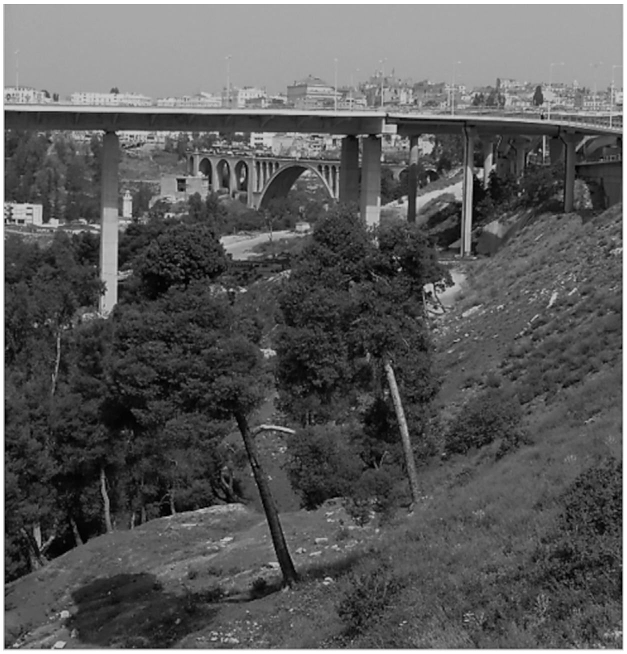

Figure 29. Signs of horizontal displacements at the surface.

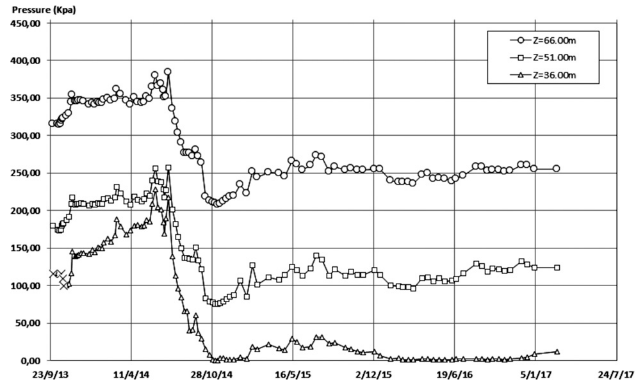

Figure 31. Pore pressure measurements of a piezometer located close to inclinometer 3003 (Fig. 26 and Fig. 27).

A complementary monitoring program was also installed. To present date tunnelling works are finished, but not all drains were installed.

7.3.4 Monitoring results

Figure 34 below shows the target drawdown of the drainage system as well as the drawdown achieved as to May 2019. The measurements and target values presented are plotted along the reference line of Fig. 32.

Drainage achieved by the tunnel has led, so far, to groundwater drawdown of around 25 m close to the new bridge. A more generalized groundwater drawdown is expected when the originally designed drains as well as drains located based on the mapped geology during tunnel excavation are installed and the slope will be stabilized.

8. Concluding remarks

The most efficient way to stabilize large unstable soil and rock masses is, usually, groundwater lowering. Other types of stabilizing solutions are often almost impossible to use, because of the enormous forces involved. Depending of the topographical and geological conditions, drainage tunnels can be a very efficient and definitive solution relying on gravity drainage. Tunnels also allow access for maintenance and drainage improvements at any moment during their design life.

Several successful projects were implemented around the world, especially in Asia, according to literature. At least 3 projects were built and successfully stabilized unstable ground masses in Brazil.

The drainage tunnel solution has several advantages, including gravity drainage, staged installation of drains during tunnel construction, permanent access to the drains

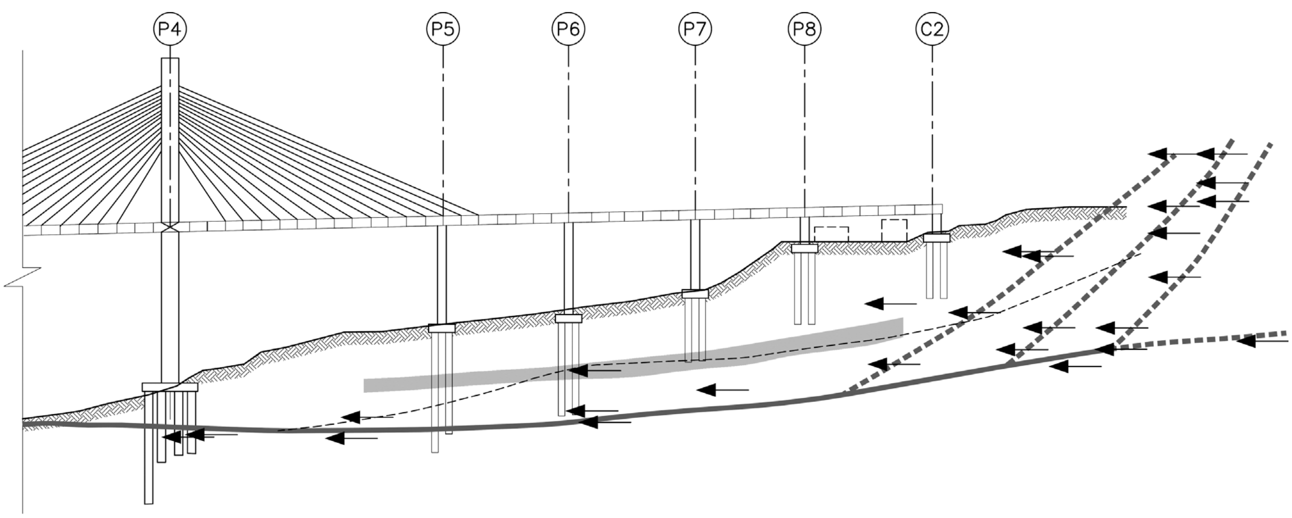

Figure 30. Cross section along the right abutment of the bridge and location of the slip surface.

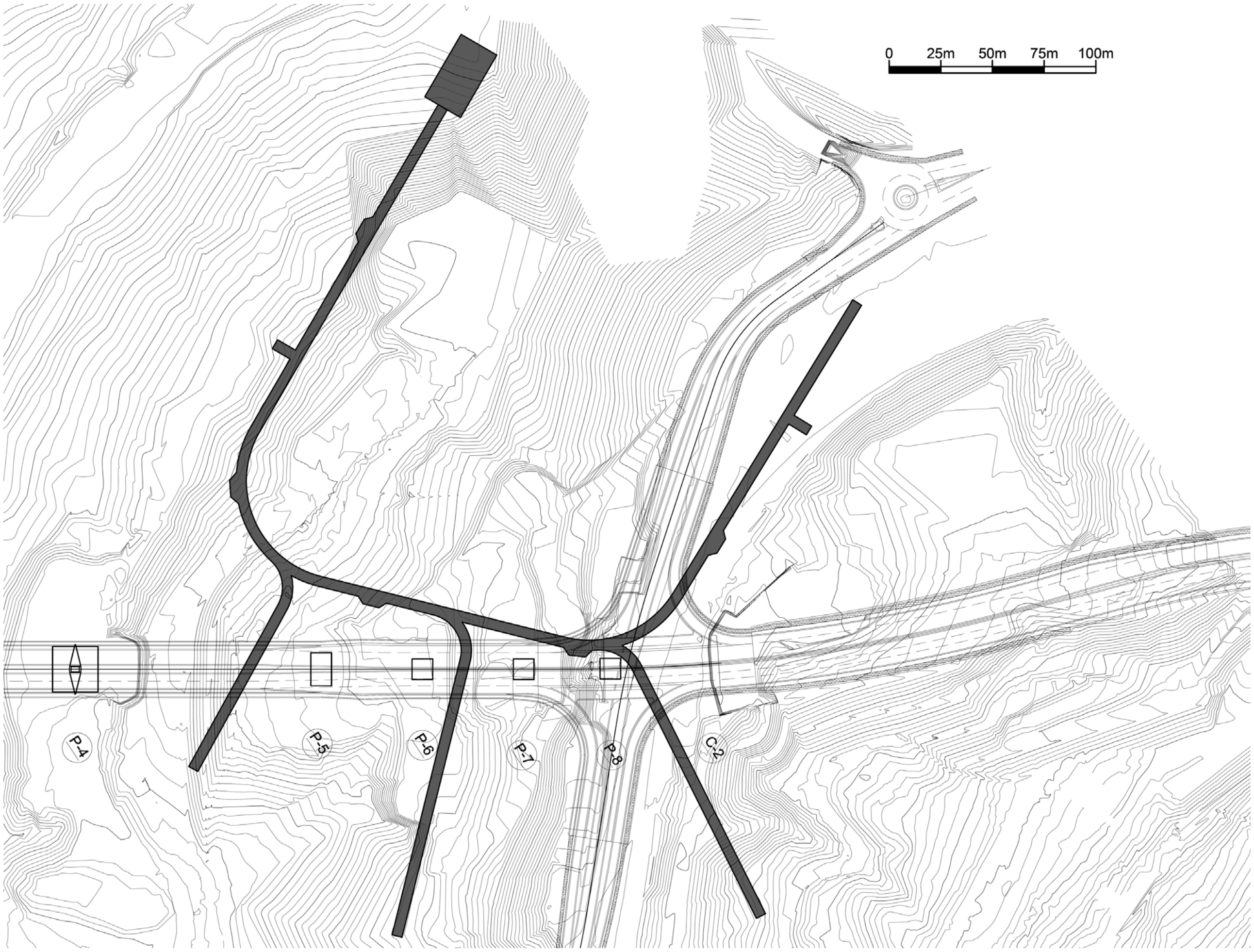

Figure 32. Plan view of main tunnel and adits, as well as location of bridge foundations.

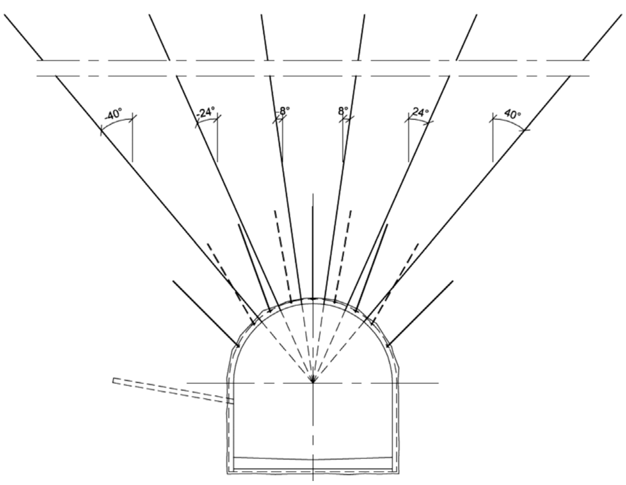

Figure 33. Tunnel cross section and drain location.

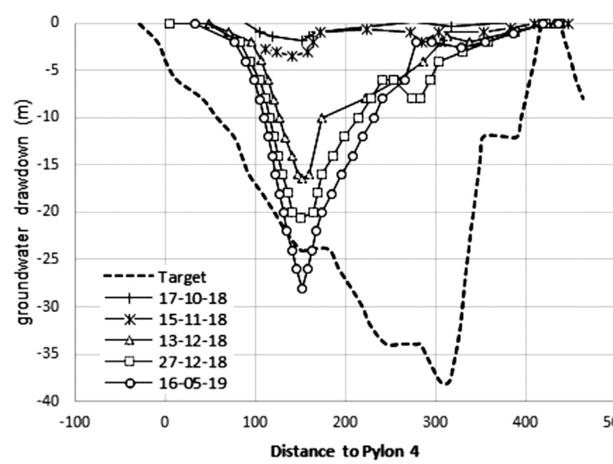

Figure 34. Groundwater drawdown monitoring results and target values.

allowing maintenance, possibility to expand drainage if necessary, among others.

The decision to build a drainage tunnel is often a long-lasting process, as well as the tunnel construction itself. Therefore, it may be interesting to initiate stabilizing the unstable ground mass, to protect the slope and existing infrastructure, by groundwater lowering through deep wells.

Acknowledgments

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