01 May 2013, 2:00 pm - 4:00 pm

Experiences in Large Slope Stability Problems Under Complex Geology

Camilo Marulanda
INGETEC S.A., Colombia

Alberto Marulanda
INGETEC S.A., Colombia

Camilo Phillips
INGETEC S.A.

EXPERIENCES IN LARGE SLOPE STABILITY PROBLEMS UNDER COMPLEX GEOLOGY

Camilo Marulanda
INGETEC S.A.
Bogotá, Colombia

Alberto Marulanda and Camilo Phillips
INGETEC S.A.
Bogotá, Colombia

ABSTRACT

This article presents a brief summary of the origin of metamorphic rocks, specially schist and the presence of shear or gouge zones in metamorphic rocks, as defects that induce weakness characteristics to the rock mass and that depart substantially from the traditionally evaluation of joints and discontinuities, turning eventually into failure surfaces that govern the stability conditions of surface works. The effect these weak zones inflict into the metamorphic rock mass, especially to schist, causing significant slope stability problems, is illustrated through three case histories. The presence of such defects in the rock mass, detected and analyzed by means of exploratory holes drilled from the surface, can be hardly anticipated during the design stage as far as location, dip direction and geotechnical characteristics, given their erratic alignment within the rock mass, and their disguise during the drilling processes when the clay infill is washed away by the drill water, making their recognition and readiness for lab test sampling even more difficult. Special care of these geologic features, often present in metamorphic rocks, must be taken through: 1) direct exploration –such as galleries–, 2) the elaboration and interpretation of adequate geological models and corresponding sensitivity analyses of shear strength parameters of the established failure surfaces and 3) sound decision making and implementation of stabilization measures based on engineering judgment.

INTRODUCTION

The presence of shear or gouge zones in metamorphic rocks are defects that induce weakness characteristics to the rock mass and that depart substantially from the traditionally evaluation of joints and discontinuities, turning eventually into failure surfaces that govern the stability conditions of surface works.

The article presents three cases histories of projects where slides generated in metamorphic rocks have posed danger to the construction and operation of the projects. The first case refers to the hydroelectric project of Porce III in Colombia, where several large slides affected during construction, the stability of the spillway slopes and the intake structure. The second case refers to the Mantaro project that generates almost a third of the electricity of Peru, and that after the construction of the Tablachaca Dam, and shortly before the filling of the reservoir an important ancient slide, named Slide No.5, was identified in the right abutment, upstream of the dam. The active zone area is approximately 7.50 ha and its volume is estimated at 3 Hm^3. The third case refers to the landslides that took place in the area of the reservoir in Clyde Dam in New Zealand, a project that has been in operation since 1993.

In these cases, the instabilities observed were intimately linked to the presence of erratic shear zones inside the metamorphic rock mass. These zones were mainly composed of schists or highly foliated metamorphic rocks. The cases also showcase the complexity of the slides formed in metamorphic rocks and their implications on slope stability analyses. In all of the three projects an important amount of effort was required to understand the deformation phenomenon and develop the geological and geotechnical models that allowed the analysis to define the stabilization measures required to reach the adequate factors of safety associates with the acceptable risk.

ORIGIN OF METAMORPHIC ROCKS AND THE DEVELOPMENT OF SHEAR GOUGE ZONES

Metamorphic rocks are derived from pre-existing rock types and have undergone mineralogical, textural and structural changes. The processes responsible for change give rise to progressive transformation in rock that takes place in the solid state. The changing conditions of temperature and/or pressure are the primary agents causing metamorphic reactions in rocks. Two mayor types of metamorphism may be distinguished on the basis of geological setting. One type is of local extent, whereas the other extends over large areas. The first type refers to thermal or contact metamorphism, and the latter refers to regional metamorphism. Another type of metamorphism is dynamic metamorphism, which is brought about by increasing stress [Bell, (2007)].

Most deformed metamorphic rocks possess some kind of preferred orientation. Preferred orientations may be exhibited as mesoscopic linear or plantar structures that allow the rocks to split more easily in one direction than in others. One of the most familiar examples is cleavage in slate; a similar type of structures in metamorphic rocks of higher grade is schistosity. Foliation comprises a segregation of particular minerals into constant bands or contiguous lenticles that exhibit a common parallel orientation. Slaty cleavage is probably the most familiar type of preferred orientation that occurs in rocks of low metamorphic grade. It is independent of bedding, which is commonly intersects at high angles, and it reflects a highly developed preferred orientations of minerals, particularly of those belonging to the mica family. Strain-slip cleavage occurs in fine-grained metamorphic rocks, where may not maintain a regular orientation, depending on the movements that have taken place under regionally stress in the different phases of deformation. Figure 1 presents a schematic of the grade of metamorphism [Tarbuck, (2005)].

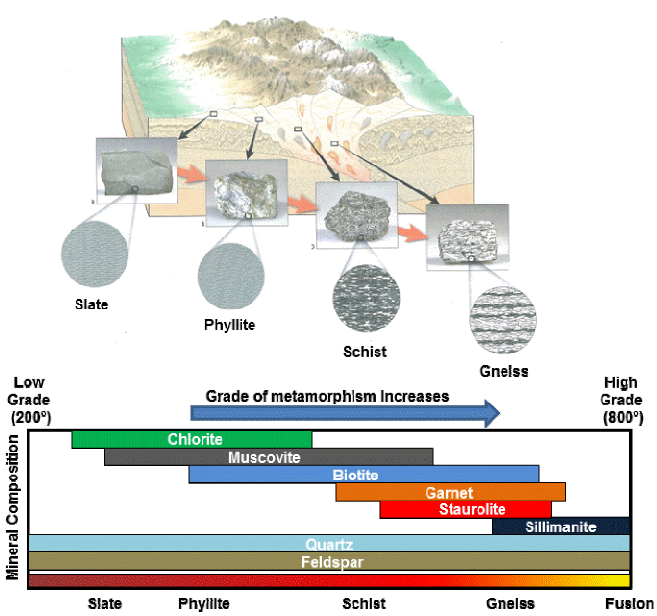

Fig. 1. Grade of metamorphism [After: Tarbuck (2005)].

Cleavage, schistosity and, to a lesser extent, foliation in regional metamorphic rocks may adversely affect their strength and make them more susceptible to decay. Moreover areas of regional metamorphism usually have suffered extensive folding so that rocks may be fractured and deformed. For instance, talc, chlorite and sericite schists are weak rocks containing closely spaced planes of schistosity.

In all types of metamorphisms, the growth of new crystals takes place in an attempt to minimize stress. When recrystallization occurs under conditions that include shearing stress, a directional element is imparted to the newly formed rock. Minerals are arranged in parallel layers along the direction normal to the plane of shearing stress; give the rock its schistose character. The most important minerals responsible for the development of schistosity are those that possess a flaky or tabular habit, the micas (e.g. muscovite) being the principal family involved. The more abundant flaky or tabular minerals are in such rocks, the more pronounced is the schistosity [Bell (2007) and Waltham (2009)] .

Foliation in a metamorphic rock is a very conspicuous feature, consisting of parallel bands or tabular lenticles formed of contrasting mineral assemblages. In contrast to schistosity that tends to disappear in rocks of high grade of metamorphism, foliation becomes a more significant feature. Rocks that have been subjected to high stresses, associated with folding or large faults or thrusts are often affected by dynamic metamorphism. This process includes brecciation, cataclasis, granulation, mylonitization, pressure solution, partial melting and slight recrystatllization. Depending on the movement of the rock segments near the fault, the minerals near the slope, shear zone or fault tend to form elongated grains giving a foliated aspect. In the process of developing these rock mass defects is necessary to distinguish between fault, fault zones and shear zones to explain with clarity the origin of these defects. A fault is by definition a fragile fracture where a visible displacement has taken place along it, generally parallel to the fault surface. A fault zone is composed by several fragile fault surfaces, subparallel and interconnected, tightly spaced and with breach zones and fault layers. The shear zone corresponds to a wide zone of deformation generated under ductile and ductile-fragile conditions and composed of mylonites. A ductile shear zone is characterized by the presence of mylonites rocks with high deformations that result in important levels of dynamic recrystallization of the affected rocks. The deformation or fragile behavior is associated with the formation of discontinuities and the loss of cohesion inside the rock.

Given the origin of these defects in the metamorphic rock mass, in particular the schists, shear strength depends on the mineral composition, alterations resulting from water, strain rate in the plastic state and size or width of the shear zone. Also, depending on shape of origin, is feasible to find different types of discontinuities inside the rock mass with variable shear strength. As a result, their characterization and parameter definition requires not only laboratory tests but also additional sensitivity analysis of the variations in the angles of friction and cohesion and thoughtful engineering judgment.

CASE HISTORIES

Porce III Hydroelectric Project

The Porce III Hydroelectric Project is located in Colombia´s Central Andean Mountain Range in the Administrative Department of Antioquia, 147 Km northeast of the City of Medellin. The Project consists of the impoundment of the Porce River with a total volume of 170 hm^3, by means of a 154 m high concrete face rockfill dam (CFRD), a 12.45 km long headrace tunnel, and an underground powerhouse with 660 MW of installed capacity. The CFRD dam has a crest length of 400 m and a total rockfill volume of 4.1 million m^3.

The spillway comprises a lateral chute controlled by four radial gates with a maximum discharge capacity of 11.350 m^3/sec, and a lower end sky jump deflector. The diversion scheme consists of a 35 m high RCC cofferdam and a 10.5 m diameter, 667 m long diversion tunnel. The low-level outlet scheme consists of a 6 m diameter tunnel, 423 m long, controlled by one radial and one rolled gate. The project will yield 3,106 GWh/year with a total cost of USD 448.2 million. Construction initiated on January of 2006, and impounding of the reservoir began the 24th of October of 2010 and finished two months later.

In the area of the Project a series of metamorphic rocks from the Paleozoic era are present, including quartz-sericite schists, quartz micaceous schists, quartzite schists, graphitic schists and a transition sector between schists and gneises. From a geostructural perspective, several inverse faults, foldings, shear zones and joints were mapped (See Figure 2). The micaceous and quartz-sericite schist are grey, finely laminated or foliated, banded, of medium to low stiffness. The quartzite schist is grey, of banded texture, stiff, with quartz lenses. The graphitic schist was grey of medium to low stiffness and lightly laminated.

During construction, several stability problems were encountered in the excavations for the spillway, dam and intake structure. All of these problems were related to the presence of shear or breacciation zones generated by cataclasis processes.

During the geotechnical investigations completed for this project, several shear and shear gouge zones were identified, with variable thickness ranging from 0.05 to 2 m parallel and across foliation planes in the area of the dam and in the tunnels. The material in the shear zone was pulverized rock with plastic soft fined-grained material accompanied by highly folded and fractured quartz veins. The shear zones that follow the foliation planes consisted of thin layers of weathered micaceous material (See Figure 3).

Based on the analysis of discontinuities of the metamorphic rock mass, it was established that the direction of the foliation system and the main sets of joints differed substantially from the shear gouge zones. These are arranged in an erratic and unpredictable way in the rock mass and are responsible for the stability problems encountered during the excavations. Figure 2 presents a geologic plan view and a cross section showing the differences found and the great dispersion of the discontinuities and the shear zones registered in various exploration galleries completed as part of the investigation of the dam abutments.

Dam Foundation

The selection process for the type of dam to be implemented in this project was a multifaceted and intricate process because of the complex geology at the dam site. The feasibility studies of the project anticipated the viability of two possible types of dams: an RCC and a CFRD dam. The first one was considered feasible by way of exploiting a quarry on the left abutment just downstream of the dam. The second was planned with a surface spillway, also located in the left abutment, the excavated materials of which could provide the necessary rockfills to construct the dam. The owner of the project, Medellin Public Utilities Company (Empresas Públicas de Medellín), selected the RCC dam and the design was completed by intensifying the geological and geotechnical investigations considering the more demanding foundation characterization for this type of dam.

Detailed geological surveys were conducted, as well as a significant number of drillholes, pits and trenches. Petite Sismique-type geophysical tests and a rock mechanics testing program were performed to determine the strength of the rock mass, the deformation modulus and the shear strength of its discontinuities' infills, especially of the clay-filled shear zones encountered at a number of locations throughout the foundation.

During the design stage, it was concluded that sub-horizontal gouge-filled shears found at the surface and inside the first exploration galleries in the abutments, could potentially generate unstable wedges in the foundation of the dam when subjected to the reservoir's hydrostatic pressure. Such conclusion was verified by anticipating construction of the grouting and drainage galleries foreseen in the design of the RCC dam and, therefore, confirming the orientation, continuity, shear strength and participation of the shears in the formation of unstable rock masses. As a result, an inclined gallery was excavated along each abutment, from which 15 smaller galleries branched off at different elevations, the presence of gouge material were recorded and samples were taken to determine the peak and residual strength, based on which a three dimensional model was prepared and stability conditions of the potentially unstable wedges were further examined.

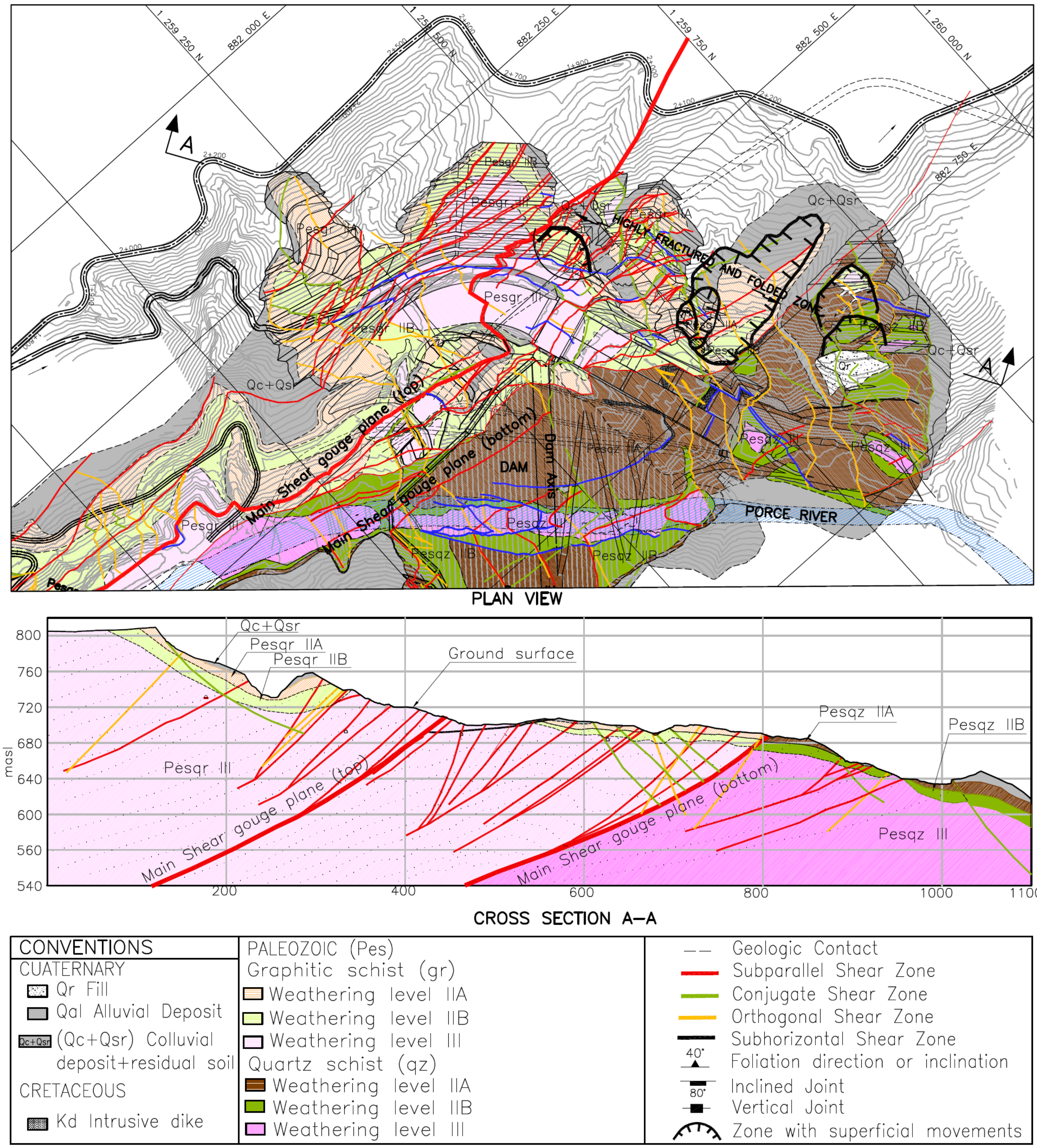

Fig. 2. Geologic plan view and cross section Porce III project

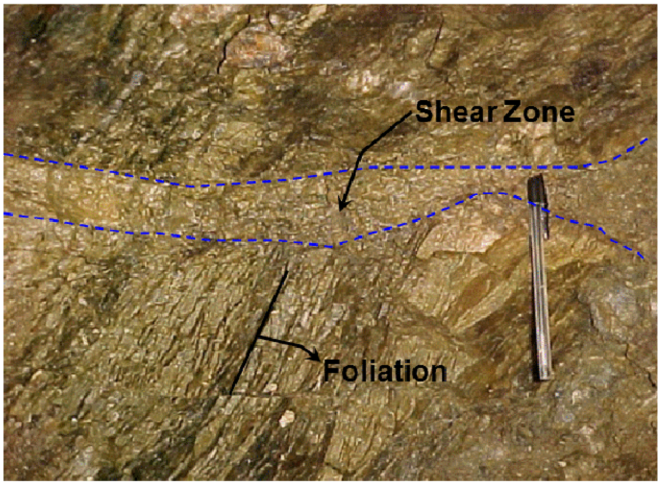

Fig. 3. Shear zone intersecting a foliation

<u>*Stability problem at the spillway left slope*</u>

Other example of the problems related with shear zones included the excavations over the diversion tunnel performed for the upper side of the spillway. In this zone, instability resulted from a shear gouge plane inside the schist bedrock. The shear plane had as spoon type of shape, with a dip angle of 26° SW towards the slope. In addition, the unstable block was delimited by the foliation planes and a lateral shear gouge zone associated with the foliation (See Figure 4 and 5). The mechanism analyzed had a three dimensional geometry and was affected by the presence of underground water accumulated behind and over the failure surface.

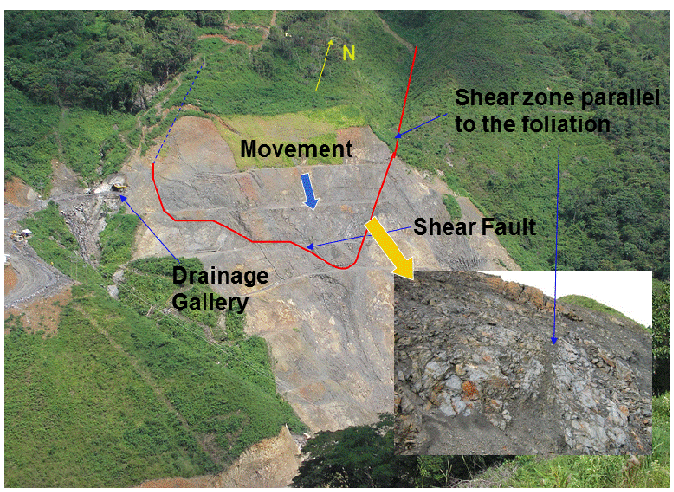

Fig. 4. Failure surface at the spillway left slope

The stability analysis in two and three dimensions with sensibility analysis for the shear strength parameters were required to establish the short and long term behavior of the unstable zone.

For the analysis and interpretation of the model, several exploratory boreholes were required. The sensibility analysis and back-analysis indicated that the residual shear strength was given by friction angle in the order of 25 to 28°. Laboratory tests of shear strength of undisturbed samples registered friction values of between 22° and 34°, with a cohesion that ranged from 0 to 80 kPa.

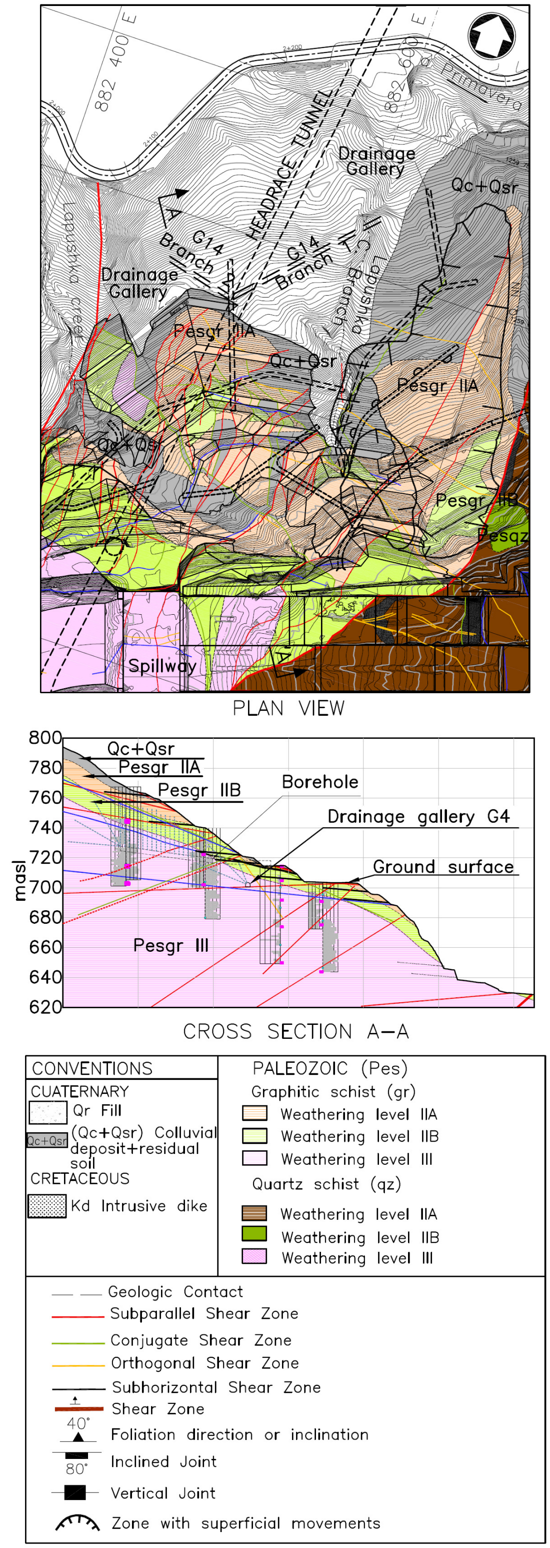

Fig. 5. Geologic plan view and cross section at the spillway.

The studies and the exploration demonstrated the presence of a high water table, affecting the stability conditions of this unstable zone. From the stability analysis, it was established the need to build drainage galleries and a superficial drainage system in order to lower the water pressures that were generating the instability problem. The model of analysis found feasible to increase the level of static safety using the drainage from around 1 to up to around 1.35. Additional measures including the installation of tendons of 72 tons where defined. Once the drainage measures were implemented, a drastic decrease in the movements was registered, therefore improving the stability conditions (Figure 6).

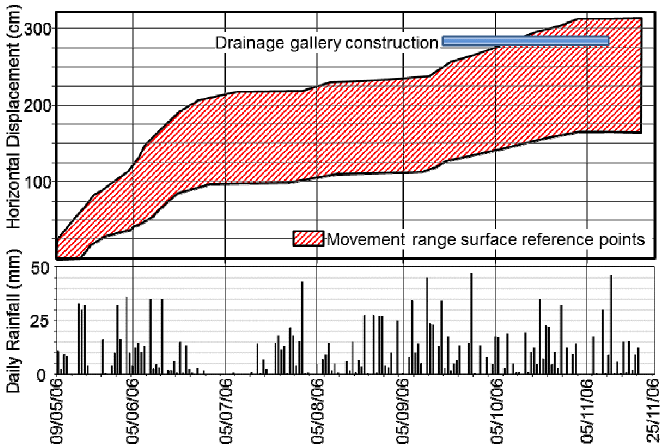

Fig. 6. Displacement rate reduction due to the excavation of the drainage galleries.

<u>*Stability problem at the intake structure zone*</u>

Another case related with the influence of shear zones in stability problems in the Porce III project was related with the excavations for the intake structure of the headrace tunnel. In this sector, a series of shear gouge planes with triturated and clayey material appeared and delimitated the potentially unstable rock wedge illustrated in Figure 7. Towards the base of the triturated material a band of clay of 0.5 m of width was located. This shear zone had a totally irregular geometry forming a sort of laydown "S" where a great quantity of water accumulated.

In order to solve the stability problem, it was necessary to perform two and three dimensional analyses together with parametric analysis and sound engineering judgment to find the best solution to ensure the stability in the short and the long term conditions for the slope and the intake structure itself.

During the parametric and sensibility analysis of the shear strength a value between 22° and 27° was established for the friction angle.

The results from the stability analysis performed in two dimensions and of the critical wedge considering the situation when the movements occurred as well was operation (with reservoir and rapid drawdown) indicated static safety factors of over 1.5 once the corrective measures were taken defined based on the analyses. These measures included the placement of a buttress behind the intake structure, reinforcement measures with tendons and a system of drainage holes. In addition, geotechnical instrumentation was implemented including inclinometers, surficial reference points and piezometers.

After the construction of the stabilization measures and the implementation of the instrumentation a considerable vertical cut was performed for the intake structures and no stability problems were observed, illustrating the effectiveness of the measures.

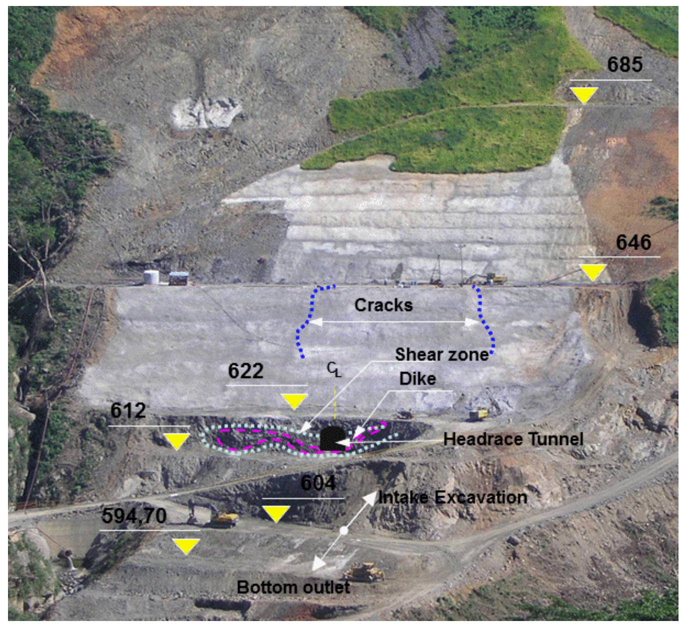

Fig. 7. Stability problems at the location of the intake structure.

<u>Mataro Hydroelectric Project- Slide No. 5</u>

After the construction of the Tablachaca Dam, and shortly before the filling of the reservoir (September 1972), an important ancient slide, named Slide No.5, was identified in the right abutment, upstream of the dam (see Figure 8). Initial investigations indicated that the slide mass could be divided into an upper zone (initially inferred inactive) and a lower active zone. The active zone extends from elevation 2660 masl to elevation 2920 masl above the dam and reaches a maximum depth of 70 m measured from the surface. The active zone area is approximately 7.50 ha and its volume is estimated at 3 Hm^3. The upper portion of the slide is comprised by an "inactive" zone that extends up to elev. 3200 masl, covering an area of 14 ha approximately and an active zone representing an additional 5 Hm^3.

Fig. 8. General view of Slide No. 5.

In September of 1972, upon completion of the dam, filling of the reservoir was initiated from river level up to elevation 2695 masl. During this period, slope movements were observed as well as the formation of cracks on the surface. Movements reached several meters and generated a slide of about 65,000 m³ of material into the reservoir. As a consequence of this phenomenon, the reservoir had to be lowered and movements monitored although not systematically until 1980 when intense rainfall occurred and the slide was again visibly active. In February 1982, movements increased to daily rates of up to 5 cm/day. Consequently the project was declared in a state of emergency and the owner decided to undertake the necessary studies to carry out the stabilization works [Marulanda et al., (2010)]

In June 1982, the emergency works required to control the movements of Slide No. 5 were undertaken. The main contingency measures consisted of (1) construction of a free draining buttress at the foot of the slide, (2) installation of prestressed anchor up to 110 m long near the dam and at locations where the buttress could not be placed, (3) construction of two drainage galleries with radial drains within the rock mass and (4) drilling of horizontal drainage holes from the surface. Construction of the buttress fill included densification of existing sediments in its foundation by means of compacted gravel columns, as these sediments were determined to be susceptible to liquefaction under a seismic event.

Construction of the buttress started in September of 1982 with the treatment of the sediments in the reservoir and was completed in September of 1983. The works comprised compaction of 1,583 gravel columns in an area of 7,600 m² and placement of 467,000 m³ of fill material to build the buttress. Subsequently, 419 anchors were installed in three different walls exerting a combined force of 486,000 kN (48,600 ton). The sub-surface drainage system consisted of the construction of two galleries excavated into the rock mass behind the slide with a combined length of 1,527 m. A total 190 radial drainage holes were drilled along the galleries with a total length of 3,290 m. Twenty-one horizontal drainage holes were drilled from the surface with a total length of 1,282 m. The superficial drainage system consisted in the construction of 5,963 m of drainage ditches at different levels across the slide. Figure 8 shows the dam site and main stabilization works at Slide No 5.

Up to date, 37 inclinometers and 37 piezometers have been installed, 130 surface survey monuments built, 16 load cells installed in the anchors, as well as 20 groups of extensometers. In 2006, a complete evaluation of the Tablachaca project was performed with the purpose of undertaking a comprehensive study of Tablachaca Reservoir (INGETEC S.A., 2006). One of the main objectives of the study was to assess and diagnose the general stability of Slide No 5 by means of an updated geological and geotechnical model to determine the stabilization works required to ensure the long term stability of the slide.

The geotechnical model was developed based on three essential aspects. First, the subsurface profile was defined based on the results obtained from the geological investigations and the understanding of the genesis of the slide. For this purpose, cartographic maps, geotechnical investigations, site inspections, geological records of the galleries and the geological models were used. Second, the field behavior was analyzed by examining the available laboratory and field tests results as well as the interpretation of the available geotechnical instrumentation. Third, the stability analyses of the slide No. 5 were performed and calibrated using computational tools that allowed the integration of the geotechnical information in a three dimensional model.

With a three dimensional model of the slide No. 5, it was possible to have an accurate knowledge of the spatial distribution of the various materials that comprise the slide area, and therefore allowed for a better understanding of the origin and current behavior of the slide. In this manner, the proposed three-dimensional geological model of Slide No. 5 may be directly applied in developing the geotechnical model, thus allowing a more realistic modeling of the sliding processes and the identification of the slip surfaces.

The slide is essentially a melange of broken black carbonaceous slate and brown quartzite phyllite and fragments in a matrix of gravel, sand and silt-sized detritus. It over-rides graphitic black slate in its upstream three quarters and quartzitic phyllite closer to the dam. A gouge zone several meters thick occurs at the contact and represents the stratum where the slip surfaces have developed. The morphology of the bedrock of the slide No.5 plays a very important role, because it controls the geometry of the slip surfaces generated and the direction of movement of the potentially slide masses.

Figure 9 presents the presence of a scour-pool or cavity formed in the bedrock which is covered by the ancient landslide and the more recent colluvium deposits. In the area immediately upstream of the dam a channel or gorge exists that descends to elevation of approximately 2800 masl down to the riverbed. The gorge is oriented towards the reservoir and coincides with the movements that have occurred in this area. In the intermediate zone, the scour-pool does not reach the bottom of the river, but rather is bound by a rock protrusion composed of highly fractured and gouged slate. In the west sector of the slide, another type of gorge, of less depth and width than the one previously described is located (descends to an approximate elevation of 2800 masl).

The piezometric records were analyzed in order to determine the effect that the stabilization works executed in the eighties (drainage galleries, horizontal drainage holes from the surface and drainage ditches) had in the groundwater behavior. Three different scenarios of analysis were defined: (1) before the construction of the emergency stabilization works (before 1982), (2) with the current conditions under a dry season and low levels of groundwater and (3) with the current conditions (after of the construction of the drainage works) and considering a rainy season.

Figure 9 also presents the phreatic surface prior to construction of the drainage galleries. This figure illustrates that prior to construction of the galleries; a buildup of water level developed in the scour-pool that forms the bedrock, thus saturating the slide material. At the same time, such a rise in water level, connected the scour-pool to the reservoir by means of two underground flow paths (gorges) located in each side of the landslide, developing seepage forces in the lower portion of the sliding masses and reducing the stability. Upon completion of the drainage galleries and under the current conditions, the water build up in the scour-pool continued to take place but its level decreased and the underground water flow disappeared. Based on piezometric data, it could be determined that the construction of the drainage galleries lowered the water table in around 30 m on the lower zone of the slide and up to 70 m in the area of the scour-pool formed by the bedrock, indicating the effectiveness of drainage works.

From the analysis of superficial and deep movements registered in the area of Slide No. 5, two zones with different behavior were identified: the lower sector of the landslide that extends from level 2660 masl to level 2900 masl and the upper area that extends from 2900 masl to 3115 masl, approximately.

Based on longitudinal and cross sections defined in the slide, different slip surfaces were defined within the entire Slide No. 5. The identification of the potential sliding masses considered: (1) morphology of the bedrock, (2) head scarps identified on the slide surface, (3) analysis of the deep and superficial movements, (4) the depth of shear locations in the inclinometer readings and (5) the existence of shear and/or gouge zones. These surfaces were distributed in space to define the three-dimensional shape and direction of each one of the potential sliding masses that are identified and described below.

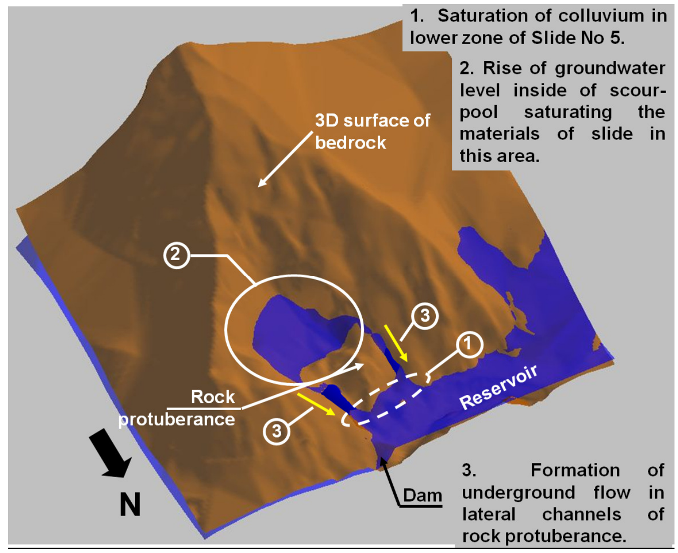

Fig. 9. Behavior of groundwater levels before construction of drainage works Superficial and Deep Movements.

In the lower area of the slide, three potential masses were identified and are referred as A, B and C. Mass A is located in the intermediate zone, Mass B is located in west zone (the farthest from the dam), and Mass C is located in east zone, immediately upstream of the dam (see Figure 10).

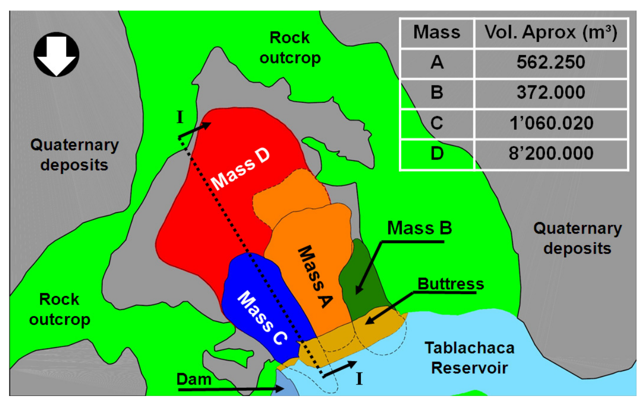

Fig. 10. Plan view of potential slide masses in Slide No. 5.

In the upper area of the slide, Mass D was identified by means of the deep shear points registered on the inclinometers above elevation 2750 masl and below elevation 3115 masl. Based on these shear points the deeper slip surface was identified behind the slip surfaces A, B and C, which have less depth than Mass D (see Figure 10).

The strength parameters for the different materials that compose Slide No. 5, shown in Figure 11, were determined based on the different investigation programs performed since the early stages of the project. Special care was given in adequately characterizing the gouge zone, where the slip surfaces were developing. The most recent sampling program performed in 2005 included undisturbed samples obtained from the walls of galleries, samples taken with triple-tube core barrel taken from the 722 m of boreholes performed in 2005.

As a result of the large displacements that Slide No. 5 has experienced throughout history, the material of gouge zone was characterized based on its residual strength. Direct shear, ring shear and triaxial tests (CU with measurement of pore pressure) were performed (Garga, V. 1996). Table 1 presents the summary results of an extensive laboratory program performed in material obtained from the Gouge zone.

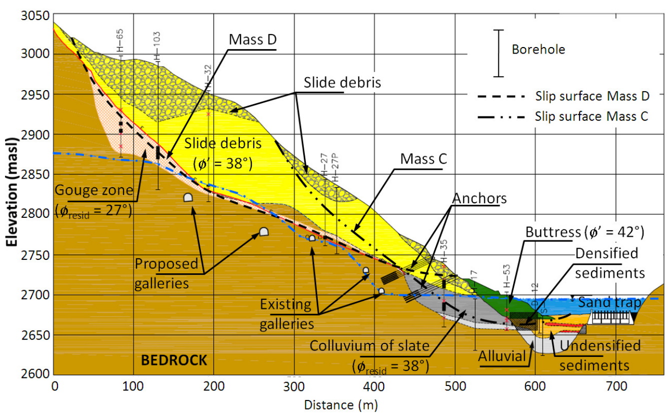

Fig. 11. Cross Section I-I through Slide No. 5 (plan view presented in Figure 10)

After defining the hydrogeological conditions, the geometry of potential slip surfaces and the geotechnical parameters of each one of zones that constitute the Slide No.5, 2D and 3D stability analyses were performed. Two scenarios for the evaluation of the stability were considered: one with the situation of 1982 (without the construction of stabilization works), and another with the current conditions (with stabilization works).

Table 1. Shear Strength parameters of gouge zone determine from laboratory tests.

S_u (kPa)	Peak Shear Strength				Residual
	Total Stress		Effective Stress		
	$c_{(cu)}$ [kPa]	$\phi_{(cu)}$ [°]	c' [kPa]	ϕ' [°]	ϕ'_r [°]
206	166	24	70	28	23

For these two scenarios, two types of conditions were considered: (1) normal conditions (static condition with variation of the reservoir level and sudden drawdown) and (2) extreme conditions (intense rainfalls or seismic event). In these extreme conditions, the undrained shear strength of the gouge zone was used.

The initial two dimensional analyses performed for different sections of the slide, gave a considerable variation on the Factors of Safety. Therefore and given the complex topography of the site, it was considered that three dimensional analyses were required to accurately reproduce the failure mechanisms of Slide No.5. Some of the three-dimensional effects included the lateral load transfer produced in the masses A and C (located on the gorges) and the fully three-dimensional effect on mass D produced by the scour-pool and the rock protrusion in the bedrock. These analyses were based on the developed geotechnical model previously described. Also, the effectiveness of the three-dimensional model was confirmed with back-analyses performed to validate the shear strength parameters. The calculated parameters were very similar to the ones obtained from the laboratory tests, which was not possible with a two dimensional analysis.

The stability of Slide No.5 was analyzed under current condition and during the rainfall event that occurred in 1982. The four identified potential slide masses A, B, C and D were analyzed. All the surfaces were implemented in three dimensions.

The analyses included the groundwater levels registered during the 1982 scenario (without the drainage galleries) and under the current conditions (with the drainage galleries), considering the maximum and minimum levels of the reservoir.

Table 2 illustrates the Factors of Safety obtained from the two and three dimensional analyses for the different slide masses. The factor of safety obtained with the three dimensional analyses was greater than the one obtained with two dimensional analyses. The difference ranges between 5% and 20%, being the highest for the Mass D. The factors of safety calculated for the three dimensional analysis were based on an extension of the Spencer's method, which was derived based on the approach proposed by Lam and Fredlund (1996) and Hungr, (2006).

In addition to the static three dimensional analyses, pseudo static analyses were also performed including an estimated induced deformations in the landslide, which is a very important issue considering the seismicity of the area (PGA=0,50g). Because of the great knowledge obtained throughout the years of investigating Slide No. 5, it was considered acceptable to admit lower Factors of Safety that what is usually utilized. Based on this premise the following complementary stabilization works were recommended.

Based on the results of the stability analyses, it was determined that masses A to C have acceptable stability conditions. Therefore, only a rehabilitation of the buttress was recommended to secure the long term effectiveness of that structure and assure the stability of these masses, especially B and C. In the case of mass D, it was determined that under a severe rainfall event (return period of 100 years) the mass would exhibit a precarious stability condition. This, due to the fact that the existing drainage galleries do not have the range to control the groundwater level at the base of mass D, and therefore a build-up of water level could occur under a severe event. Based on this assessment, two additional drainage galleries were recommended as shown in Figure 11.

Table 2. Comparison of Factors of Safety from two and three dimensional analyses for the different masses of Slide No.5.

Mass	**Type of analysis**	**Factor of Safety (Max level of reservoir)**			
		CONDITION			
		1982-without rains	**1982-With critical rainfall**	**Present Condition - Normal without rains**	**Present Condition - With critical rainfall**
A	2D	1,04	0,95	1,06	1,02
	3D	1,21	1,06	1,25	1,18
B	2D	0,95	0,88	1,60	1,57
	3D	1,00	0,94	1,69	1,67
C	2D	1,00	0,90	1,10	1,05
	3D	1,12	0,99	1,37	1,33
D	2D	1,27	1,21	1,32	1,25
	3D	1,54	1,29	1,62	1,35

<u>Shear zones in the hydroelectric Project of Clyde-Cromwell Gorge-New Zealand</u>

The hydroelectric project of Clyde Dam is located in the southwest of New Zealand. Clyde Dam has an elevation of 102 m and a capacity of 432 Mw. With the construction of the dam a 62 m deep and 15 km long lake was created. Several old landslides of big magnitude existed at the Cromwell Gorge (where Clyde Project is located) which were triggered during the impoundment of Dunstal Lake (See Figure 12). The volume of these landslides ranged between 1 and 1000 Mm^3.

The site geology comprises Mesozoic micaceous and chloritic schists, presenting foliation planes, joints, breccia faults and shear planes that weaken the rock mass.

From a geological perfective, two types of schists were found. One type consisted of grey micaceous schists with dominant foliation and high anisotropy with regard to their strength and stiffness. The other type consisted of massive and poorly foliated green chloritic schists.

Most of the landslides were found in schists and are associated with translational creep and movement effects through unfavorable foliation planes, breccia zones, shears and joints. The origin of these old landslides has been associated with buckling effects of the weaker schists located in steep slopes and with river erosion as shown in Figure 13.

Three types of instabilities were identified. The first type included landslides with planar failure surface along foliation planes or shear zones as the main mechanisms, including also movements across the shear zones intersecting foliation planes. The second type included wedge failures, given the conjunction of joint planes, foliation and shear surfaces, respectively. The third type consisted in failures by buckling generally located in the upper parts of the slopes and controlled by high angles of parallel planes of the fractured and relaxed rock.

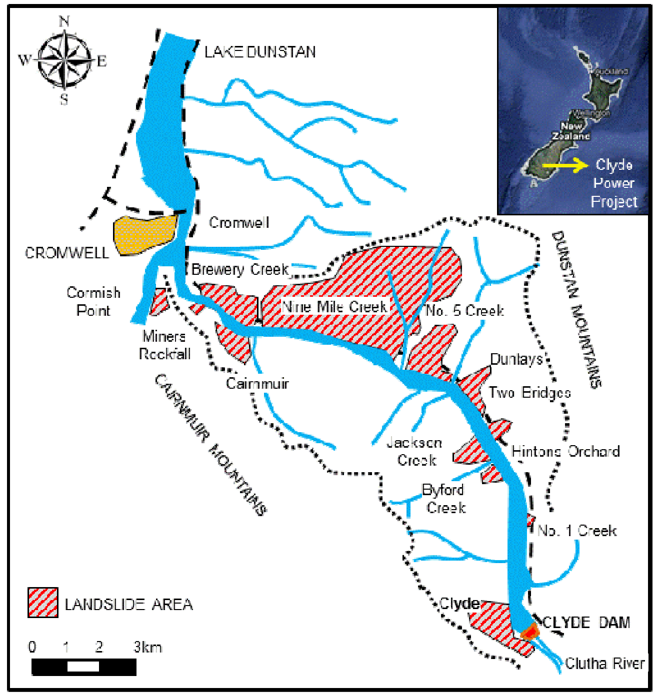

Fig. 12. Cromwell Gorge landslide location [After: Gillon et al., (1991)]

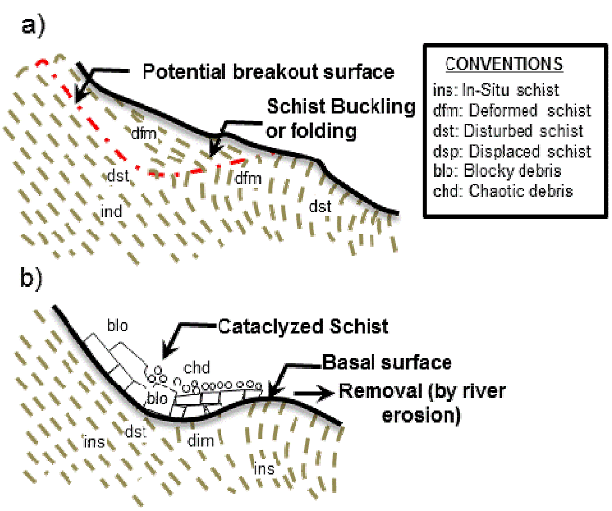

Fig. 13. Origin of some landslides at Cromwell. a) bedrock flow in sheared rock. b) development of landslide from bedrock flow [After: Gillon et al., (1991)]

It was established that the development of the movements was controlled not only by the aforementioned geo-structural defects, but also by the underground water and by the strength properties of the rock mass [Gillon et al (1991) and Beetham et al. 1991].

Initially the hydrogeological conditions were believed to be very simple. However, the information obtained in exploratory boreholes and galleries allow the identification of the underground water conditions complexity. These conditions included single or multiple aquifers containing both confined and unconfined elements. In addition it was recognized that high pressure groundwater could be confined beneath the basal failure zone behind the buckled schist, creating a permeability barrier [Gillon et al (1991)].

Most of the rock landslides were characterized as translational/rotational, developing partially at the base of an anticlinal structure. The landslide material consisted of blocks and fragments of schists of various sizes of up to 10 m in diameter that moved over a fault surface with a width between 0.5 and 2 m. Generally, the shear surfaces contained triturated schist material, fault breccia or cataclized rock with a mixture of fine gravel, sand, silt and clays with thin discontinuous clay-silt lenses. In one of the landslides, known as Jackson Creek, studies revealed that it originated by a fault surface in rock that formed along a preexisting defect of tectonic origin, that corresponded with another series of shear zones dipping 20 to 40 degrees towards the river and affected by the affluence of the water table [Gillon et al (1991)].

The micaceous schists exhibit a dependence of the strength and modulus with the inclination angle of the test against the rock foliation plane. The strength determined through a series of unconfined compression test varied from 5 to 105 MPa, with an average value of 40 MPa. The rock average rock modulus was 30 MPa measured parallel to the foliation and 60 MPa when measured normal to the foliation planes. The shear strength also presented variations between 50 and 195 percent depending on the orientation of the foliation planes.

An effective friction angle of 25° and zero cohesion were back-calculated based on the slope stability analyses developed to define the remedial measures. These friction angle values were compared with direct shear tests results developed to measure the residual strength of unaltered fault material samples. These tests provided cohesion values between 10 and 20 kPa and friction angles ranging from 20° a 23.5°.The differences between the back-calculated and test measured friction angles are explained by shear plane undulations and asperities.

The analyses developed in a period of 17 years revealed the complexity of the landslides, which required additional analyses and field investigation. The reservoir impounding was postponed until all of the stabilization work was completed. The stabilization work consisted mainly in the construction of tunnels and galleries located below the bedrock with drillings directed towards the fault surface that served both as additional field-investigation and drainage. In addition, in some of the landslides, buttresses were built at the landslide base and pumping dwells and impervious curtains were constructed to control de effect of the reservoir impounding on some of the unstable zones.

The reservoir was filled in several stages with continuous monitoring of the displacements and pore water pressures using inclinometers, surficial reference points and piezometers. The monitoring of the water levels and the deformation rates of the landslides provided the basic information to assess the effectiveness of the stabilization works. The subsurface investigation and remedial measures developed for Clyde Power project allowed also to identify the toe buckling phenomena which in some cases triggered landslides with a clear basal failure zone [Gillon et al (1991)].

CONCLUSIONS

The presence of shear zones, joints or cataclastic rocks in metamorphic rock mass constituted by schists are very common defects of the rock mass as presented in the case histories presented. The strength of these defects is related to their origin, the type of mineralogy, the presence of rock blocks embedded in the clay matrix and the weathering effects from water including the deformation in the plastic regime and the size and thickness of the shear zone. These defects determine the behavior of the rock mass and in particular are related to slope stability problem. Given the nature of these defects their strength parameters are quite variable and therefore parametric analysis are required to have adequate results from the stability studies.

During the design stages, is practically impossible to determine in detail the location of these defects inside the schist rock mass given their erratic presence and the difficulties of borehole sampling. As a result, in many cases it is necessary to use the exploratory galleries to have a better understanding and a more representative model of the problem.

The foundation of dams in concrete, for example, required the excavation of galleries along the abutments to cover the foundation completely and ensure that there are no defects that can compromise the dam stability.

In excavations where the presence of shear zones and the associated stability problems is evident require permanent observation, instrumentation and control, requirement that follows the premise of the observational method proposed by Karl Terzaghi and Ralph B. Peck [Peck (1969)]. These include continuous geological investigation, sensibility analysis of shear strength parameters, well developed geological and geotechnical models to define the real behavior of the unstable zones and sound and efficient stability measures.

As a result of its clay composition, shear zones tend to concentrate or limit aquifers and porewater pressures, inducing a decrease of the slope factor of safety and in some cases triggering landslides. Therefore, it is important to put great emphasis in drainage systems using galleries or directed boreholes towards the failure surfaces.

REFERENCES

Marulanda, C., Amaya, F., Ruiz, W. and Gutiérrez R. [2010]. "Evaluation of a Complex Landslide by Means of a 3D Geotechnical Model" *GeoFlorida 2010: Advances in Analysis, Modeling & Design*. GeoFlorida 2010

Bell, F. G., [2007]. "*Engineering Geology*", Second Edition. Butterworth-Heinemann. Oxford, MA, USA.

Beetham, R.D, Moody K. E, Fergursson, D. A, Jennings D.N y Waugh, P.J, [1991]. "Landslide development in schist by toe buckling". *Landslides,Vol. 1.* pp.17-23.

Gillon. M.D., Anderson. C.K., Halliday, G.S and Watts, C.R., [1991]. "Jackson Creek Landslides stabilisation". New Zealand". *Landslides,Vol. 1*. pp.707-713.

Gillon, M.D, and Hancox, G.T, [1991]. "Cromwell Gorge Landslides- A General Overview". *Landslides, Vol. 1,* pp.83-102.

Lupini, J.F., Skinner, A.E. y Vaughan, P.R., [1981]. "The drained residual strength of cohesive soils". *Geotechnique* Vol. 31, No.2., pp.181-213.

Mesri, G. y Cepeda-Díaz, A.F., [1986]. "Residual strength of clays and shales". *Geotechnique* 36, Vol. 2. pp.269-274.

Peck, R.B [1969]. "Advantages and limitations of the observational method in applied soil mechanics". *Geotechnique,* 19, No. 1, pp. 171-187.

Skempton, A.W., [1985]. "Residual strength of clays in landslides, folded strata and the laboratory". *Geotechnique,* Vol. 35, (1), pp.3-18.

Stark, T. D. y H. T. Eid., [1994]. "Drained residual strength of cohesive soils" *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE. Vol 120, No GT-5, May, pp. 856-871.

Tarbuck, E.J, and Lutgens,F., [2005]."*Earth: An Introduction to Physical Geology*". 8th Edition. Pearson Prentice Hall.

Waltham, T. [2009]. "*Foundations of Engineering Geology"*, Third Edition". Taylor and Francis.