

ALTO MAIPO PROJECT – CHILE

REPORT TO CONSTRUCTORA NUEVO MAIPO JV

**REVIEW OF THE TUNNELLING CONDITIONS RELATING TO THE VOLCAN
AND ALFALFAL II TUNNELS**

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REVIEW OF THE TUNNELLING CONDITIONS RELATING TO THE VOLCAN AND ALFALFAL II TUNNELS

EXECUTIVE SUMMARY

At the request of CNM Joint Venture (CNMJV) an independent review of the geological and geotechnical information provided at the time of tender (Appendix Z) has been carried out. In addition, it reviews the March 2015 Update of the geology supplied by the Owner and the tunnelling problems experienced to date during construction. It also looks at the tunnelling risks going forward and draws conclusions with regard to the ground conditions and their impact on safety, progress and tunnelling methodologies.

The general conclusion from the review is that the data provided at the tender stage in Appendix Z was not sufficient for the CNMJV to understand the range of ground conditions and behaviour in the tunnels. More importantly, it painted an optimistic picture of the ground conditions and this was reinforced by the Owners decision to employ open faced TBM's to facilitate construction. The tunnelling conditions and progress to date has confirmed the conditions are far more challenging than this picture suggested. It is now not possible for the CNMJV to continue with the project without a substantial review of the construction methodologies, the risks that are faced in both the Volcan and Alfalfal II Tunnels and their impact on programme, costs and safety.

Current experience has shown that the percentage of Type IV and V rock support classes, whether D&B or TBM, is much higher than could have been predicted from Appendix Z. The percentages of Poor to Very Poor ground (based on RMR values) have been 6.8%, 14.4% and 21.4% respectively. This has had significant impact on progress; to date V1 has averaged 2.9 m/day, V4 3.2 m/day and V5 (in total) 1.3 m/day (based on CNMJV records).

It is evident that the Drill & Blast (D&B) construction methodology can control deformations at the face although progress is slow. It is also evident from V5 that where there are sub-horizontal shear zones and the McNally support system has to be used to control overbreak progress is very slow and will continue to be the case. Overstressing in Poor and Very Poor ground conditions at the face plus the presence of aquifers will continue throughout the tunnels. Because of the difficulty of controlling deformations in Poor and Very Poor rock mass quality, using an open-face TBM in these ground conditions will be difficult and there will be safety concerns.

In any highly stressed rock which has experienced compression and both folding and thrusting the potential exists for rock bursts to occur. This is also the case for any igneous intrusions which have been subject to deformation.

Both squeezing ground conditions and cave-ins have occurred and in particular these are associated with the sub-horizontal shear zones (both VA4 and V5). In these weaker shear zones the strength-stress ratios are significantly less than 1.0 but it is also the case that the squeezing potential of these bands is controlled in the multi-layered sequences by the stronger bands. With load transfer to the stronger beds, the weaker bands will undergo relaxation and a number of cave-ins have occurred and this phenomenon will continue throughout the tunnels. Going forward, deformations in the weaker layers is a problem which can be expected to increase in severity as the cover increases, particularly the Volcan Tunnel which reaches a maximum of 1500m. The

principal concern is that failure will take place rapidly at the face with overbreak in the crown and the possibility the collapse of material could work back from the face to the cutterhead and the working areas behind.

The sub-horizontal shear zones contain clay and act as aquicludes. The presence of water in the bands above and below the shear zones results in aquifers and significant water inflows which exceed the contract limits and those allowable by the environmental permit. This will continue at intervals throughout the tunnelling whether the bands are sub-horizontal or sub-vertical. More importantly, unless the hydrostatic pressures are relieved they will contribute to overstressing of the weak shear zones. In VA4 the artesian conditions exist due to the downgradient of the tunnel.

Appendix Z is a geotechnical baseline report. It is important to establish this is the information the CNM JV had available at the tender stage and to make judgements about the ground conditions and ground behaviour.

The general stratigraphy is explained but the detailed stratigraphy of the Formations is not well understood due to a lack of sub-surface investigations. While there are descriptions of some of the rock types, the sub-horizontal shear zones are not identified and their distribution and frequency within the Formations was impossible to predict for tunnelling purposes.

The detailed structural geology of the project area was not explained either in Appendix Z or the 2015 Update. This is a major deficiency since all of the problems currently being experienced underground could have been anticipated by the Engineer and if properly described would have enabled CNMJV to better understand the hazards and associated risks relating to the ground conditions and ground behaviour.

Specific issues relating to the detailed structural geology are the type of folding. This is primarily buckling at low temperatures of the Cenozoic multi-layered sequence and the flexural type folding has resulted in intense shearing parallel to the banding. This particularly affects the weaker tuffs and pyroclastic beds and these are frequent throughout the Formations.

The significant internal deformation of the weaker tuffs and pyroclastic bands during folding (which can range in thickness from 1-5m thick) is characterised by shearing, slickensiding and thin clay seams. These are both sub-horizontal and sub-vertical depending on their position in relation to the fold structures. None of this was adequately described in Appendix Z.

There are two main fault structures expected in the Volcan Tunnel, Las Cortaderas and the El Fiero-Chacayes-El Yesillo Fault System (estimated to be a 320m wide zone). No geological or geotechnical information is available in Appendix Z to assist the CNMJV with understanding how these could impact on the tunnelling conditions and construction methodology.

With Appendix Z providing no information on either faults or weak shear zones this omission results in an optimistic view of the tunnelling conditions. This is also reflected in the GSI and rock strength values of the R system they use for the compressive strength. This information is qualitative as opposed to quantitative and therefore it is impossible at the tender stage to assess the actual proportions of the Very to Very Poor ground for support purposes.

It is important at the tender stage to be able to understand how the ground will behave. The CNMJV needed this information to predict how they would control deformations under the full range of ground conditions. Without accurate geotechnical information, e.g. Unconfined Compressive Strength, modulus values and in situ

stresses, and no significant discussion about the characteristics of the rock mass, e.g. the nature, orientation and frequency of the discontinuity sets, CNMJV would find it impossible to appreciate the principal hazards and associated level of construction risks.

Most projects try and gain an understanding of the permeability of the various rock types that will be encountered. Without boreholes along the route alignment it is not possible to do this. Such testing as has been carried out has been very limited, not on the tunnel alignment and led to misleading conclusions with regards to permeability values at depth and to the possible presence of aquifers. Some of the sub-horizontal shear zones will act as aquicludes and therefore provide the conditions for aquifers to be present. The same is true for the sub-vertical to vertical bands in the Volcan Tunnel which will be confined by shear zones and at depth will give rise to very high hydrostatic pressures when encountered in the advancing headings (V1). In VA4 where the tunnel is on a down gradient water under pressure has been encountered in the invert whenever penetrating the more competent water bearing bands. These are in effect artesian conditions.

Where the ITA Guidelines for Risk Management are used on major tunnelling projects it is recommended the Owner supplies a Risk Assessment at the tender stage to the CNMJV for them to understand the risks they were accepting. More importantly, it enables an assessment of how the risks could impact their construction methodologies, programme and costs. Without a proper risk matrix identifying key residual risks the CNMJV could not properly consider the impact the risks would have on safety to personnel.

1. REPORT OBJECTIVES

This report has been requested by the CNM Joint Venture (JV) which has experienced problems with the excavation and support of both the Volcan and Alfalfal II tunnels.

The report assesses the information supplied at the tender stage (Appendix Z) and the constructions performed to date in both of these tunnels (three headings are currently being advanced). These have been excavated either by Drill & Blast (D&B) techniques or by Tunnel Boring Machine (TBM), or a combination of both.

Specifically, it looks at the problems which have been experienced in V1, VA4 and V5 and evaluates the tunnelling risks which will apply to the remainder of the construction. The assessment of the risks is based on a review of the geology, the structural geology, the design input parameters, the geological mapping of each of the headings (including photographic and video records) and a review of the geotechnical information available from the mapping in order to consider in detail the nature and contributing factors to the problems.

Based on the findings, the report summarises the level and severity of the risks which are expected to occur as each of the headings progress. In this context, the report generally follows the guidelines in the “CODE OF PRACTICE FOR RISK MANAGEMENT OF TUNNEL WORKS” prepared by the International Tunnelling Insurance Group^[1]. While the recommendations in the report are not mandatory, when applied they are important in ensuring the risks on any major tunnelling project are reduced to “as low as reasonably practicable”, i.e. ALARP.

In addition, the report also takes into consideration the recommendations on preparing Geotechnical Baseline Reports prepared by the ASCE in 2007^[2]. This is generally regarded as best practice for major projects of this type and provides excellent guidance when providing data for bidders at the Tender Stage.

To facilitate the understanding of the key aspects in the report a list of Definitions of key technical terms is provided in Appendix A.

2. INTRODUCTION

The Alto Maipo Hydroelectric Project (AM Project) is located southeast of the city of Santiago, in the municipal district of San José de Maipo, Cordillera Province, Metropolitan Region, specifically in the high Maipo River basin.

It includes the construction of two hydroelectric plants which are Alfalfal II and Las Lajas, and these are laid out in a hydraulic series and provide a combined total generating capacity of 531MW.

The Alfalfal II plant, whose powerhouse will be located underground in the Colorado River valley, opposite its confluence with the Aucayes creek, will use the waters contributed by the Yeso River and several tributaries of the Volcán River. All these waters, limited to a design flow volume of 27m³/s and associated with a gross head of approximately 1,160 m, permit the installation of a two unit plant with a capacity of about 264 MW. It is this part of the project which requires the construction by the CNM JV of two long tunnels and these are:

- The 14.1km long El Volcan Tunnel (constructed from portals V1 and V5)
- The Alfalfal II Tunnel which is scheduled to be 6.23km long (constructed from the VA4 portal). This is one section of a tunnel which in total is 17.5 km long; the balance is being constructed by another contractor under a separate contract.

The following sections review the geological and geotechnical information provided at the tender stage in Appendix Z for Contracts AM-C0610^[3] and AM-C0620^[4] (the Volcan and Alfalfal II Tunnels respectively) and assesses the actual geological and geotechnical conditions encountered in both tunnels. Because of the difficulties experienced to date, and the very slow progress using either D&B or TBM excavation methods, this report looks at the ground conditions and, more importantly, the ground behavior where problems have occurred. This is necessary in order to review the compatibility of the proposed construction methods with the ground conditions and the support systems.

This provides a basis for assessing the risks attached to both the construction methods and the safety of the works should similar problems be experienced as both tunnels are advanced.

Current progress at the time of preparing this report is as follows (5th May 2017) – Table 2.1:

TUNNEL	HEADING	LENGTH (M)	EXCAVATION METHOD
Volcan Tunnel	V1	1886.25	D&B
	V5	1090.45	D&B + TBM
Alfalfal II Tunnel	VA4	1066.40	D&B + TBM

Table 2.1 - Summary of Progress for the Volcan and Alfalfal II Tunnels

3. GEOLOGY – BASIC ENGINEERING

3.1. GENERAL

In the “Introduction” section to the geological and geotechnical information presented in both AM-C0610 and AM-C0620 it is noted *“This document presents the results of the geological studies, geotechnical analyses, hydrogeology and geological risk (mass removal, volcanics and snow) assessment performed”*.

It goes on to say in paragraph four *“In a project of this nature, the information on the geology and geotechnics of the area affected by the project’s work, in particular by underground works, is essential. Both the knowledge and characterization of the rock masses to be excavated are essential in order to assess construction methods, support types required in the concept design, and the subsequent stability analyses of openings, whether caverns, tunnels, shafts or chambers.”*

These are important statements and it goes on to say in paragraph 5 *“For this reason, the main purpose of the results of this study presented in this report is to meet the specific need of having appropriate geological knowledge and geotechnical information on the site for the subsequent definition of the geotechnical classification and estimation of the geomechanical parameters of the rock masses.”*

The intention of this part of the contract is therefore to provide the Contractor with a baseline of information with which the project can be managed. In this context, it is consistent with the aims of the suggested guidelines for preparing “Geotechnical Baseline Reports” as recommended by the ASCE. A baseline is important in ensuring that risks which are consistent with or less adverse than predicted are allocated to the Contractor. Risks which are materially more adverse than predicted are accepted by the Owner and Owner’s Engineer, and this particularly applies where there are significant changes to the support levels and support methods to meet the actual conditions. In the Contract there is a clause covering “differing ground conditions” and the intention of this clause has to be to allow the Contractor to determine when the ground conditions are materially more adverse.

The overall structural geological setting is a key element to understanding the patterns of deformation within the fold belt and how they might influence the ground conditions and ground behavior. This is not covered in a regional context in AM-C0610 and AM-C0620 but it is covered in the update of March 30th, 2015 ^[5] prepared on behalf of the client and titled “Geological Model Revision and Update”. This, and the influence on folding and fracturing, is discussed in the following section.

Later sections look in detail at what was provided to the Contractor at the time of tender in both AM-C0610 and AM-C0620 and this includes:

- Stratigraphy
- Structural Geology
- Rock Mass Quality
- In situ stress regime
- Geomechanical Properties

● Hydrogeology

3.2. STRUCTURAL GEOLOGICAL SETTING

3.2.1. Introduction

The structural geological setting of the project is a complex subject but one which is extremely important since how stratified sequences behave during tectonic movements, in this case the Nazca Plate subduction, influences strongly the deformational and, fundamentally, the geomechanical characteristics of the rock masses. Most are subjected to a number phases of sub-horizontal compression (and sometimes tension) over geological time.

On March 30th, 2015 Alto Maipo submitted an update of the information on the Volcan Tunnel titled “Geological Model Revision and Update”. The report repeats much of what is in Appendix Z but in Section 2 there is a substantial and very comprehensive presentation of the tectonic history and the structural geology of the Santiago Region including the project area (none of this information was available at the time of tender). Figure 3.1 of this report (see below) confirms that buckling and flexure of the Formations in the project area has produced open to more closed folds where the degree of tightening of the folds has often been excessive and overturning has led to thrusting, e.g. the San Ramon Thrust Fault. This is a major Fault but it should also be expected that these could occur within any of the Formations in the project area.

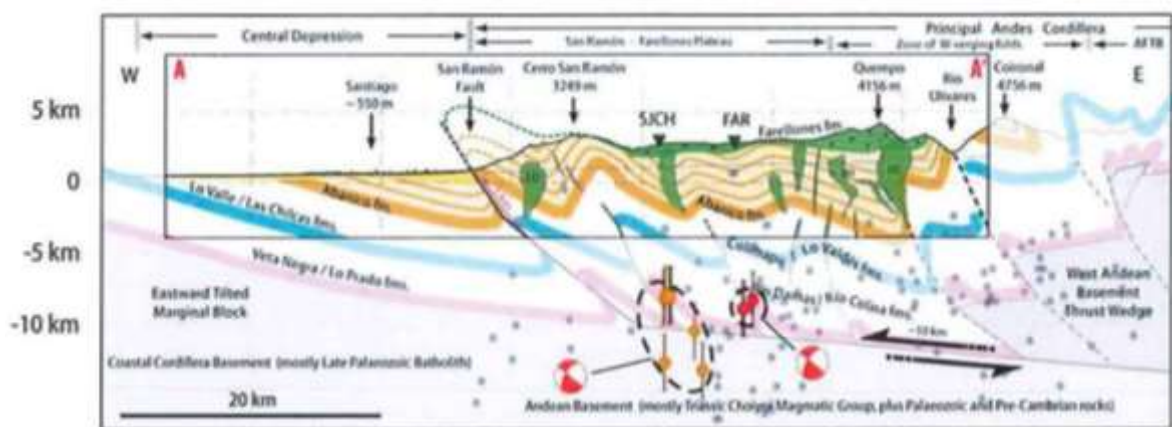


Figure 2.2.7. Major geological features along the western Principal Andes Cordillera and the detachment ramp zone that connects with the San Ramon Fault toward the surface at the eastern border of the city of Santiago (Armijo et al. 2010). The plotted beach balls highlight the dominant focal mechanism for the set of events studied (Pérez et al., 2014).

Figure 3.1 - Major geological fold and thrust features from Santiago through to the High Andes (taken from 30th March 2015 Report)

3.2.2. Project Structures

The Formations which make up the geology of the Volcan and Alfalfal II Tunnels are shown as either near vertical, or in some cases slightly overturned, or inclined and sub-horizontal (this is discussed in more detail in section 3.3.2 and 3.4.2). The strata are described in AM-0610 and AM-0620 as highly deformed but no information is provided to understand what that refers to but it is assumed to be the folding. The folding is regional in nature, i.e. large scale with a wavelength of many kilometres. The easterly compression during

plate movements has given rise to folds with a NS to NNE axial trend. The generally consistent axial trends are typical of folds produced by buckling of the strata.

The relatively flat lying sequences forming the Alfalfal II Tunnel are part of the limb of a major fold whose hinge is located to the east of the Yeso valley. This implies a fold where continued deformation has led to a flat axial plane and a major dislocation separating the flat lying limb and the fold hinge. While travelling up the Yeso Valley to the V5 portal there is evidence of major deformations on the east slopes of the valley (Fig. 3.1). This continues all the way through to the area of Las Cortaderas and the principal conclusion is that the major fault (thrust) separating the flat lying limb and the hinge passes through this area.

From V5 to the Las Cortaderas Fault the banding is flat-lying and has a gentle synclinal form. The section from V5 to Las Cortaderas is therefore is probably a remnant of the flat lying limb.



Figure 3.1 - Access road to the camp site with major disturbance of the rock mass – east side of Yeso valley south of Las Cortaderas

In effect, the project has three structural domains; the flat lying limb to the west of the Yeso Valley; the fold hinge to the east of the Valley; and a 3km section of the Volcan Tunnel from V5 to the Las Cortaderas Fault. It is likely that the effects of the fault separating the limbs will be fully encountered in V5 from the portal all the way through to beyond Las Cortaderas.

3.2.3. Deformational Features

The deformations which have taken place in each of the Formations on a project scale are the result of a combination of both folding and fracturing. The shearing and fracturing is a direct result of the folding and will impact the ground conditions and ground behavior during construction. What can be demonstrated from field observations is that displacements have occurred parallel to the strata (offsetting of sub-vertical dykes – Figure 3.2). While it is not possible to estimate the total extent of the displacements because the overall deformational history is complex, the mechanisms controlling these matter in terms of the geomechanical

properties of the different strata and therefore their response to the redistribution of stresses that can occur around the tunnel openings, particularly the high cover sections during construction (up to 1500m).

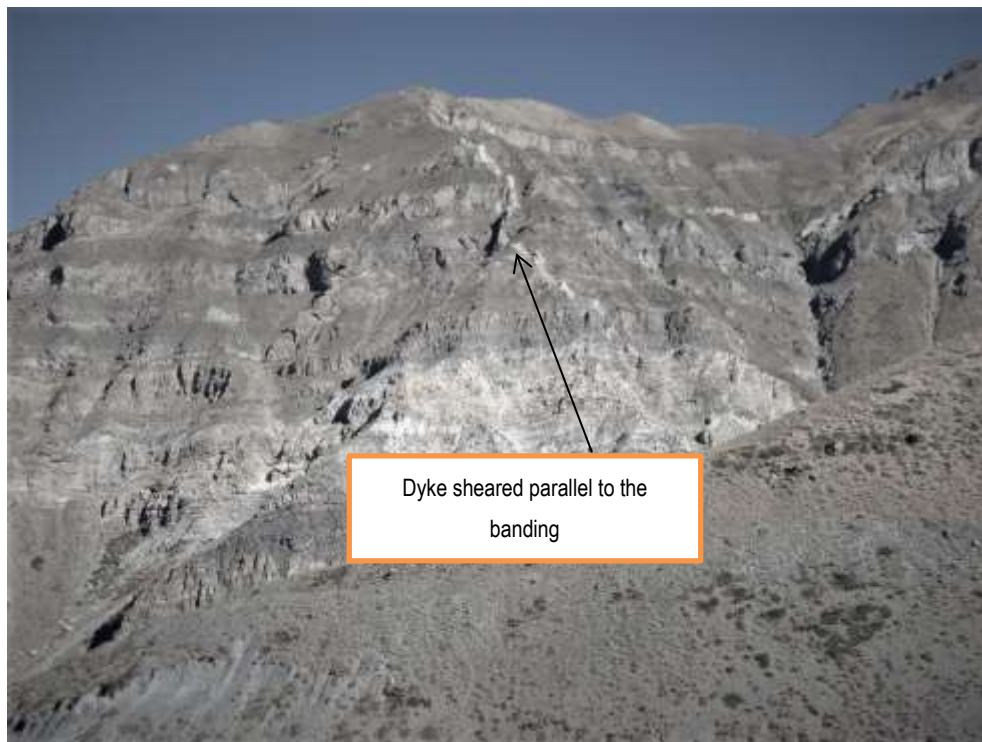


Figure 3.2 - Offsetting of the sub-vertical dyke parallel to the banding – Abanico West formation

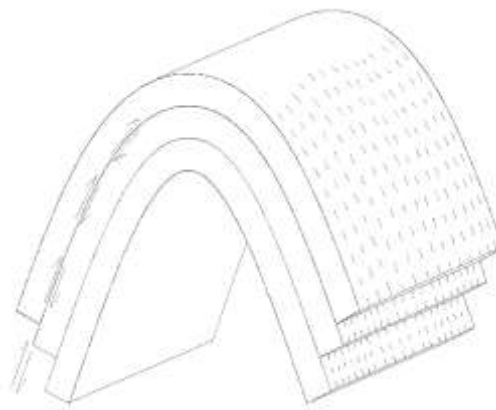


Figure 3.3 - Typical response of multilayered sequences to flexural folding (after Ramsay, 1968) ^[6]

The ground being tunnelled through comprises multilayered sequences composed of strata which are variable in composition and strength. In this context, the tuffaceous layers will not be as strong as the andesitic lavas and the pyroclastics will be weaker than the tuffs. Mostly the tuffs consist of welded and compacted rock fragments and detritus while pyroclastics are a looser accumulation of ash which is usually unworked. This will influence how the rock mass responds to buckling of the strata.

One of the consequences of this type of deformation (primarily dynamic with no or very low grade thermal effects) is flexural slip between the strata layers and the strain is always concentrated in the more deformable

strata, in this case the tuffs and pyroclastics (Figure 3.3). The Abanico West sequence demonstrates that there is pervasive shearing which affects the weaker bands and these are in effect shear zones (Appendix A).

The models in Figure 3.3 and 3.4 are simplistic but they provide a general indication of the type of deformations that can be experienced during flexural folding. The typical response is for the thrust surfaces to directly affect the weaker layers leading to significant internal deformation which can include slickensiding and cleavage as a consequence. On the flat-lying limbs of the fold (the Alfalfal II Tunnel) a slaty cleavage almost parallel to the banding is typical where folds tighten and this may be accompanied by decomposition and the formation of clay coated surfaces (see Figure 3.4a).

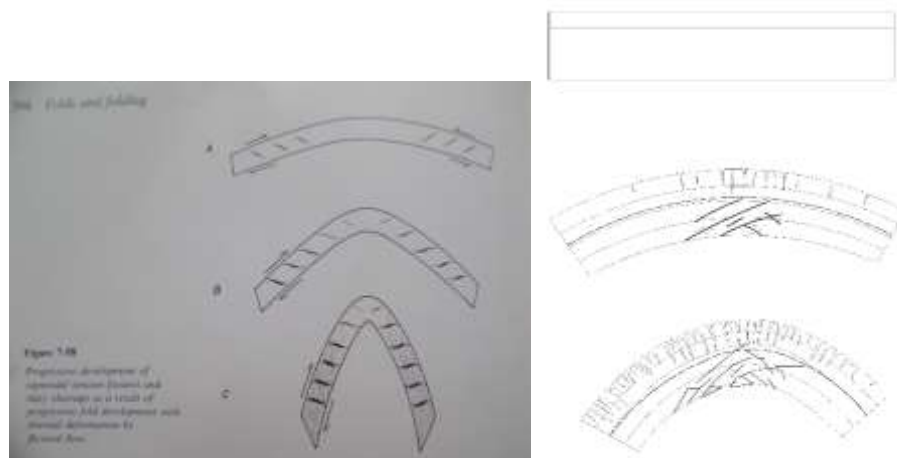


Figure 3.4 - (a) Slaty cleavage and tension cracks typical of progressive flexural fold development (after Ramsay 1968); (b) Shearing and fracture cleavage around a fold hinge

Around the hinge of the fold the structures are more complex. In the early stages of folding shearing of the weaker bands can occur around the hinge. As the folds tighten a fracture cleavage develops due to extension and tension on the outer layers and this often, with fold tightening, can develop into an axial planar cleavage (Figures 3.4b). Local individual shear surfaces can occur on the around the hinge itself where compression predominates (Fig. 3.4b).

What is evident is the importance of the thrusting and shearing of the weaker beds (shear zones) was not appreciated during the preparation of Appendix Z and therefore the impact of these zones on ground behavior could not be anticipated by the CNM JV. This issue is discussed in more detail in Section 4.



Figure 3.5 - (a) Slaty cleavage in limestones of the Lo Valdes Formation and (b) Fracture cleavage developed in the Rio Damas Formation (taken from 30th March 2015 Update).

3.3. VOLCAN TUNNEL (CONTRACT AM-C0610)

The Volcan Tunnel will be constructed through the mountain range separating the Yeso and Volcan rivers. A general description is provided which states the tunnel will be constructed through outcrops of stratified volcanic, volcanoclastic and sedimentary rocks which range in age from Upper-Jurassic to Tertiary. It is also noted the outcrops are intruded by veins, dykes and other irregularly shaped igneous bodies.

3.3.1. Stratigraphy

The Formations which will be encountered from east to west during construction include Rio Damas, Lo Valdes, Colimapu and Abanico. A summary of the main characteristics of each is given below in Table 3.1.

FORMATION	PRINCIPAL LITHOLOGIES	ESTIMATED AGE AND THICKNESS
Rio Damas	A sedimentary sequence of conglomerates and coarse to medium conglomeratic breccias interspersed with sandstones, limonite and andesitic lavas.	Upper Jurassic: 3000m
Lo Valdes	Predominantly a sedimentary sequence composed of limestones, calcilutites, lutites calcareous sandstones, conglomerates and breccias (note: lutites are fine grained and essentially composed of clay or clay sized particles). In the lower part of the sequence medium to thick bands of andesitic lavas are intercalated with thin lenticular layers of	Upper Jurassic to Lower Cretaceous: 1350m

FORMATION	PRINCIPAL LITHOLOGIES	ESTIMATED AGE AND THICKNESS
	gypsum.	
Colimapu	Composed both of sedimentary and volcanic sequences. The sediments described as predominantly sandstones and lutites, conglomerates with a sandy matrix (typically reddish in colour). The volcanics are made up of andesitic lavas and tuffs in which sandstones and secondary gypsum are present in laterally discontinuous layers.	Lower to Upper Cretaceous: 2000m
Abanico (Eastern)	The sequence is composed of tuffs which are described as volcanic breccias of predominantly violet, purple and grey colours. These are interbedded with andesitic and rhyolitic lavas and what are described as volcanoclastic rocks – essentially pyroclastics.	No age or thickness is indicated.

Table 3.1 - Formations and Principal Lithologies of the Volcan Tunnel

In addition, intrusive volcanics are represented by bedded veins (no description or composition given) and both andesitic and dacitic dykes. A small granodiorite intrusion is noted extending south of Cerro El Morado and several larger dacitic bodies have been observed in the narrow pass between Morales ravine and the El Morado stream.

Some petrographic studies have been carried out but it is not clear from the report what this refers to relative to either the main lithologies or any of the intrusives.

3.3.2. Structural Geology

Section 3 of the 30th March Report contains important new information and it revised the overall structure along the Volcan Tunnel. It also discusses rock quality (see section 3.3.3 below) and concludes generally that the Rio Damas, Lo Valdes and Colimapu Formations will provide Fair to Good Rock Quality (assumed to be based on GSI system in Appendix Z).

The discussion of the Abanico Formation is instructive in that it identifies the presence of “*cineratic and lapilli tuffs and volcanic breccias interbedded with coarse andesitic to rhyolitic lava flows*”. It generally concludes the rock quality will be Fair to Good although where there are some “*highly altered tuffaceous levels the geotechnical rock quality is Poor*”.

The discussion of alteration in the Abanico Formation is important and concludes that where it is present has occurred as a result of burial metamorphism and/or old geothermal fields. One of the primary products of

alteration is clay minerals and the report goes on to say “Many of these minerals are formed in situ, either by supergene weathering or by low temperature hydrothermal processes, which alter the aluminium silicates forming clay levels and other alteration minerals. This confers on the rocks a high degree of plasticity and dispersion.” The presence of clay is important when considering ground behavior and its impact will be discussed in more detail in Section 4. The report in reviewing tunnel construction records on other projects identifies horizons containing montmorillonite as highly problematical (incompetent) and notes “This has resulted in several construction issues (e.g. the Los Broncos mine, due north, and the Tinguiririca hydropower plants, south of the Alto Maipo area).”

The March 2015 report^[5] provides the first general assessment of the possible location of the principal fault structures which could be encountered along the alignment. This information is summarized in Table 3.2.

FORMATION	MEMBER	LITHOLOGIES	PRINCIPAL STRUCTURES
Colimapu	Kc1	Shales, siltstones, red and brown sandstones	Fractures and minor faults could occur between 4810-4930.
	Kc2	Red sandstones and grey siltstones	No data presented
	Kc3	Andesitic lavas, breccias and intercalated tuffs	Minor faults or fractures predicted to occur at the following tunnel chainages: 1700, 2170, 2300, 2320, 2560, 2650, 2850, 3130.
	Kc4	Limestones and gypsum	El Fiero-Chacayes-El Yesillo Fault System and noted to intersect the tunnel from 3580-3900m, i.e. 320m. Sub-vertical minor faults may intersect the tunnel between 4520-4640m.
Abanico	No sub-divisions of the Formations is given	Volcanics consisting of andesitic lavas, tuffs and pyroclastics. An intrusive body is predicted to occur between 6615-6900m.	Fractures and minor faults may intercept the tunnel at the following chainages: 6630, 6650, 7090, 7420, 7580, 9410, 10280, 11520m.

Table 3.2 - Principal Structural Features from March 30th 2015 Report

It is important to separate out this information into two specific types of feature:

- Minor faults and fractures. Chainages are given and it is noted these are projected to depth so the locations are approximate only. Mostly they refer to vertical to inclined rather than sub-horizontal to horizontal strata
- Major Faults. Two are predicted to occur; the first in the Colimapu Formation and referred to as the El Fiero-Chacayes-El Yesillo Fault System and the second is within the Abanico Formation and referred to as the Las Cortaderas Fault (site investigations are currently on-going by the Owner).

Section AM-C0610 provides no specific information on either of these types of features (Fig.3.6). The update rectifies this to some extent and two major fault systems are now recognized. While this may allow the CNM JV to anticipate approximately where difficult tunneling conditions could occur, there is still a considerable amount of information required to characterize these major faults.

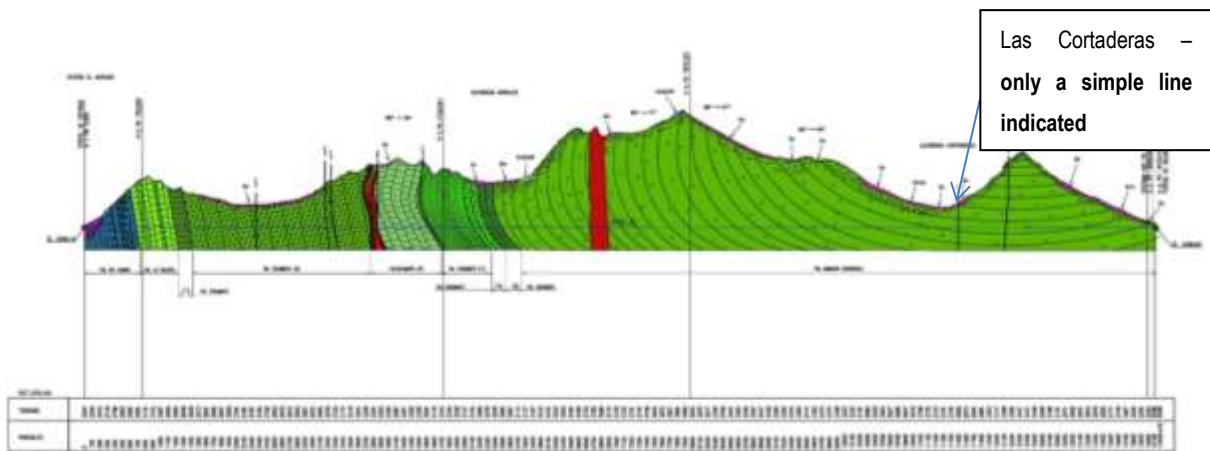


Figure 3.6 - El Volcan Tunnel Geological Profile, taken from 610-GE-PLA-002, showing the geological structure

At Las Cortaderas, the attitude of the bedding from the V1 portal to approximately chainage 11+100 is very steeply inclined and part of a major fold hinge. From approximately chainage 11+100 it is flat-lying. There is therefore a major structural dislocation at Las Cortaderas but no disturbance of the banding is shown on Fig.3.6 only a line which does not deflect the banding. There is evidence from the east side of the Yeso valley to the south of thrusting (there are large outcrops of highly disturbed steeply dipping strata). The field evidence strongly suggests this passes through Las Cortaderas and is responsible for the valley and the low cover. As an alternative, it is possible that where the banding is sub-horizontal this forms part of major rotational slide. In effect this is gravity tectonics and the active seismicity of the region is able to facilitate and trigger quite deep seated rotational movements where there are steep valley sides and the banding is in a favourable orientation. However, this is less likely even though slope failures are a regular occurrence.

More information is required to define this surface and the fault zone itself including the extent of weathering and decomposition, rock mass quality, geomechanical properties and permeabilities, including piezometric data. In particular, it is likely that the rock mass on both sides of the fault will be disturbed for some distance, possibly several hundreds of meters. What is important is that the Las Cortaderas Fault was not recognised as a major feature at the tender stage so for the CNMJV there was absolutely **no possibility** of appreciating the very difficult ground conditions this would present in terms of tunnelling. Recently borehole investigations been undertaken by the Owner to investigate this feature but to date no information has been made available to the CNMJV even though this would **normally be required as part of a risk sharing process**.

One other aspect related to the minor faults and weak shear zones which is not discussed in the update is the importance of attitude. Where they intersect the tunnel at a high angle – generally within the Rio Damas, Lo Valdes, Colimapu and part of the Abanico East Formations – and are not greater than 2-3m in width (i.e. less than one tunnel diameter) these can be managed and may well be passed through quickly. In the Abanico Formation where the strata are flat-lying for up to 3 km (from approximately chainage 11+100 to 14+100) these will always have the potential to be extremely disruptive when present in the crown of the tunnel.

3.3.3. Rock Mass Quality

On most projects every effort is made when preparing Baseline Reports at the tender stage to predict the ground conditions for tunneling purposes and rock mass rating systems such as the RMR of Bieniawski (1989) ^[7], the Q system of Barton (1974) ^[8] or the GSI system proposed by Hoek-Brown (1980) ^[9] are often used for this purpose.

Section 3.1 of AM-C0610 proposes to use the GSI system – which can be directly related to the RMR system of Bieniawski – and the classification is designed to reflect the ranges of GSI values applicable to the project. These are summarized in Table 3.3. There are limitations on the use of GSI which are discussed by Hoek et al., 1998^[10]. Given the stratified nature of the Formations and taking into consideration the principal fabric which has developed in many of the weaker bands, including the alteration and presence of clay in many of these layers, it is evident that the use of Hoek & Brown will have to be combined with specific analyses that reflect the actual ground conditions and the potential failure modes that could impact on tunnel stability during construction – this is discussed in more detail in Section 4.

GEOTECHNICAL CLASS	GEOTECHNICAL CLASS**	GSI RANGE	GEOTECHNICAL QUALITY
I	R1	76-95	Very Good
II	R2	56-75	Good
III	R3	36-55	Fair
IV	R4	16-35	Poor
V	R5	0-15	Very Poor
*Taken from Table 3.2 of AM-C0610			
**From the Feasibility Study			

Table 3.3 - GSI Based Geotechnical Classes*

The GSI values are obtained from the RMR using the equation recommended by Hoek et al, 1998:

$$GSI = RMR_{89} - 5$$

It is noted in Hoek et al., (1998) that for RMR values less than 25 the Q system should be used to determine the GSI values. No discussion is provided for how GSI values less than 25 have been obtained in AM-C0610 but it assumed the entire GSI range for Classes I to V has been determined using the above equation. For comparison purposes the R classification system used in the Feasibility Study Report has been included in Table 3.3.

While The GSI system can be used to classify the ground conditions this is not normally applied in this way since the GSI values are empirical rather than a rock mass classification system for tunnels such as the RMR of Bieniawski (1989). It would have been better practice for tunnelling purposes to use RMR to provide a baseline for the projected tunnelling conditions along the route.

AM-C0610 Section 3.2 also presents a table which provides an indication of the percentage of the rock strengths for each of the Formations using the ISRM R classification system^[11] and this is reproduced as Table 3.4. The R system in Table 3.3 is not the same as that in Table 3.4. It would have been better to avoid this potential source of confusion since uniaxial compressive strengths are included in the RMR values (Bieniawski, 1989) and therefore the GSI values also. It is not clear therefore how the data in Table 3.4 should be used and what it is meant to represent in terms of the ground conditions, especially since the data excludes local faults or sectors of severe fracture and/or alteration. The justification for excluding these important structural features is that they were not significant at a mapping scale of 1:25000. This is a major omission in AM-C0610. Without including areas of Poor or Very Poor Quality (Classes IV and V) which are the faults, intra-folial shear zones, shear zones and zones of alteration it automatically biases the data in favour of the better rock classes, i.e. it reflects an optimistic picture of the ground conditions. More importantly, it does not provide an accurate baseline on which to determine differing ground conditions.

FORMATION	UNIT	R0 0.25- 1.0*	R1 1-5	R2 5-25	R3 25- 50	R4 50- 100	R5 100- 250	R6 >250
Rio Damas						20	80	
Lo Valdes					30	70		
Colimapu	Lavas, tuffs, andesitic breccias					20	80	
	Limestones				30	70		
	Gypsum and intrusive rocks			15	15	50	20	
Abanico East	Tuffs, breccias, lava intercalations and andesitic and basaltic veins					20	80	
	Rock affected by hydrothermal				70	30		

FORMATION	UNIT	R0 0.25- 1.0*	R1 1-5	R2 5-25	R3 25- 50	R4 50- 100	R5 100- 250	R6 >250
	alteration							
	Diroritic, andesitic and dacitic porphyry					100		
	Slide Mega Blocks (from Meson Alto)				70	30		
Data taken from Table 3.3 of 610-GE-INF-001 Report (Basic Engineering)								
* Strength Values are given in MPa								

Table 3.4 - Percentages of average intact rock strengths making up each Formation

Complimentary to Table 3.3 is an estimate of the range of GSI values for each of the Formations, again excluding the most valuable information for tunnelling purposes which is the rock mass quality in major faults, local faults, shear zones and zones of alteration (Table 3.5). Also included is the range of RQD for the given GSI range. Since this is automatically considered in deriving RMR, and hence the GSI values, it does not add value.

Table 3.5 is important in that the range of GSI values is typically 30-70, i.e. in terms of the GSI Classes in Table 3.3, indicating mostly Fair to Good conditions with some of the values, i.e. <36 falling into Poor. Again, this paints an **optimistic** picture of the ground conditions and the fact that the majority of the strength values given in Table 3.4 are in the range 25-250MPa support this. Inevitably, the real problems with tunnel construction are not from GSI Classes I, II and III but IV and V. As discussed above, these will typically be major faults, local faults, shear zones and/or zones of alteration or closely spaced fracturing. Since no indication is given in AM-C0610 or the March 30th update of how wide these zones are, and there are no descriptions of the weaker more challenging zones, it would have been **impossible** at the tender stage for the CNM JV to gain any appreciation of the rock mass quality and therefore how significant these features would be in terms of tunnelling.

FORMATION/UNIT		GSI (AV)	GSI (MIN)	GSI (MAX)	RQD (AV)	RQD (MIN)	RQD (MAX)
Rio Damas		50	30	70	80	65	100
Lo Valdes		60	50	70	-	-	-
Colimapu		45	40	50	-	-	-
		55	50	60	65	50	60
Abanico – Mega Block		40	30	55	60	0	80

*RQD – Rock Quality Designation (defined as the % of cores from a borehole exceeding 100mm in length)

**Taken from Table 3.4 of AM-C0610

Table 3.5 - Rock Mass GSI/RQD* Classification**

Section 3.3 looks in more detail at both surface and sub-surface geological and geotechnical conditions. Two tables considering the sub-surface conditions are provided for information and these are presented in Tables 3.6 and 3.7.

CHAINAGES (KM)	LITHOLOGIES	FEATURES
0-0.7	Lavas, breccias, sandstones and lutites from Rio Damas Formation,	Sub-vertical strata with variable dips ranging 80° to 85° downstream of the tunnel
0.7-1.2	Limestones from Lo Valdes Formation	Sub-vertical strata with variable dips ranging 80° to 85° downstream of the tunnel
1.2-1.4	Red sandstones from Colimapu Formation	Sub-vertical strata with variable dips range 75° to 85° downstream of the tunnel
1.4-3.8	Volcanic rocks from Colimapu Formation	In this section strata dips range from 75° to 80° downstream of the tunnel
3.8-3.9	Volcanic rocks from Colimapu Formation	Sub-vertical strata with variable dips ranging 75o to 90o downstream of the tunnel. Sector limited downstream by a regional fault.
3.9-4.5	Limestones from Colimapu Formation	Strata with a strong general dip West to sub-vertical. Faults parallel to stratification
4.5-5.1	Sandstones and lutites from Colimapu Formation	Strata strongly inclined
5.1-5.3	Volcanic rocks from Colimapu Formation	Strata with a strong general dip West to sub-vertical. Faults parallel to stratification
5.3-5.5	Red sandstones from Colimapu Formation	Strata with a strong general dip

CHAINAGES (KM)	LITHOLOGIES	FEATURES
		West to sub-vertical. Faults parallel to stratification
5.5-13.2	Volcanic rocks from Abanico Formation	Strata with dips progressively changing from vertical to sub-horizontal towards the portal in the Yeso River valley
13.2-14.1	Landslide from Meson Alto	Mega block slide from Cerro Meson Alto
	Sub-vertical or strongly dipping faults downstream	At km 2.3, 3.3, 3.4, 11.7

Table 3.6 - El Volcan Tunnel Lithologies and Principal Structural Features

Table 3.6 is simply a brief description of what is presented on the generalized tunnel cross-section – Figure 3.7.

GEOLOGICAL UNIT		CLASS – SEE TABLE 4				
Formation/Lithologies		I	II (56-75) 5-25MPa	III (26-55) 25-50MPa	IV (16-35) 50-100MPa	V (0-15) 100-250MPa
Rio Damas (lavas, breccias, sandstones and lutites)			X	X	X	
Lo Valdes (limestones and marls)			X	X		
Colimapu	Lavas, tuffs and andesitic breccias			X		
	Limestones	X	X	X		
	Lutites, red sandstones		X	X	X	X
Abanico East	Tuffs, breccias, intercalations of andesitic lavas and andesitic-basaltic bedded	X	X	X	X	

GEOLOGICAL UNIT		CLASS – SEE TABLE 4				
	veins					
	KStia Unit affected by hydrothermal alteration		X	X	X	X
	dioritic, andesitic and dacitic porphyries	X	X	X	X	
	Mega Block Slide			X	X	
	Faults			X	X	X

Table 3.7 - Geotechnical Classification of the Volcan Tunnel by Geological Unit

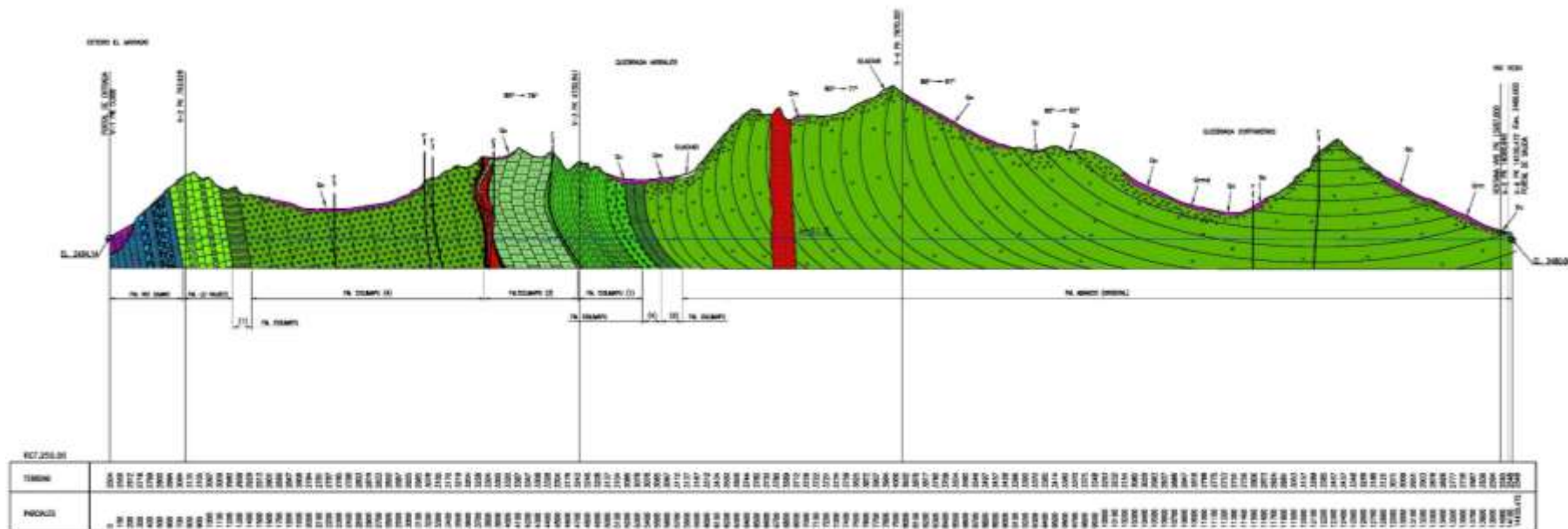


Figure 3.7 - El Volcan Tunnel Geological Profile, taken from 610-GE-PLA-002, with the geological Formations.

Of more importance is Table 3.7. For the first time in AM-C0610 the tunnel ground conditions for each of the Formations and their units are classified. In this case, all the GSI classes are included and many of the units are indicated to have classes IV and V present. However, this is qualitative only and for the CNM JV this is a problem since it is not useable for design and construction purposes. While it implies there could be faults, shear zones and alteration zones present there is no objective information to determine how many of these there are, where they are located, what their thickness and orientation is, what the composition of these zones are and how much stand-up time they would have under the full range of overburden conditions – which reaches a maximum of 1500m. This is essential information since these represent Poor to Very Poor rock mass quality for tunnelling purposes. The information also should define a baseline such that if changes occur during construction and they materially affect progress of the tunnel in terms of the construction methodology, quantities of support and safety then the Contractor has a basis for quantifying the changes. As it stands, it means that all the tunnelled sections which are in Poor and Very Poor rock mass quality represent differing ground conditions. This subject is dealt with in Sections 4 and 5 in more detail but without borehole investigations to depth to assess the ground conditions and major structural features to the maximum extent possible all of the above information is subjective.

3.3.4. Geomechanical Properties

These are fundamental to determining the ground behavior. It is considered best practice in the industry to carry out subsurface drilling to obtain representative samples of each of the rock types at tunnel elevations (especially faults and shear zones as well as intact units) and from the boreholes and borehole samples carry out as a minimum the following:

- Unconfined Compression Tests (UCS) to determine the intact strengths for each of the rock types.
- Instrumented UCS Tests to determine the intact modulus values for the main rock types.
- Triaxial tests to determine the effective shear strength parameters and Poissons ratio.
- Direct shear tests to determine the effective shear strength parameters for the principal joint sets.
- Downhole deformability tests, especially in the weaker zones, e.g. shear zones etc.
- Hydrofracturing tests to estimate the magnitude and direction of the principal stresses.
- Downhole permeability testing (see section 3.3.6).

Since no borehole investigations were carried out for preparation of AM-C0610 there is no information to allow the CNMJV to carry out analyses using appropriate software to analyse and assess the mechanisms of behavior applicable under the full range of ground conditions. Boreholes are currently being undertaken at Las Cortaderos which could provide better data but no information has been made available to date.

3.3.5. In-Situ Stress Regime

As noted above, hydrofracturing tests are often carried out to obtain an indication of the magnitude and direction of the principal stresses. This is important in active tectonic areas such as the Andes where strong sub-horizontal compression due to subduction is capable of generating high sub-horizontal compressive

stresses. Where buckling and progressive deformations occur of the type seen in the area of the Alto Maipo Project this typically results in a ratio of horizontal to vertical stresses which can be substantially greater than 1 (k value).

This matters since the maximum principal stresses concentrations around an opening are dependent on this. No discussion or data is provided about k values in AM-C0610. This is a **major deficiency** and should have been included, even qualitatively, since a lot is understood about the tectonics of the region.

Typically where there has been strong sub-horizontal compression the k-value at shallow depths (from the surface down to 1500m depth) would be in the range 2-3. This has a real bearing on the actual behavior of the ground during excavation and is discussed further in Section 4 and 5 and in the case of the sub-horizontal shear zones and faults parallel to the banding will give rise to squeezing ground conditions under high cover (in the context of the project and this report this is defined as zones where the strength-stress ratios are less than 1.0). This is discussed further in section 4.2.3.

3.3.6. Hydrogeology

Section 4.2.3 defines the Abanico, Colimapu, Lo Valdes and Rio Damas Formations as belonging to a “*Poor-permeability hydrogeological unit*”. In section 4.2.5 the impact of the El Fiero-Chacayes Fault is discussed – which is considered permeable but no mention is made of the conditions for the Las Cortaderos Fault.

No specific information is provided on what the permeabilities are for the different units in the Volcan Tunnel Formations and the conclusions in this section are based on a number of boreholes which are considered to apply to the Volcan tunnel. Of those listed in Section 4.4 most are for the Alfalfal II Tunnel area and STV-01 only is relevant to the Volcan Tunnel (however, no borehole log was made available in any of the AM-C0610 Annexes). It appears that many of the rock units where the testing was carried out, particularly the andesitic lavas, are quite highly fractured (mostly vertical jointing and some shearing) and have reasonably high k values locally, particularly at shallow depths. Since most tests were conducted at depths of up to 280m (the exception is SAM-1 which went to 350m) it is not possible to assess permeability values for the greater depths which apply to much of the Volcan Tunnel. From all of the test data (regardless of their location) some general comments are made in Appendix Z which are assumed to be relevant to the Volcan Tunnel and in summary these are:

1. Secondary porosity is the result of the presence of interconnected fractures allowing meteoric water to percolate towards the relevant rock units
2. Permeability values decrease with depth.
3. It gives general permeability values of zero to 10^{-2} cm/s for depths below the surface and down to 300m. Beyond this depth it states “*The possibility of aquifer creation is minimal*”.
4. A series of boreholes and tests were carried out for the project area and the results are suggesting that “*Below 300 m permeability is zero or, at least, does not exceed 10^{-7} cm/s*”.

These raise real concerns since some of the conclusions listed above are not justifiable. With regard to two of the points:

Point 3 states that the possibility of aquifer creation is minimal. This is **definitely not the case** and from km 11.1-14.1 the shallow synclinal banding with less permeable shear zones confining jointed and more permeable jointed andesitic lavas creates exactly the conditions for an aquifer.

Point 4 states that below 300m the permeability is zero or very low. There is **no justification** for making this statement. While depth is important and reduces the effective permeability and porosity of the rock mass, the anisotropic nature of the jointed rock and, for example, where vertical banding and/or faults are present, there will generally be the potential for the storage and direct transmission of groundwater to depth. Small but significant amounts of inflow under high pressures should have been expected and to dismiss this as a concern is suggesting that the **risk levels are very low**.

Calculating the inflows requires certain information such as the groundwater levels or the effective hydraulic head and actual permeabilities for each rock type. **None** of this information is available in AM-C0610 and this is discussed further in section 4.

3.4. ALFALFAL II TUNNEL (VA4)

The Alfalfal II Tunnel is located between the El Alfalfal and Seca valleys. The Tunnel will be mainly constructed in the Western Abanico Formation with a smaller proportion in the Farellones Formation.

3.4.1. Stratigraphy

The Abanico Formation has been grouped into four sub-units that in descending order of age are:

- Unit A (lower series) formed by tuffs and sandstones interspersed with river volcanoclastic breccias outcropping in the lower slopes of the Colorado valley
- Unit B (lower middle series) formed by volcanoclastic and pyroclastic breccias and tuff and sandstone intercalations outcropping on the upper side of the slopes of the Colorado River valley as well as on the middle sector of Aucayes and El Sauce ravines
- Unit C (upper middle series) formed by volcanic breccias outcropping on the upper middle sector of Aucayes ravine
- Unit D (upper series) formed by sandstones and lutites outcropping around the Echaurren range on the headwaters of the Aucayes ravine.

The Farellones Formation can be seen overlying Unit D at the headwaters of the Aucayes ravine. These are referred to as Unit E.

The CNM section of the Alfalfal Tunnel will be excavated in Unit C (it is possible the tunnel construction could be extended further depending on the relative progress of the 12.5km adjacent tunnel contract being undertaken by Strabag). However, for the purposes of this report the focus is only on this Unit and the associated tunnelling conditions. One of the main features of the Abanico West Formation is the presence of thick veins – up to 10's of metres in thickness – and these are common in other Units also. Sub-vertical to vertical dykes are also present.

Unit C outcrops on the middle and upper slopes of the Aucayes ravine. The basal contact with Unit B is estimated to be at elevations ranging from 2300-2500m AMSL. The units consists of thick strata of andesitic lavas, andesitic auto-breccias, agglomerates and pyroclastic breccias with thin intercalations (<20%) of tuffs and poorly sorted brecciated volcanoclastic sandstones. In addition a granodiorite stock (Las Tortolas) has been encountered and there is some limited contact metamorphism which does not appear to have resulted in metamorphism which is significantly different from the general regional metamorphism.

Table 2.1 of C0620 provides a summary of the alterations observed from petrographic analyses of a sample from the contact aureole of Las Tortolas and one, 1807-2-1 is relevant to Unit C. The rock country rock affected is described a *“Lithic-crystalline tuff”* and the alteration effects are assessed as *“Presents interstitial chlorite and a few biotite crystals intensely altered to chlorite and fine limonites. Presents juvenile fragments with slight to moderate impregnation of chlorite-limonite and dissemination of fine leucoxene crystals. Matrix with moderate development of epidote and impregnation of fine limonites (hematite-goethite)”*.

3.4.2. Structural Geology

The largest structure affecting the western sector is a large very open syncline with the strata on the limbs dipping at 5°-15° with a general NNW to NNE strikes towards the East.

Faulting, fracturing and lineaments are present throughout the area and it is noted that NE trending faults are *“particularly significant”* and have normal offsets where seen in the Aucayes ravine.

What can be observed in the Abanico West Formation is shearing parallel to the banding. This is on a large scale and is pervasive and can be seen affecting all the weaker beds where offsets of rhyolitic dykes can be observed above and below the more competent andesitic lavas (Figure 3.2).

3.4.3. Rock Mass Quality

A similar approach to the Volcan Tunnel is used for assessing rock mass quality. A number of surface observations and tests on field samples have been made in order to derive the values in the following table (Table 3.8). This is the same format as Table 3.3 for the Volcan Tunnel and therefore the comments made in section 3.2.3 also apply to this section.

Since the principal rock types belong to Unit C the focus is confined to this horizon rather than providing data for all of the Units.

GEOTECHNICAL CLASS	GEOTECHNICAL CLASS**	GSI RANGE	GEOTECHNICAL QUALITY
I	R1	76-95	Very Good
II	R2	56-75	Good
III	R3	36-55	Fair

GEOTECHNICAL CLASS	GEOTECHNICAL CLASS**	GSI RANGE	GEOTECHNICAL QUALITY
IV	R4	16-35	Poor
V	R5	0-15	Very Poor
*Taken from Table 3.2 of AM-C0620			
**From the Feasibility Study			

Table 3.8 - GSI Based Geotechnical Classes*

Table 3.9 summarises both the average levels of intact rock strength and Table 3.10 the characteristic rock mass parameters based on the framework in Table 3.8.

FORMATION	UNIT	R0 0.25- 1.0*	R1 1-5	R2 5-25	R3 25- 50	R4 50- 100	R5 100- 250	R6 >250
Abanico West	Unit C	0	0	0	0	67	33	0
Intrusive Bodies		0	0	0	0	67	33	
Faults		0	25	40	35	0	0	0
Data taken from Table 3.3 of 620-GE-INF-001 Report (Basic Engineering)								
* Strength Values are given in MPa								

Table 3.9 - Percentages of average intact rock strengths making up each Formation

FORMATION/UNIT	GSI (AV)	GSI (MIN)	GSI (MAX)	RQD (AV)	RQD (MIN)	RQD (MAX)
Abanico Unit C	55	35	90	55	0	75
Intrusive Bodies	57	40	80	74	40	100
*RQD – Rock Quality Designation (defined as the % of cores from a borehole exceeding 100mm in length)						
**Taken from Table 3.4 of AM-C0620						

Table 3.10 - Rock Mass GSI/RQD* Classification**

Table 3.9 and 3.10, as with the Volcan Tunnel, do not allow for “*areas of local faults nor with occasional sectors with severe fracturing and/or alteration*”. This also states “*The properties of any fault areas included in the geological map have been estimated based on in situ observations. These characteristics may be subject to significant changes at depth.*” Table 3.9 does contain estimates of intact strength values for faults which are in the range 1-50 MPa (R1-R3). R1 with a 1-5 MPa compressive strength range is very weak material and should be a real cause for concern. However, the above quotes from AM-C0602 provide no basis for justifying the range of values given. There are no references to any of the Annexes to further support the data and in general no characteristics are provided so in the **absence of any meaningful data or explanations** it is **not possible** to determine in any **technical** way what these “*areas of local faults nor with occasional sectors with severe fracturing and/or alteration*” and the fault areas are supposed to be or represent.

With regard to Unit C, the final paragraph of Section 3.2 states that because of the topography of the area few observations points were possible. It goes on to say “*At the observation points, intact rock presents moderate to high strength (R4 to R5) as well as medium GSI values, indicative of fair rock mass quality. In some sectors the rock locally presents intense fracturing or foliation, with a frequency exceeding 15 fractures/m and resulting in zero RQD in the direction normal to the fractures.*” This comment is very important.

What this implies is there are shear zones with very low GSI values and they are almost certainly normal to the banding. However, from the information provided this is just a qualitative comment since **no context** is provided to understand what this statement refers to in the field. In addition, no particular importance is attached to this occurrence when in fact such information is critical to understanding the range of ground behavior which could occur during tunnelling.

No information is provided in AM-C0610 on the number or characteristics of the main discontinuity sets (e.g. orientation, spacing, continuity, condition). However, Section 3.2.2 of AM-C0620 presents some information for the Alfalfal II Tunnel. For Unit C only 9 measurements of discontinuities were taken and stereoplots of this information presented. Discontinuity sets are identified and summarised in Table 3.11.

SET	DIP (°)	DIP DIRECTION (°)
1	10	100
2	85	020
3	85	273
4	70	305

Table 3.11 - Discontinuity Sets in Unit C.

With only 9 poles the relevance of this data is questionable since none of the concentrations can be regarded as statistically significant.

Section 3.3.2 discusses the sub-surface geology and geotechnics. It provides the following information on the section from VA4 portal to 12 km. The distribution of the units within the Formation is shown in Figure 3.8.

SECTION (KM)	LITHOLOGY	FEATURES
0-12.0	Unit C of the Abanico West Formation	Sub-horizontal strata. Approximately from km 9 strata dip gently upstream of the tunnel. Between km 4.7 and 7.5 abundant sub-vertical dykes and faults striking E-W
1.4-3.8	Faults	Sub-vertical faults striking N-S at km 16.7-17.7-14.3

Table 3.12 - Lithology of the Alfalfal II section from 0-12km

The information on the faults is inconsistent in that the chainages given (Features column) does not correspond to the selected section (1.4-3.8).

Table 3.13 Summarises the Estimated Geotechnical Classification for Unit C. In Table 3.14 the estimated percentages of different intact compressive rock strengths in accordance with the ISRM classification is also provided.

GEOLOGICAL UNIT	CLASS (GSI)				
	I	II	III	IV	V
Unit C Abanico Formation	X	X	X	X	
Faults			X	X	X

Table 3.13 - Classification of Geological Units to be crossed by Alfalfal II Tunnel

STRENGTH (MPA)	R0	R1	R2	R3	R4	R5	R6
% of tunnel	0	0.2	2.7	8.5	55.5	32.5	0.7

Table 3.14 - Intact Rock Classification, Alfalfal II Tunnel

The general comments about Table 3.14 in the final paragraph of Section 3.3.2 notes the following: “*there is a clear predominance of high strength rocks (R4) over almost 55% of the tunnels trace, and secondly of very high strength rocks (R5) implying that most of the tunnel alignment would cross units B and C, relating to high competent volcanic and volcanoclastic rocks. Rocks of moderate to low strength have little presence and are associated to intercalations of finer rocks and to some fault sectors*”. This paints an **optimistic** picture of the ground conditions with only 2.9% in rocks with strengths of less than 25MPa.

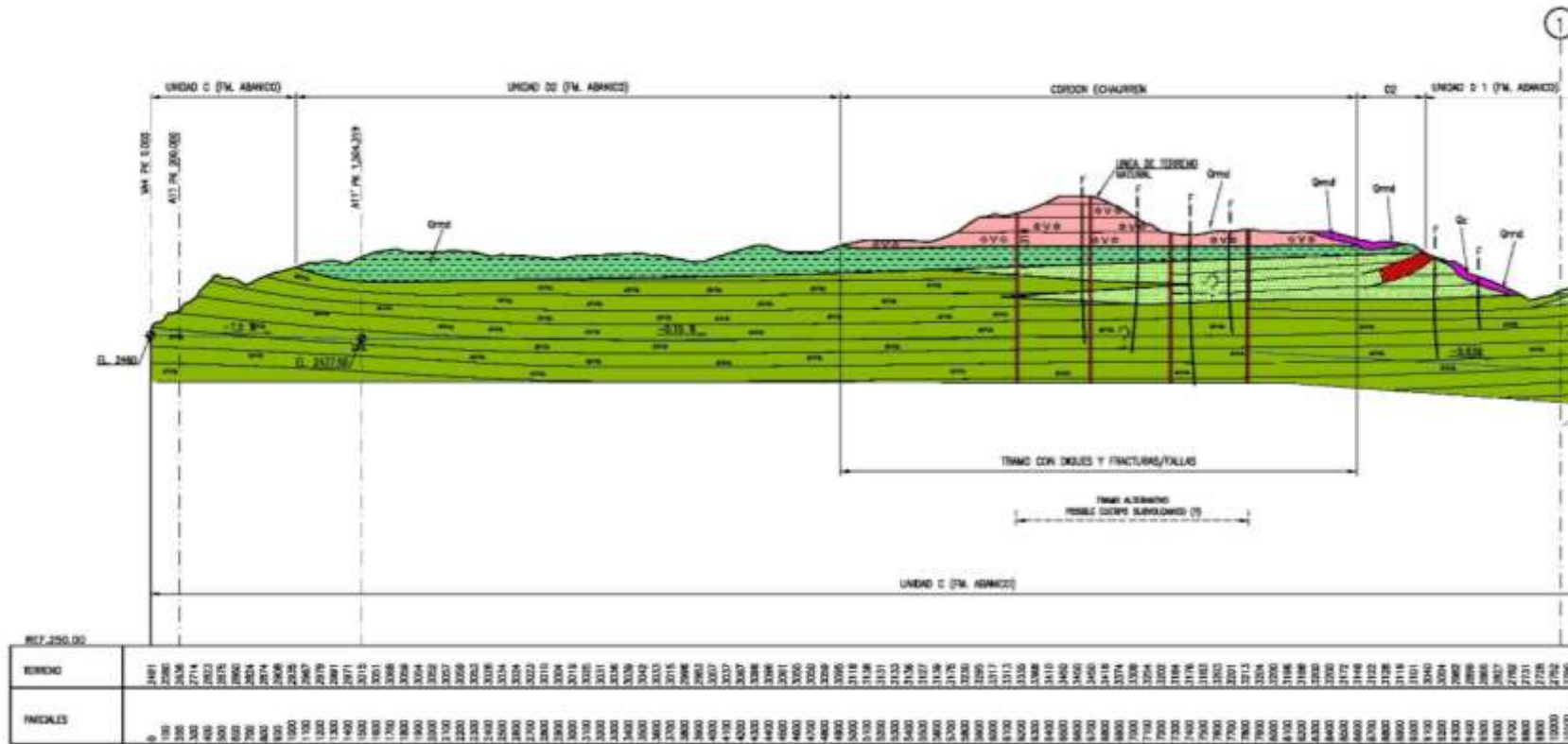


Figure 3.8 - Alfalfal II Tunnel Geological Profile, taken from 620-GE-PLA-002.

3.4.4. Geomechanical Properties

One borehole has been drilled approximately 4km south of Maitenes on the left bank of the Aucayes ravine to 350m depth (SAM-1).

This borehole does not penetrate Unit C and therefore the comments on Units A and B are not necessarily relevant. However, they do note that 43% of the lithologies are represented by Volcanoclastic Breccias (tuffs) in Unit B and 34% in Unit A. General comments on both strength and RQD indicate Fair quality rock but it is noted that two fault planes with striations (slickensides) occurred in the core at 175 and 180 m depth. It is also noted that there are some bands with RQD's ranging from 30 to 100% and with fractures/m ranging from 0-13.

Section 4.4.5 discusses the results of UCS tests and a general statement is made *"The values returned by uniaxial compressive strength tests are reasonable and within the expected values for the lithologies considered"*. It also notes that the equivalent UCS values determined from Point Load strength tests were lower than those from UCS compression tests. This is not uncommon and it is normal to derive a calibration of the Point Load tests from UCS tests although this section goes on to say that the *"UCS values are comparable to outcropping strength estimates according to ISRM system..."*

In general, this section seems uncoordinated and it is not possible to determine the full range of strengths that will actually be encountered in practice. It is expected that the samples tested in the field (the better rock types) and those easily tested from the cores has resulted in a bias towards the stronger rocks. This is certainly the case for the intact modulus, the values are summarised for SAM-1 in Table 3-15

ROCK TYPE	DEPTH (M)	INTACT MODULUS (GPA)
Volcanic Breccia	314.90-315.35	7.8
Volcanoclastic Breccia + Tuff	342.25-342.75	32.0
Volcanoclastic Breccia + Tuff	342.60-342.90	12.7
Volcanoclastic Breccia + Tuff	342.90-343.20	32.0
Tuffaceous Breccia	345.25-345.75	23.4
		21.6 (Av)

Table 3.15 - Intact modulus values (from SAM-1)

3.4.5. In Situ Stress Regime

The same comments apply to 3.4.5 as in 3.3.5 and are not repeated here.

3.4.6. Hydrogeology

Lugeon tests were carried out in SAM-1 and STA-01 and the results are summarised as:

- Boring SAM-1: Test values obtained between 250 m and 350 m generally ranged from 0 to 10^{-7} cm/s.
- Boring STA-01: Test Values obtained between 10 m and 210 m mainly ranged from 10^{-5} to 10^{-7} cm/s.

It is noted that hydraulic fracturing tests were attempted but proved inconclusive with regards to the pressure and stress field (fracturing of the borehole wall could not be achieved due to the inability of the borehole test section to reach the required pressure).

One conclusion from this statement is that the rock mass is reasonably well jointed and therefore will be permeable and obtaining a section without some form of fracturing would have been difficult. This certainly seems to be the case from observations of Unit C in the Yeso valley where offsets of dykes indicate movement has occurred in the bands adjacent to the more competent tuffs and lava flows (Figure 3.2). Inevitably there will be more brittle deformation of the tuffs and lava flows resulting in pervasive fracturing and the ability of these zones to act as aquifers.

3.5. GENERAL COMMENTS ON APPENDIX Z

The general impression from Appendix Z is that tunnelling is feasible and that no major problems should be experienced. On this basis it could be concluded at the tender stage that both D&B and TBM methods of excavation will not present major difficulties. Furthermore, with a predominance of classes R4 and R5 it could also be expected that an open face TBM could cope well with the ground conditions.

It is clear from the discussions on both the Volcan and Alfalfal II Tunnels presented above that if the shear zones and major faults are not represented in any of the tables an **optimistic** view of the tunnelling conditions is provided. It is fundamental at the tender stage that bidders can assess the effect that either sub-vertical or sub-horizontal shear zones can have on tunnel stability (i.e. these represent Poor to Very Poor rock mass quality and are critical to assessing ground behaviour). Bieniawski (1989) in particular adjusts his RMR values for the orientation of the principal rock mass fabric and where the banding is sub-horizontal this orientation is considered as no better than Fair for tunneling. Given what was known about the structural geology (section 3.2) and the current impact of shear zones on ground behavior and tunnel stability it was **absolutely necessary** these were highlighted in Appendix Z (and in any risk assessments provided to CNMJV). In other words, as a baseline of information there are many gaps in the information and these are highlighted in the relevant sections. In particular there is a lack of recognition of the importance of the structural geology and the influence that flexural folding can have on the development of shear zones. Much of the assessment, particularly for the Volcan Tunnel, is from surface mapping and observations. Using only the outcrops biases any assessment towards the better ground conditions, particularly as the shear zones and faults will be weathered and not well exposed. Boreholes are necessary to obtain objective data on poorer ground conditions and without this information it was not possible for the CNMJV to fully understand the impact these zones could have on construction.

What information is available on the geotechnics is mostly very general and qualitative in that it does not at any time attempt to apply the information to specific horizons, rock types or particular bands within any of the Formations. There is nothing presented on ground behavior. This is one of the most important considerations in determining the most appropriate construction methodology. Again, without a proper assessment of the geomechanical properties, particularly local faults and shear zones, and especially the major faults known or predicted to occur, it is extremely difficult to reach a point where a proper evaluation can be made of the most appropriate construction methodology and whether this is compatible with the expected ground behavior and

the support system available for controlling deformations. Borehole investigations, even if they do not extend to actual tunnel depths, would have provided an objective assessment of what variations could be expected and allow at least a provisional quantitative assessment of how much Poor or Very Poor ground would be encountered when the banding was either sub-horizontal or sub-vertical. The failure to do this was a **major omission** which compromised the ability of Appendix Z to be a useful baseline. Furthermore, it presented the CNMJV with not only an **optimistic** picture of the ground conditions but an **inability** to understand fully assess risks they would face during tunnelling.

4. TUNNELLING CONDITIONS

4.1. GENERAL

All the tunnel headings are under construction and progress to 5th May is as follows:

TUNNEL	HEADING	LENGTH (M)	EXCAVATION METHOD
Volcan Tunnel	V1	1886.25	D&B
	V5	1090.45	D&B + TBM
Alfalfal II Tunnel	V4	1066.40	D&B + TBM

Table 4.1 - Summary of Progress for the Volcan and Alfalfal II Tunnels

The intention of this section is to review construction related issues. The CNM JV has developed rock support systems for the expected range of ground conditions for both the Volcan and Alfalfal Tunnels. These are presented in the following drawings:

- Volcan Tunnel (V1) - 6155-TU-PLA-0011 (D&B Initial Support 6 Sheets), 6155-TU-PLA-0026 (D&B Initial Support 6 Sheets) and 6155-TU-PLA-0020 (D&B Initial Support Application Criteria).
- Volcan Tunnel (V5) - 6160-TU-PLA-0007 (D&B Initial Support 6 Sheets), 6160-TU-PLA-0008 (TBM Initial Supports 3 Sheets), 6160-TU-PLA-0010 (McNally Initial Supports 3 Sheets), 6155-TU-PLA-0015 (D&B-TBM Initial Support Application Criteria, 3 sheets) and 6160-TU-PLA-0022 (D&B Initial Support 6 Sheets).
- Alfalfal Tunnel (V4) - 6210-TU-PLA-0007 (D&B Initial Support 6 Sheets), 6210-TU-PLA-0008 (TBM Initial Supports 3 Sheets), 6210-TU-PLA-0015 (D&B-TBM Initial Support Application Criteria, 3 sheets).

Typically for Fair and better rock mass quality the support consists of combinations of rockbolts, mesh and shotcrete. Heavier support is specified where the rock mass quality is lower in Poor and Very Poor ground (IV and V). In Poor or Very Poor ground either lattice girders or steel sets are employed and the invert closed as required to provide a full ring of support (e.g. in squeezing or swelling ground conditions). This type of approach is standard for most hard rock tunnels.

4.2. VOLCAN TUNNEL

4.2.1. Tunnel V1

The tunnel from the V1 portal passes through a mixture of both sedimentary and volcanic bands. Progress to date and the main lithologies encounters are summarised in Table 4.1.

SECTION	LENGT H (M)	EXCAVATION METHODOLOGY	GEOLOGICAL UNIT	LITHOLOGY
---------	----------------	---------------------------	-----------------	-----------

-0+001.5 to 0+162.8	164.80	Mechanical	Qc/Qm d	Colluvium and mass movement deposits	Gravel and boulders in sandy matrix
0+162.82 to 0+352.5	189.7	D&B	Jrsd	Rio Damas	Andesites and andesitic breccias, calcareous shales and sandstones
0+352.5 to 1+004.01	651.5	D&B	Kilv	Lo Valdes	Gravel and boulders in sandy and clay matrix
1+004.0 to 1+186.4	182.4	D&B	Kilv/Kic	Lo Valdes/Colimapu	Andesites & Red Tuffs
1+186.4 to 1+885.25	698.8	D&B	Kic	Colimapu	Red Tuffs and Andesitic breccias

Table 4.2 - Volcan Tunnel V1 - Summary of Excavation Methods and Progress

The majority of the tunnel excavation has been through vertical or slightly overturned banding with shear zones present at the contact between the bands. Progress to date has been 1885.25 m with the first 162.8 m excavated using soft ground methods through colluvium. For the section in rock average daily progress has been 2.9m up to chainage 1+726.00.

GSI				RMR			
Classification	Range	Metres	%age	Classification	Range	Metres	%age
I	76-95	0.0	0.0	I	81-100	0.0	0.0
II	56-75	880.1	46.7	II	61-80	484.5	25.7
III	36-55	810.8	43.0	III	41-60	1132.9	60.1
IV	16-35	31.6	1.7	IV	21-440	105.0	5.6
V	0-15	163.9	8.7	V	0-20	163.9	8.7

Table 4.3 - Summary of GSI and RMR Values

SUPPORT CLASS	METRES	%AGE
I	37.4	2.0
II	626.4	33.2
III	670.9	35.6
IV	322.8	17.1
V	61.9	3.3
Soil	166.8	8.8

Table 4.4 - Rock Support Classes (combines D&B and TBM sections)

To date there have been problems with the driving of V1. This has mainly been persistent overbreak due to individual block failures (Figures 4.1 and 4.2) and water inflows. The presence of continuous discontinuity surfaces (possibly fracture cleavage – Fig.4.1b and 4.2a) and slaty cleavage in the finer grained bands define large blocks which can usually be managed during D&B – they tend to fail at the face during blasting, but might be more problematical with TBM construction depending on their geometry and size.



Figure 4.1 (a) and (b). - Overbreak in Rio Damas Sandstone due to joints parallel to tunnel axis (a) and in the Lo Valdes Formation in limestone (b)



Figure 4.2 - Overbreak in andesites (a) and limestones (b) of the Lo Valdes-Colimapu Formations



Figure 4.3 - Water inflows at chainage 1+401.8 (a) and 1+509.4 (b) in the Colimapu Formation

Water inflows have also been encountered in the tunnel and grouting to control these has been instructed by the Engineer – Figure 4.3. These inflows are significant and demonstrate that there are permeable bands of more competent rock that are acting as aquifers. This is a problem going forward when the water pressure because of the amount of cover will be substantial – 10-15 MPa over a long section (approximately 304 km). These pressures have the potential to cause face instability where the aquifers are confined by clay bearing shear zones. This problem was **not identified** as a risk in Appendix Z.

4.2.2. Tunnel V5

The portal and initial excavation of the Volcan Tunnel through colluvium was started in December 2014. The colluvium required 23.05 m of tunnelling through soft ground from the portal at station 14+127.55 to 14+104.50.

Excavation by D&B commenced from chainage 14+104.50 and continued through to 13+968.50 m a distance of 136.00 m. Most of the tunnelling was through the Abanico East Formation which comprised andesites and red tuffs (Table 4.4). During excavation, a number of sub-horizontal joints were present and these resulted in overbreak and modifications to the tunnel profile.

SECTION	LENGT H (M)	EXCAVATION METHODOLOGY	GEOLOGICAL UNIT		LITHOLOGY
14+127.5 to 14+126.5	1.0	Mechanical	Qav	Alluvial Deposits	Gravel and boulders in sandy and clay matrix
14+126.5 to 14+118.3	8.2	Mechanical	Qc+Qa v	Colluvium + Alluvial Deposits	Gravel and boulders in sandy and clay matrix
14+118.3 to 14+104.5	13.8	Mechanical	Qc+Tia	Colluvium + Abanico Formation	Gravel and boulders in sandy and clay matrix
14+104.5 to 13+968.5	136.0	D&B	Tia	Abanico Fm	Andesites & Red Tuffs
13+968.5 to 13+187.8	780.7	TBM			Red Tuffs and Andesitic breccias
13+187.8 to 13+037.1	150.7	D&B			Andesites and Red Tuffs

Table 4.4 - Volcan Tunnel - Summary of Excavation Methods and Progress

The D&B section was followed by TBM boring which began on June 15th 2015 from chainage 13+968.50 and by the 10th May 2016 it had reached chainage 13+187.80, a total of 780.57 m. In the first 15 days of boring excavation rates were reasonable up to chainage 13+880.07, in total 106.33 m of progress at a rate of about 7 m/day. From the 15th December 2015 to the 10th May 2016 a total of only 49.94 m was achieved. In effect, the

overall progress for the section from chainage 13+880.07 to 13+187.80 has been just over 2 m/day. Excavation by TBM was halted at chainage 13+187.8 and after removal excavation continued by D&B with preparation of a launch chamber for a second TBM. The overall progress rates for both D&B and TBM is 1.3m/day.

Details of the ground conditions (GSI and RMR) and rock support installed progressively during excavation and support of the tunnel are summarised in Tables 4.3 and 4.4. The GSI and RMR Tables should be the same but using the equation for calculating GSI from RMR distorts the percentages. It would be appropriate to compare the actual rock support classes encountered (Table 4.5) with those predicted in Appendix Z but there is no basis on which to do this. The closest is the percentages of rock in Table 3.4 which are derived based on the Point Load Index but are not a classification system in terms of tunnelling. Appendix Z is careful to note that the percentages in Table 3.4 do not include shear zones and faults.

GSI				RMR			
Classification	Range	Metres	%age	Classification	Range	Metres	%age
I	76-95	0.0	0.0	I	81-100	0.0	0.0
II	56-75	357.4	32.8	II	61-80	219.1	20.1
III	36-55	586.1	53.7	III	41-60	639.1	58.6
IV	16-35	136.2	12.5	IV	21-40	220.7	20.2
V	0-15	10.8	1.0	V	0-20	11.6	1.1

Table 4.5 - Summary of GSI and RMR Values

As already discussed in section 3.3.3, without an estimate of the actual percentages of ground using an appropriate classification system such as RMR or Q (using GSI is less useful) and because the information presented was selective in not taking into account fault and shear zones, the information from V5 provides the first realistic estimate of the actual support classes installed during excavation.

Table 4.5 establishes that the proportion of Poor to Very Poor ground in terms of the GSI classification system is not less than 13.5%. Because many of the RMR values are close to the boundary of the GSI values then RMR actually provides a better estimate and this indicates 21.3%.

The actual support classes installed in IV and V which can be considered as Poor to Very Poor ground conditions is 21.3%. These are high proportions and will inevitably affect progress, particularly if collapses occur – and there have been a number of cave-ins and large amounts of overbreak. There is **no possibility** of estimating from Appendix Z that the amount of Very Poor and Poor ground would be of this order of magnitude.

SUPPORT CLASS	METRES	%AGE
I	270.4	24.8
II	356.2	32.7
III	152.9	14.0

IV	197.9	18.2
V	68.9	6.3
Soil	44.1	4.0

Table 4.6 - Rock Support Classes (combines D&B and TBM sections)

Details of the ground conditions (GSI and RMR) and rock support installed progressively during excavation and support of the tunnel are summarised in Table 4.6. The GSI and RMR Tables should be the same but using the equation for calculating GSI from RMR distorts the percentages. It would be appropriate to compare the actual rock support classes encountered (Table 4.6) with those predicted in Appendix Z but there is **no basis** on which to do this. The closest is the percentages of rock in Table 3.4 which are derived based on the Point Load Index but this is not a classification system in terms of tunnelling. Appendix Z is careful to note that the percentages in Table 3.4 do not include shear zones and faults.

What is apparent from an inspection of the tunnel is that progress has been reasonable where there is Fair or better rock mass quality and the main concerns have been to support individual blocks which might be free to fall or slide into the excavation. The overbreak which has been experienced has at times been significant but the support systems installed, whether for D&B and for TBM methods of excavation, consisting of rockbolts, shotcrete and welded wire mesh have controlled single or multiple block failures – see Figure 4.1 and 4.2. However, if persistent overbreak occurs this will slow progress due to the need to prevent loosening and to ensure the safety of the excavations.



Figure 4.4 - Overbreak due to individual blocks failing at Chainage 13+438

What has been of much more concern are the sub-horizontal shear zones that halted progress at chainages 13+884 and 13+242.80. The collapse at chainage 13+884 is recorded on a video and shows in detail the ground failing in the crown above the fingershield. Approximately, 3-4 m³ of material collapsed progressively in a matter of minutes. The rock in the crown was very broken and unable to carry the loads due to the stress redistributions.



Figure 4.5 - Slabbing and block failures in the crown at Chainage 13+293

Problems also occurred at chainage 13+272.00 when it was recognized that crown stability was a problem and that the TBM fingershield was not able to deal with the broken ground. The McNally system (Figure 9) was introduced from chainage 13+255.00 to help control overbreak and reasonably sized rock fragments that were failing progressively in the crown. A video recording of checks on the depth of overbreak above the McNally rebar indicate that it is substantial, of the order of 1-2 metres.



Figure 4.6 - McNally roof support system installed immediately behind the TBM cutterhead

(The McNally system consists of reinforcing bars (rebar) temporarily supported using steel set sections and tied by welding rebar to ensure the spacing of the main supporting bars. This system has proved effective to date in controlling crown stability but meant progress was slow.)

It became evident that the sub-horizontal shear zone (fault) was reasonably thick, at least 5 m in width, and with the presence of thin clay seams and clay coated shear/fracture surfaces the shear zone was also acting as

an aquiclude. This has been facilitated by the shallow synclinal form that the banding takes in this section of the tunnel and also means the more competent jointed material above the seam has the potential both to store water and act as an aquifer. The amount of water recorded by the probe holes of 200 l/min confirmed this.

At the time of the tunnel inspection inflows of 25 l/s were still being experienced close to the face (chainage 13+120). While the water pressures normally reduce to zero close to the tunnel face when the rock mass is free draining, if the shear zone is preventing the migration of water from bands above then it will prevent the dissipation of hydrostatic pressures and can contribute to failure and collapse of the shear zone in the crown. At the point at which the failure occurred, in the D&B section for the launch chamber, approximately 20m³ of highly fractured rock collapsed (typical profile in this section with extent of overbreak shown in Figure 4.7). The amount of cover above the tunnel was 420 m.

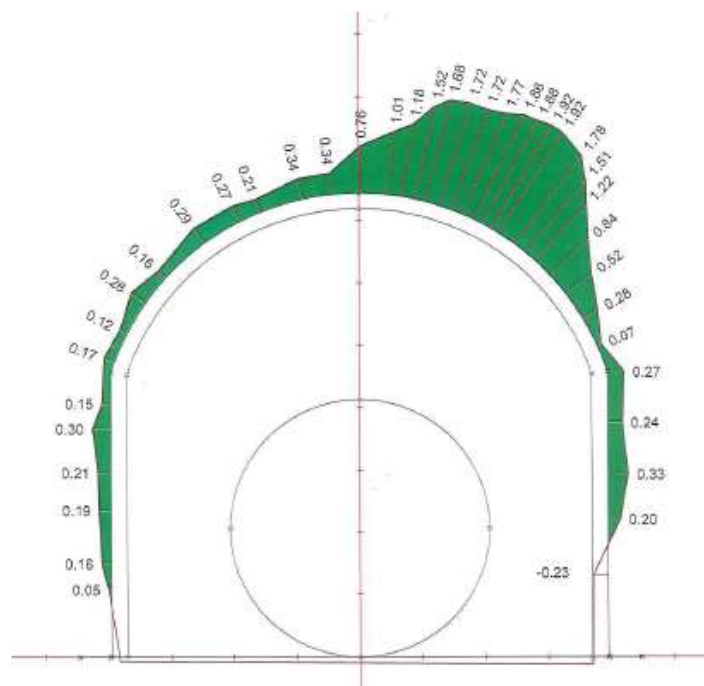


Figure 4.7 - Profile at chainage 13+120.8 showing extent of collapse (green zone)

Due to the Very Poor ground conditions (Class V) which continued to chainage 13+188 the decision was taken to remove the TBM and relocate it to the Alfalfal II tunnel. A second TBM has been procured and it is intended to continue driving the Volcan Tunnel from V5 with a refurbished Robbins open face hard rock TBM, i.e. similar to the TBM being moved to the Alfalfal II Tunnel. A launch chamber and extension of the tunnel is currently being constructed for this purpose.

The presence of sub-horizontal shear zones was not identified in Appendix Z or in the Update of 30th March 2015. The presence of these weak zones and the associated conditions are a cause for concern and they represent differing ground conditions. In particular, the orientation of the shear zones will ensure they stay in the crown for some distance unless displaced by local faults or shear surfaces. The revised geological interpretation issued in March 2015 indicates these conditions will continue at least until the Las Cortaderas Fault, i.e. a distance of approximately 3000 m. Progress is therefore going to be very slow, especially if the McNally roof support system continues to be used and will continue to deteriorate as the Las Cortaderas Fault is approached.

The exact nature of the Las Cortaderas Fault is very much an unknown at the time of preparing this report, see also section 3.3.2. Borehole investigations are on-going to try and assess what the impact will be on tunnelling but the presence of a major shear zone or fault at this location will prove substantially more difficult than the sub-horizontal shear zones, particularly if there is water under pressure present. From field observations (Figure 4. 8) it is possible the effects will be at least several hundred metres wide and probably in excess of this. Given that there was no information in Appendix Z to allow the C/NM JV to anticipate the problems this would bring these are also differing ground conditions. More importantly, it brings into question whether an open face TBM will be the most appropriate construction methodology in this section since this type of approach carries significant safety risks if crown stability is a problem.

It appears that the banding after the Las Cortaderas Fault will quickly steepen to near vertical. The shear zones will still be present and will dip into the excavation and continue to provide difficult ground conditions, especially as they will be dipping into the tunnel face. This will be compounded by the high cover which reaches 1500m and from chainage 6400 to 8400 is substantial (above 1000 m). The implications of the high cover are discussed in the following section but it should be noted that at no time was the impact of high stresses on tunnel stability discussed at the tender stage in Appendix Z.

The March 30th 2015 Update also highlight the presence of the El Fiero-Chacayes-El Yesillo Fault System. This is predicted to intersect the tunnel from chainage 3580-3900, i.e. giving a width of 320m. As with the Las Cortaderas Fault, there is no information in Appendix Z or in the 2015 Update to determine how this will impact tunnelling. As noted above, use of an open face TBM in this type of ground, which is potentially unstable, carries significant risks for an open face TBM and it is doubtful if this type of machine could cope with all of the problems likely to occur. Since this was not really addressed in Appendix Z it also represents changed ground conditions.



Figure 4.8 - Surface exposure of the thrust zone just south of Las Cortaderas

Lastly, but also important, is the possible presence of swelling minerals such as laumontite. To date this appears not to have been a problem but may occur in other sections of the Abanico Formation as the tunnel progresses.

4.2.3. Tunnelling Concerns

The problems experienced to date, principally in V5, highlight three particular issues (problems) the CNM JV has faced and one which could be faced at some stage. These are:

1. Where the strength-stress ratios are reasonably high, greater than 1, problems experienced during excavation have been related to individual blocks or narrow weak zones. There will be individual block failures and occasional crown or sidewall failures of thin shear zones but in general there will be no major problems with stability. The excavation profile can and will be affected regularly by overbreak but progressive failures should not be a major cause for concern. In these areas RMR values are likely to be in the Fair to Good range and there is sufficient stand-up time for support to be installed.
2. Where the strength-stress ratios are low, less than 1 and certainly less than 0.5, then there will be the potential for squeezing ground conditions, particularly under the high cover sections. This has been the problem where a wide sub-horizontal shear zone has been encountered in the crown of the tunnel in V5. Maintaining crown stability has been a major undertaking requiring the use of the McNally support system. Under high stress conditions and wide shear zones located in the crown this may not be effective.
3. Inflows into the tunnel in V5 indicate higher permeability values than provided in Appendix Z. More importantly, the evidence strongly indicates that the andesitic lavas and stronger tuffs are behaving as aquifers and the shear zones are confining these.
4. Where the tunnel heading encounters relatively less deformed bands, e.g. andesitic lavas which are more massive and with a high intact modulus the locked in stresses may lead to strain extension and rock bursts.

The main focus to date is on the second and third issues.

Looking at the second issue first, the following estimates the ground parameters for the major shear zone at chainage 13+120 (Table 4.7) The UCS values are essentially the limits of R3 in terms of estimating the field compressive strength using the ISRM system (although field estimates suggest lower values, from 5-25MPa). At the launch chamber location the cover is approximately 420 m.

PARAMETER	VALUE	COMMENT
K value	2-3	Probably greater than 2 but no test data to confirm
RMR	30	Derived from face mapping – considered appropriate for the ground conditions

PARAMETER	VALUE	COMMENT
GSI	25	Reduced as RMR ⁸⁹
UCS (Intact)	5-50 MPa	No data but assumed to be variable ranging from R1-R3
Em (Intact)	4 GPa	No testing so best estimate
mi	19/15	Andesite/Tuffs
D	0.2	Limited because of stiff layers above
Amount of Cover	1. 440m* 2. 500m 3. 1000m 4. 1500m	*Current amount of cover in Volcan Tunnel and covers yet to be reached by the tunnel headings

Table 4.7- Volcan Tunnel estimated ground parameters for sub-horizontal shear zones

Using the analysis developed by Hoek & Marinos (2000)^[12] and modified by Hoek and Diederichs (2006)^[13] it is possible to investigate the behavior of the collapsed zones (V5) during excavation assuming a full face of Poor quality rock (for 420 m cover). This is a simple approach which assumes a k value of 1 but for the purposes of this report is adequate to gain an understanding of the possible depths of any plastic zones. Using the values in Table 4.7 these are calculated for compressive strengths of 25 MPa and 50 MPa. With the amount of cover and 25MPa strength a large plastic zones is predicted and this will increase with increasing cover (Table 4.8).

	25MPA		50MPA	
Amount of Cover	Depth of Plastic Zone (m)	Deformations (mm)	Depth of Plastic Zone (m)	Deformations (mm)
420m	5.36	324	2.08	43

Table 4.8 - Depth of the plastic zones and predicted deformations for R3 strengths

Bearing in mind that the weak sub-horizontal bands are confined by the stronger andesitic lavas – and this is a considerable benefit – these values are considered to be on the high side unless a particularly thick weak shear zone is present. However, what the evidence suggests (video and observations of failures in the tunnels) is that squeezing and deformation of the weak zones is occurring and that overbreak and instability will be a normal part of the excavation and support cycle through these. Failures may be immediate and catastrophic for high cover weak shear zones as a result of overstressing leading to overbreak and cavitation ahead of the face. They may take longer where the shear zones are narrow. In the latter case, load transfer to the stronger confining layers will occur and as relaxation and loosening progresses in the shear zones due to stress relief there is the potential for failure, in effect cave-ins. The speed with which this process proceeds will depend on the residual effective shear strength parameters of the shear zone but if these occur close to the face they have the potential to work back behind the cutterhead. If the bands are more than several metres wide and hydrostatic pressures are present in more permeable layers above, even if support is installed, it is possible

that failure over several metres could occur due to overstressing and extend behind the advancing face if not relieved.

One of the difficulties of undertaking analyses is the need for it to be representative of the actual conditions. To obtain a picture of the ground behaviour and potential failure mechanisms requires a more sophisticated analysis using software such as FLAC 2D or 3D. It also requires information on the physical properties of the individual layers. As noted in Section 3, there is a conspicuous absence of testing because of the lack of borehole samples. It is recommended however that a back-analysis is carried out of the failure at 13+120.8 and that testing is used out to determine the detailed stratigraphy and physical properties of the bands, especially as a permeable layer of andesitic lava acting as an aquifer is present above the crown.

What should be noted is that the strength-stress ratios for the weak tuffaceous or pyroclastic bands that have undergone internal deformation will generally be less than 1.0. At the maximum cover of 1500m and assuming a k value of 2.0 this will give a maximum tangential compressive stress of approximately 75MPa. If the intact strength is 5-25MPa this would give ratios of 0.07-0.33. Barla (1995)^[14] notes these are ratios at which severe squeezing can be expected to occur (Table 4.9).

CLASSIFICATION	BARLA*(1995)	CLASSIFICATION	HOEK & MARINOS (2000)**
Non-squeezing	>1.0	Insignificant squeezing	<1%
Mildly squeezing	0.4-1.0	Minor squeezing	1-2.5%
Moderately squeezing	0.2-0.4	Severe squeezing	2.5-5%
Highly squeezing	<0.2	Very severe squeezing	5-10%
		Extreme squeezing	>10%
*Strength/stress ratio **Volume Loss			

Table 4.9 - Squeezing classifications

There is no meaningful information on convergence data to check volume losses. Hoek & Marinos (2000) would suggest that with volume losses of less than 2.5% it is likely that minor squeezing is occurring. As noted earlier, given the constraining effect of the stronger bands of andesitic lavas and tuffs the overall effect is to limit the volume loss.

At present there is no data to assess the impact of the Last Cortaderas Thrust and the El Fiero Fault systems. What is known is that Las Cortaderas is a major Fault and the rock mass conditions are likely to be Very Poor. There may well be hydrothermal alteration and large deformations due to thrusting leading to decomposition of the tuffs and the presence of clays. There will be an impact on the rock mass adjacent to the thrust for some distance away from the main thrust surface. If clay is present it raises the possibility that it will act as a barrier to flow and that water under pressure could be encountered. Therefore, depending on the actual characteristics of the Thrust zone, very difficult tunnelling conditions are likely and stability of the crown as

well as the ability to use the grippers are key concerns if the rock mass strength is very low. This generally raises questions about the ability of an open face hard rock TBM to tunnel through this area.

Little is known about the El Fiero Fault System other than it is predicted to be 320m in width. As above, this raises questions about the ability of an open face TBM to drive through this type of ground successfully. More importantly, there is every likelihood water will be confined between individual shear/fault surfaces leading to artesian pressures. These will have the potential to overstress the face lead to punching failures. This could also affect the stability of the crown, leading to cavitation.

4.3. ALFALFAL II TUNNEL

The tunnel is 6.23 km long and is being driven from the V4 Portal. To date, the tunnel has reached chainage 1+040.0, a distance of 1152.7 m. The overall length of the tunnel is 17.5 km and the balance of 11.27 km is being excavated by another contractor.

4.3.1. 4.3.1 Tunnel VA4

The lithologies encountered to date are summarised in Table 4.10 below.

SECTION	LENGTH (M)	EXCAVATION METHODOLOGY	GEOLOGICAL UNIT		LITHOLOGY
-0+026.7 to - 0+007.0	19.7	Mechanical	Qc	Colluvium	Gravel and boulders in sandy matrix
-0+007.0 to 0+043.4	50.4	D&B	Tia C	Abanico West Formation Unit C	Andesites and rhyolites
0+043.4 to 0+209.3	165.9	D&B			Metamorphic Sandstone
0+209.3 to 0+312.4	136.0	D&B			Metamorphic Sandstone/Breccia
0+312.4 to 1+040.5	781.2	D&B			Andesites, Breccias and Tuffs
1+040.5 to 1+047.7	7.2	TBM			

Table 4.10 - Alfalfal II Tunnel Excavation details

Table 4.11 summarises the ground conditions encountered to date.

GSI				RMR			
Classification	Range	Metres	%age	Classification	Range	Metres	%age

I	76-95	0.0	0.0	I	81-100	0.0	0.0
II	56-75	349.5	32.8	II	61-80	175.1	16.4
III	36-55	671.5	63.0	III	41-60	819.1	76.8
IV	16-35	16.9	1.6	IV	21-440	43.7	4.1
V	0-15	28.5	2.7	V	0-20	28.5	2.7

Table 4.11 - Summary of GSI and RMR Values for D&B Tunnelling

SUPPORT CLASS	METRES	%AGE
I	138.2	13.0
II	250.4	23.5
III	161.1	15.1
IV	481.6	45.2
V	15.8	1.5
Soil	19.4	1.8

Table 4.12 - Rock Support Classes for D&B Tunnelling

The portal was established in colluvium and following completion of this section the balance of the tunnel has been constructed using Drill & Blast techniques (D&B).

Details of the ground conditions (GSI and RMR) and rock support installed progressively during excavation and support of the tunnel are summarised in Tables 4.11 and 4.12. The GSI and RMR Tables should be the same but using the equation for calculating GSI from RMR distorts the percentages. It would be appropriate to compare the actual rock support classes encountered (Table 4.12) with those predicted in Appendix Z but there is no basis on which to do this. The closest is the percentages of rock in Table 5 which are derived based on the Point Load Index but this is not a classification system in terms of tunnelling. Appendix Z is careful to note that the percentages in Table 5 do not include shear zones and faults.

Progress has averaged 3.2 m/day but it is noticeable there is a much higher percentage of Class IV support. This is largely due to the presence of a sub-horizontal contact between andesites, breccias and tuffs (Figure 4.8). The tuffs and breccias are sheared and while no major stability problems were experienced up to chainage 0+555 they present a risk when tunnelling at depth and for the long terms stability during operation. This confirms the difficulties of the shear zones and the Owners and Engineers understanding of the problems now that construction is under way.



Figure 4.8 - Chainage 0+542.9 Sub-horizontal contact between the andesite and volcanic tuff

4.3.2. Tunnelling Concerns

The contact shown in Fig. 4.8 precipitated a collapse at chainage 0+555 (Figure 4.9). The material is very fragmented indicating these zones are fractured internally and a large amount of water is recorded as present in the collapse area. Extensive remedial works had to be implemented including lattice girders and backfill concrete to reinstate the section. This is very similar to the collapse in V5 and, although the ground conditions generally appear to be better in VA4 (it is not close to Las Cortaderas), the presence of sub-horizontal shear zones will always carry the risk of a roof collapse. This is particularly the case if they act as an aquiclude and water pressures in the bands above the crown or below the invert cannot be relieved.



Figure 4.9 - Collapse at Ch.0+555, 11th February 2016

The relatively flat-lying banding will continue throughout the length of VA4 and, depending on the thickness and how fractured the shear zones are, there is the risk of more roof instability. As noted earlier, there is no

reference to this type of occurrence in either Appendix Z or the 2105 Update so the ground conditions being experienced differ from those which could have been expected at the start of construction.

Water inflows are present in VA4. Up to 130l/min were recorded at chainage 0+211.8 (Figure 4.10) and higher inflows during the collapse at chainage 0+555. These inflows are at considerable depth and are artesian in nature since the down gradient of the tunnel is providing an external head when these are intersected (and they will regularly occur as the tunnel descends). The inflows confirm the higher permeability of the stronger and more jointed competent bands which will have a much higher storage capacity than the shear zones.

The information on water inflows from borehole tests suggested very low or zero permeability values at depth (Appendix Z). This is clearly not the case and the current inflows being experienced to date are a cause for concern and because they will exceed 10^{-7} cm/s will represent differing ground conditions. They also exceed by some margin the environmental limit of 1l/s/km. Given that other shear zones and more competent bands will be tunnelled through while excavating the remaining 5 km it should be expected that further significant water inflows under pressure will occur, especially through the invert, as the tunnel progresses. In other words, the andesitic lavas which are confined by much less permeable shear zones which often contain clay will act as aquifers.



Figure 4.10 - Chainage 0+211.8 metamorphic sandstone (inflows of 130l/min experienced at the face)

Swelling pressures and cracking of the shotcrete have been encountered between chainages 350 to 555m. Once initiated it is difficult to control and while shotcrete and rockbolts have been installed to limit the cracking which occurs, unless there is pressure applied to the ground – and this typically requires a full load bearing permanent lining – it will continue slowly. It is in the interests of the project that the Engineer provides support recommendations as soon as possible for the long term to address this condition.

4.4. COMPARISON OF PROGRESS RATES

In general, tunnelling conditions to date in V1 and VA4 have proved better than in V5 although Table 4.13 indicates there is a much higher percentage of Class IV support installed in VA4 by comparison with V1 and V5. The principal reason for this has been the contact between the andesites and a sub-horizontal shear zone which have required additional support.

The comparison therefore does not reflect the fact that progress in V1 at 2.9 m/day and VA4 at 3.2 m/day is significantly higher than in V5 (2m/day for one of the TBM sections). In this context, a comparison of the RMR values is more useful and these are given in Table 4.14. The combined total of Classes I to III in V1, VA4 and V5 are 85.8, 93.2% and 78.7% respectively. Having 6.8% and 14.3% versus 21.3% of Classes IV and V in V1, V4 and V5 respectively will inevitably result in slower rates of progress generally but especially in V5.

CLASSIFICATION	VA4* PERCENTAGES	V5* PERCENTAGES	V1* PERCENTAGES
I	13.0	24.8	2.0
II	23.5	32.7	33.2
III	15.1	14.0	35.6
IV	45.2	18.2	17.1
V	1.5	6.3	3.3
*Does not include soft ground sections			

Table 4.13 - Comparison of V4 and V5 Support Class Percentages

CLASSIFICATION	VA4 PERCENTAGES	V5 PERCENTAGES	V1 PERCENTAGES
I	0.0	0.0	0.0
II	16.4	20.1	25.7
III	76.8	58.6	60.1
IV	4.1	20.2	5.6
V	2.7	1.1	8.7

Table 4.14 - Comparison of the RMR Values for V4 and V5

Although there is a relatively high percentage of Class IV and V in V1 the vertical to sub-vertical attitude of the banding has proved less problematical. What is of more importance is the 46.7% of support Class IV and V, much of which has been directed by the client. These percentages simply reflect that fact that the ground conditions as encountered are substantially more adverse than those presented in Appendix Z.

5. DIFFERING GROUND CONDITIONS

5.1. INTRODUCTION

As discussed in Section 1, the management of any project is fundamentally determined by the geological, geotechnical and hydrogeological baseline established at the tender stage. Section 3 of this report has reviewed the information supplied in Appendix Z, and a later update issued post-contract award in March 2015.

In effect, Appendix Z is the baseline on which the clause on differing ground conditions in the contract (113.1.7) is referenced. While this is not stated explicitly, it is in effect a Geotechnical Baseline Report regardless of how it is referred to in the tender documents. The Owner and Engineer responsible for Appendix Z during the various stages of site investigation may feel that if they adopt what is often referred to as the Norwegian Model whereby the contractor is entirely responsible for the initial support (sometimes referred to as primary or temporary support) and the Owners Engineer is only responsible for the permanent support, then they have passed all responsibility for the ground conditions on to the contractor. This is manifestly not the case. The contractor when faced with the tender documents can only be expected to judge the risks based on what he is presented with. While he will use his expertise when reviewing this information they will not be in the same position as the Engineer in terms of the time and effort expended in providing a sound framework for the bidders.

As already noted when reviewing Appendix Z, there are gaps in the information supplied and much of the information presented is of a qualitative rather than a quantitative nature. The main deficiencies in terms of the information provided in both Appendix Z and the 30th March 2015 Update are summarised in Table 5.1. These are the basis on which differing ground conditions should be assessed and all those identified will impact in some form on the rock mass quality and the ground behaviour during construction. More specifically, item 3 in the Introduction to Appendix Z states *“Notwithstanding anything to the contrary in this Contract, the rock formation descriptions provided in this Section 3 of this Appendix Z, and any documents or other information referenced therein, shall be deemed to be Owner-Provided information, except for the limited purpose set forth in Section 11.3.1.7 (a)(i)(w)”*. It can be concluded from this statement that Appendix Z is a baseline of information and is intended to be used for that purpose under the contract. It also goes on to say *“Descriptions set forth in this Section 3 of this Appendix Z are based on surface mapping, and core drilling samples at some particular locations, and conditions described herein may be different at the elevation of the tunnel.”* Even if the information is from surface mapping it does not relieve the Owner of responsibility for the tunnel ground conditions simply because of a statement to that effect. There is no other information for the Contractor to understand the risks on the project.

ITEM	DESCRIPTION	COMMENTS	RELEVANT CLAUSE IN 113.1.7
1	No explanation given of the type of folding and the	Many of the features should have been identified but there is no recognition of the importance of intrafolial shearing which will result in displacements, internal deformation and fracturing of the	(z)

ITEM	DESCRIPTION	COMMENTS	RELEVANT CLAUSE IN 113.1.7
	structures typical of flexural folding	weaker tuffs and breccias. This directly affects the ground conditions and rock mass quality.	
2	Volcan Tunnel - Stratigraphy - Details	The Formations identified but no detailed descriptions given of the main rock types and their variations, including the relative proportions of the main rock types, e.g. tuffs, breccias, andesitic lavas, sediments, throughout the Formations or where swelling minerals were likely to be present.	(z) and (zz)
3	Volcan Tunnel - Discontinuities	No detailed information provided on the discontinuity sets likely to occur in each of the Formations	(z)
4	Alfalfal II Tunnel - Discontinuities	Figure 12 summarises information on discontinuities. Given only 9 measurements are used the data provided on the discontinuity sets is not statistically significant.	(z)
5	Volcan Tunnel - El Fiero-Chacayes-El Yesillo Fault System predicted to intersect the tunnel from 3580-3900m, i.e. 320m.	No information supplied in the documentation to explain the ground conditions, e.g. rock mass quality, geomechanical properties, hydrogeology.	(z)
6	Volcan Tunnel - Las Cortaderas Fault	A major Thrust/Fault surface - no indication of its presence therefore no data supplied in the documentation to explain the type of fault or the ground conditions, e.g. rock mass quality, geomechanical properties, hydrogeology. Site investigations currently under way.	(z)
7	Volcan and Alfalfal II Tunnels - Sub-horizontal shear/fault zones	These are pervasive throughout the Rio Damas, Lo Valdes, Colimapu and Abanico East and West Formations. No mention of their origin or importance in Appendix Z (they are not included in the geotechnical sections). Where the banding is flat lying they will have the potential for either squeezing ground conditions or cave-ins if not supported immediately.	(y) and (z)
8	Volcan Tunnel – Hydrogeology -	No borehole investigations were carried out for the Volcan Tunnel so no direct information on permeability values. Appendix Z states	(z)

ITEM	DESCRIPTION	COMMENTS	RELEVANT CLAUSE IN 113.1.7
	Permeabilities	that below 300m the rock will have either very low 10^{-7} cm/s or zero permeability. Given the inflows into the tunnel this information was not accurate and is misleading	
9	Volcan Tunnel – Hydrogeology - Aquifers	Appendix Z states “ <i>The possibility of aquifer creation is minimal</i> ”. Given the sub-horizontal shear zones (with clay), the presence of either shallow synclines or confined sub-vertical beds, and the jointing systems in the more competent andesitic lavas, there is the potential for aquifers to form and this comment is misleading.	(z)
10	Alfalfal II Tunnel - Hydrogeology	See items 7 and 8 – the same comments apply.	(z)
11	Volcan Tunnel - RMR Values	The GSI values are derived from the field determined RMR values. Without borehole information to depth it is impossible to determine on a quantitative basis the relative proportions of each RMR Class for tunnelling purposes. More importantly, since information on the shear zones is missing and no attempt has been made to include this in Appendix Z it is impossible therefore for the CNM JV to anticipate the main problems and reliably or better estimate the progress of each tunnel drive	
12	Alfalfal II Tunnel – RMR Values	See items 10 and 11 – the same comments apply.	(z)
13	Volcan Tunnel – Geotechnical Classification of the Geological Units	As with the RMR and GSI values these are qualitative only and provide no information on the likely percentages of each class for V1 and V5 drives. This particularly applies to Class IV and V which will provide the most challenging tunnelling conditions. Impossible therefore to develop reasonable estimates of daily progress without a reliable database.	(z)
14	Alfalfal II Tunnel – Geotechnical Classification of the Geological Units	See items 13 – the same comments apply.	(z)

ITEM	DESCRIPTION	COMMENTS	RELEVANT CLAUSE IN 113.1.7
15	Volcan Tunnel - Geomechanical Properties (1)	It is important to have reliable testing of the physical properties of the intact rock. Some data is available based on the Point Load Strength Index (PLT) using surface collected samples but because of the lack of borehole samples and Unconfined Compression Strength Tests (UCS) on intact core samples no calibration could be established for the PLT (important and the corrections values may vary from 15-24).	(z)
16	Volcan Tunnel - Geomechanical Properties (2)	No boreholes drilled on the route of the tunnel therefore no testing possible for intact UCS and Modulus (E) values – important, in combination with the RMR values when carrying out analysis for the design of the rock support systems for the expected range of ground conditions and behaviour. This particularly applies to Classes IV and V.	(z)
17	Volcan Tunnel – Geomechanical Properties (3)	Without boreholes no downhole deformability tests possible of shear zones and other weak bands to determine modulus values. Not all shear zones will have the same properties so tests should have been carried out for the full range of conditions in Class IV and V ground. While the boreholes would not go to the maximum depth necessary for sections of the alignment they would have given a proper basis for considering the properties	(z)
18	Volcan Tunnel – In Situ Stresses	The k value (ratio of horizontal to vertical stresses) critical in terms of assessing whether squeezing ground conditions will occur (along with realistic UCS and E values) for the full range of applied stresses. In thrust belts such as the Andes typically greater than 1.0 but no discussion of this issue in Appendix Z	(zz)
19	Alfalfal II Tunnel - Geomechanical Properties (1)	It is important to have reliable testing of the physical properties of the intact rock. Some data is available based on the Point Load Strength Index (PLT) and Unconfined Compression Strength Tests (UCS) on intact core samples. Only one borehole (SAM-1) relevant to the tunnel so difficult to achieve calibration for the PLT and UCS data.	(z)
20	Alfalfal II - Geomechanical	No boreholes drilled on the route of the Tunnel therefore no reliable information available on the intact UCS and Modulus (E)	(z)

ITEM	DESCRIPTION	COMMENTS	RELEVANT CLAUSE IN 113.1.7
	Properties (2)	values – important, in combination with the RMR values, when carrying out analysis for the design of the rock support systems for the expected range of ground conditions and behaviour. This particularly applies to Classes IV and V.	
21	Volcan Tunnel – Geomechanical Properties (3)	Without boreholes no downhole deformability tests possible of shear zones and other weak bands to determine modulus values. Not all shear zones will have the same properties so tests required for the full range of conditions in Class IV and V ground.	(zz)
22	Alfalfal II Tunnel – In Situ Stresses	The k value (ratio of horizontal to vertical stresses) critical in terms of assessing whether squeezing ground conditions will occur (along with realistic UCS and E values) for the full range of applied stresses. In thrust belts such as the Andes typically greater than 1.0 appendix but no discussion of this issue in Appendix Z. Hydrofracturing tests attempted but no definitive data obtained (tests could not be completed).	(zz)
23	Volcan and Alfalfal II Tunnels – Swelling	The number of bands which are liable to swelling because of the presence of laumontite or other swelling minerals not defined.	(zz)

Table 5.1 - Information Deficiencies

The following summarises the key differing conditions experienced to date based on clause 113.1.7 “Due to a Materially Differing Subsurface Conditions” clauses (y), (z) and (zz) – Table 5.2.

CLAUSE 113.1.7	ITEM	DIFFERING CONDITIONS	COMMENTS
(z)	1	El Fiero-Chacayes-El Yesillo Fault System and noted to intersect the tunnel from 3580-3900m, i.e. 320m.	No information in Appendix Z or 2015 Update characterizing these so automatically differing ground conditions
(z)	2	Las Cortaderas Fault – probably a major thrust/shear surface and likely to be 50+m	No information in Appendix Z or 2015 Update so automatically differing ground conditions. Possible

CLAUSE 113.1.7	ITEM	DIFFERING CONDITIONS	COMMENTS
		wide.	some information will be available from current site investigations – providing boreholes are planned to investigate likely position and width of the Fault System.
(z)	3	Volcan Tunnel V5 heading: sub-horizontal shear zone – chainage 13+884	Collapse in the crown of 3-4m ³ due to squeezing ground conditions (video available showing failure)
(z)	4	Volcan Tunnel V5 heading sub-horizontal shear zones – chainage 13+120	Collapse in the crown of the launch chamber of 20 m ³ due to squeezing ground conditions – Class IV/V ground.
(z)	5	Volcan Tunnel – presence of aquifers immediately above sub-horizontal shear zones	Hydrostatic pressures applied to the sub-horizontal shear zones in the crown and excessive water inflows when aquifer penetrated – permeability values significantly greater than predicted in Appendix Z.
(z)	6	Alfalfal II Tunnels – sub-horizontal shear zone	Collapse (combination of squeezing and cave-in) of the crown of 5-6 m ³ . Highly fragmented wet material – Class IV/V ground.
(z)	7	Tunnels generally	Percentages of Class IV and V ground significantly higher than could be predicted from Appendix Z - 6.8%, 14.3% and 21.3% in V1, VA4 and V5 respectively. Daily rates of progress much reduced to cope with these conditions, e.g. the McNally system.

Table 5.2 - Differing Ground Conditions

6. RISK MANAGEMENT

6.1. INTRODUCTION

It is crucial to the success of any project that the hazards and the associated risks are properly understood and their impact on progress and costs evaluated. The ITA has published guidelines on responsibilities and in general the obligations on the Client (Owner) and/or Owners Engineer for tunnel projects are as follows:

“6.3.1. Assessments and evaluations of project options should be carried out during the Project Development Stage by the Client (or on his behalf by the appointed Client's Representative). For a selected alignment or alignment options, such assessments and evaluations should take into account:

- a) the geology (including the potential for gases of a potentially harmful nature) and the hydrogeology (as characterised by site and ground investigations);
- b) tunnelling methodologies (and other methodologies as appropriate associated with works such as caverns, shafts, adits) appropriate to the nature of the ground and the environment (for example, open- and closed-face tunnel boring machines, partial face tunnelling machines (roadheaders, excavators), drill and blast) for the selected alignment or the alignment options;
- c) temporary and permanent ground support systems (for example, sprayed concrete linings, rockbolts/dowels, pre-cast concrete segmental linings, cast-iron segmental linings, cast in-situ concrete linings);
- d) ground and groundwater treatment measures (for example, the use of compressed air, grouting, dewatering/depressurisation, ground freezing) and their impact on the environment and to Third Parties (for example, groundwater abstraction/depressurisation leading to settlements, noise, vibrations);
- e) ground movements and settlements at the ground surface and their impact on a Third Party or subsurface ground movements and their impact on buried structures such as utility services, adjacent tunnels and underground structures;
- f) environmental considerations including dust, noise, vibrations, traffic, plant movements; g) associated costs, health (including occupational health considerations), safety and programme implications;
- g) appropriate forms of contract;
- h) hazardous materials including gasses, chemicals, other pollutants or naturally occurring substances that could be injurious to health or affect durability;
- i) all other particular factors relevant to the proposed project location, geology and environment.

6.3.2. The assessments and evaluations of project options shall include the identification and evaluation of associated option-related hazards and consequent risks. These shall be presented in formalised Risk Assessments for each identified project option. The Risk Assessments shall be continually reviewed and revised as appropriate during the Project Development Stage to take into account the results of site and ground investigation results and further and better information that becomes available during this Stage.”

Section 6.3.2 states that following a risk assessment process a *“formalized Risk Assessment will be presented for each identified project option”* – this is usually presented in the form of a risk register. A number of residual risks usually arise from the risk register and it is important these are passed on to the Contractor at the tender stage. The importance of the register is to assist the Contractor to reduce these risks to “as low as reasonably practicable” (the ALARP principle).

With the responsibility for design of the initial support placed with the CNMJV it is vital that he has a risk register to work with based on the Owners Project Development Stage. If this is not provided the normal risk management process is compromised for the contractor who will not be made aware of the residual risks on the project. It also forms a basis on which the Engineer and the contractor can work together to help define the permanent support.

6.2. RESIDUAL RISKS

The obligations on the Owner at the Project Development Stage are as follows:

6.4.2 A Risk Assessment shall be carried out and a Risk Register shall be prepared for the preferred project option (or options). This Risk Register should include the perceived hazards and associated risks for the preferred project option (or options) and indicate potential mitigating measures with comprehensive explanations for their basis, based on the studies carried out during the Project Development Stage. This Risk Register shall be included within the information provided to tenderers during the Construction Contract Procurement Stage.

No risk assessment or live risk register was passed to the contractor by the Owner at the tender stage. The following assessment (Table 6.1) therefore considers the main hazards retrospectively in order to identify potential concerns going forward with the remainder of the construction. This is not quantitative (no formal risk assessment has been performed) and is designed only to provide an indication of the principal residual risks being faced by the contractor. However, these are the geological, geotechnical and hydrogeological hazards that needed to be evaluated as part of the risk management process at all stages.

ITEM	HAZARD	RISK MITIGATION	RESIDUAL RISKS	COMMENTS
Geology (1)	Stratigraphical Formations not identified	Carry out sufficient surface mapping to understand regional and project geology	Low since all the principal Formations are identified	
Geology (2)	Lithological Variations not described	Perform sufficient borehole investigations and carry out surface mapping	High on both tunnels since no boreholes on the tunnel routes and no detailed profiles available on variations in rock type with depth	Appendix Z is very general and notes all the descriptions provided are based on surface mapping
Geology (3)	Structural Geology – Regional	Carry out surface mapping	Low since understood In general and covered in the March 2015 Update	The main fold structure has been identified
Geology (4)	Structural Geology – Project issues not identified	Investigate all the major and minor structural features to understand the	High as no real understanding presented of the type of folding and the related structural features	A major concern since the type of folding has strongly influenced rock mass quality

ITEM	HAZARD	RISK MITIGATION	RESIDUAL RISKS	COMMENTS
		fold mechanisms		
Geology (5)	Major Fault Zones not properly identified	Carry out both surface and sub-surface investigations using boreholes	Very high since no information available on the characteristics of the either the Las Cortaderas or El Fiero-Chacayes-El Yesillo Fault System (320m wide)	A major concern for the Volcan Tunnel but on-going investigations of the Las Cortaderas Fault may assist in providing quantitative information (no details but drilling in progress). These will always have an impact on the construction methodologies and safety
Geology (6)	Minor faults and shear zones not identified	Carry out detailed surface and sub-surface investigations	High since no interpretation and no descriptions provided of these structures in Appendix Z. The weak zones separating the stronger tuffs and lavas are all sheared and will occur regularly during tunnelling	A major concern since these will always impact on the construction methodologies, support systems, progress and safety
Geology (7)	Groundwater (1) – Inaccurate Permeability values	Perform sufficient borehole investigations to establish stable groundwater levels (GAR Table 7) and permeabilities (k values)	High since no boreholes on the tunnel alignments and therefore no downhole testing.	Some conclusions drawn from limited borehole testing in other areas (only SAM-1 relevant to the Alfalfal II Tunnel). Data not correct based on observations of tunnel inflows to date.
Geology (8)	Groundwater (2) – Presence of aquifers	Review the permeabilities and the conditions which can produce aquifers	High since no obvious understanding in Appendix Z of this issue. Shear zones will act as aquicludes and the more competent fractured beds are storing groundwater	Evidence from current tunnel drives shows that aquifers are present and tunnel inflows are relatively high by comparison with required contract inflow limits
Geology (9)	Groundwater (3) – Glacial Meltwater	Control Tunnel Inflows	High. The presence of aquifers that will contain meltwater make it impossible to control unless grouting is used extensively in areas of high inflows	A real concern and testing required to determine if present in existing inflows
Geotechnical (1)	Inaccurate GSI and RQD values	Used in absence of site and laboratory testing to provide a basis for estimating rock mass properties	High – data provided is so general that it has little relevance to actual tunnelling conditions.	The GSI values need to be applied to actual variations in rock types, to confinement at depth and particularly to poor ground such as shear zones
Geotechnical (2)	Physical Properties - Compressive Strengths	Usually obtained from laboratory Unconfined Compressive Strength (UCS) tests	Moderate – Point Load Tests (PLT) used as an alternative since no boreholes on the actual tunnel alignments.	While these can be estimated from PLT tests it usually requires UCS tests in parallel to provide a proper calibration of the PLT results. Some data provided but it is so general that it has no relevance to design and ignores the weaker horizons such as highly fractured and deformed shear zones
Geotechnical (3)	Physical Properties – Deformability (1)	Usually obtained from instrumented UCS Tests	High – no meaningful tests from rock types on the alignment, particularly the tuffs and sheared bands.	Can be obtained from GSI but this data is empirical and relies on laboratory tests for a proper calibration
Geotechnical (4)	Physical Properties – Deformability (2)	In weak bands such as minor faults and shear zones use downhole deformability	High – no downhole testing possible	The information provided ignores weak bands – and these are the most critical in terms of tunnel stability

ITEM	HAZARD	RISK MITIGATION	RESIDUAL RISKS	COMMENTS
		tests		
Geotechnical (5)	Physical Properties – Rock Bursts (3)	Use flexible support systems to control deformations close to the face – mesh, bolts and shotcrete	High - unpredictable	Depends on local factors such as how massive individual bands are and whether there are locked in stresses – face stability critical
Geotechnical (6)	In situ stress ratios (ratio of horizontal to vertical stresses - k)	Without specialised testing in either an exploratory adit or hydrofracturing in boreholes difficult to reliably estimate	High – it is really important in Poor and Very Poor ground in high cover section to properly understand the in situ stress regime and their impact on the mechanisms of behaviour	Certainly higher than 1 but because of shearing and jointing may not be greater than 2. Parametric studies possible but current evidence from relatively low cover sections in V4 and V5 indicate that squeezing ground conditions are present in weak shear zones and will become increasingly severe.
Geotechnical (7)	Rock Mass – Artesian Conditions	Carry out sufficient downhole tests to determine the range of rock mass permeability values	Moderate to high. Some general test results available but none from depth	A real concern in V4 where artesian conditions exist when intersecting aquifers in the invert under a positive pressure. May also be present close to major Faults
Geotechnical (8)	Seismic Events	Use crack control measures as necessary	High. Some hairline cracking of the shotcrete lining and small local collapses possible where support system is at limit equilibrium	Linings generally vibrate sympathetically with the ground so no significant damage expected unless a dislocation occurs due to movement on faults and/or shear zones
Design Issues (1)	Input Parameters not representative	A range of values have to be assumed but difficult for Poor and Very Poor ground	High – little representative data available for either the rock types or bands strongly affected by internal deformation	Only possible to establish during construction, particularly where Poor to Very Poor ground occurs
Design Issues (2)	Squeezing Ground	Carry out representative analyses using appropriate ground parameters and modelling of the construction process	High. Ground parameters need validating during construction to determine the full range of ground conditions.	Following validation, a best assessment of the likely range of input parameters is required to provide proper guidance on the sections where this can occur (strength-stress ratios known to be substantially less than 1.0 for the weak shear zones). Known to be present in V4 and V5 where sub-horizontal shear zones are present in the crown. A major safety concern
Design Issues (3)	Swelling Ground	Known to be present in Alfalfal II Tunnel. Observations and petrographic and chemical analyses required to determine presence. Support to be	High. Known to occur in	Often requires substantial remedial works to repair damage to linings – as has been needed in V4. A full load bearing lining required as soon as possible to control swelling potential otherwise possible safety concerns

ITEM	HAZARD	RISK MITIGATION	RESIDUAL RISKS	COMMENTS
		adjusted to reflect presence.		
Design Issues (4)	Face Instability	In the high cover sections the stress levels are high enough close to the advancing face to cause face loss and instability leading to failures in the crown	High – given the amounts of cover exceeds 1000m for much of the Volcan Tunnel (with a maximum of 1500m) and numerous shear zones are known to be present.	The principal concern is that once cavitation occurs it will extend ahead of the face and back around the front of the shield
Design Issues (5)	Block Failures	Mapping and support required where individual blocks can either slide or fall from the excavation surfaces	High but generally dealt with by the available support systems, e.g. rockbolts and shotcrete	Decisions made on site based on actual geology and applied effectively to date
Design Issues (6)	Cave-ins	Where sub-horizontal, inclined or sub-horizontal shear or fault zones occur in the crown ensure sufficient support to the roofs	High due to the frequency of shear zones	Initial squeezing of shear zones followed by relaxation and loosening. Serious risk of collapse if shear zones are wide and banding sub-horizontal. A major safety concern
Construction (1)	Drill and Blast - overbreak	Support installed progressively close to the face as heading advances	Low to moderate since relatively easier to assess the ground conditions and install required support to ensure compatibility with ground conditions.	Difficult in Poor and Very Poor ground to always predict extent. A safety concern.
Construction (2)	Tunnel Boring Machines	Open face machines OK in Fair to Good ground conditions – not so effective in Poor, Very Poor rock mass quality and major faults	High – difficult to prevent overbreak with an open face machine and substantial risk an open face machine cannot cope with shear zones and wet ground or the conditions in the Major Fault	McNally system only useful for a low cover sections where overbreak is limited. Real safety concerns with this system under high cover sections and in major fault zones, especially in low cover section of Las Cortaderas Fault
Construction (3)	Water Inflows – excessive inflows	Grouting required to control water inflows	High – lots of evidence of inflows in all tunnels	A major issue given the contract inflow limits and environmental concerns. A decision required on the right way forward.
Construction (4)	Water Inflows – excess hydrostatic pressures	Pressure relief required if aquifers present to prevent excess hydrostatic pressures above the crown	High – not always easy to anticipate presence of aquifers	Conflict of interest in that inflows exceed contract limits and drains rock mass and not be compatible with environmental concerns
Construction (4)	Water Inflows – high hydrostatic pressures	Probe ahead of the advancing face	Moderate – conditions exist in steeply dipping banding beyond Las Cortaderas Fault. More permeable beds are confined and penetration into an aquifer under pressure could result in face instability	Difficult to predict but where beds dip into the advancing face and pressures are not relieved until aquifer is penetrated. A major safety concern
Contract – Clause 113.1.7 item (z)	TBM operation	Probe drilling to investigate difficult ground conditions – always a	High. Very difficult with an open face TBM to assess the face geology. Probe drilling to anticipate difficult ground conditions time consuming and not effective if progress rates to be maintained. Probe drilling can be carried out	Impossible with a TBM to reasonably anticipate ground conditions because of limited access to the face. Problems in high cover sections can occur

ITEM	HAZARD	RISK MITIGATION	RESIDUAL RISKS	COMMENTS
		contentious issue because progress is reduced if carried out systematically	during maintenance but angle of drilling often limits useful information.	early due to squeezing and no effective means of installing support early enough to control deformations at the face.

Table 6.1 - Volcan and Alfalfal II Tunnels – Assessment of Main Hazards

6.3. RISK MANAGEMENT

Both the Volcan and Alfalfal II Tunnels will continue to experience problems and therefore both the programme and the construction methods used for advancing the tunnels requires a thorough review.

It is clear from Table 6.1 that there are a number of residual risks that need to be managed. It is difficult to mitigate all of these on any tunnelling project but an awareness of the issues and how to manage them during construction is essential.

The key residual risks are as follows:

- The presence of major faults.
- The regular and frequent occurrence of sub-horizontal, inclined, sub-vertical or vertical shear zones parallel to the banding.
- Water inflows in excess of the environmental limits.
- Aquifers due to confinement of more jointed andesitic lavas by sub-horizontal shear zones.
- Artesian groundwater conditions in Tunnel VA4
- Squeezing ground conditions in the weak sub-horizontal and sub-vertical shear zones where the strength-stress ratios are less than 1.0
- Cave-ins associated with the shear zones due to transfer of load to stronger bands and relaxation and loss of effective shear strength in the shear zones.
- Rockbursts
- Swelling ground conditions.
- Incompatibility of the construction method with ground behaviour – a serious problem for an open-face TBM in the existing and ground conditions (a thorough review of the most appropriate TBM excavation methodology required to ensure continued maximum progress and safety of all of the tunnel headings).

A major consequence of these residual risks is safety concerns relating to cave-ins in the sub-horizontal shear zones and Very Poor rock mass quality in the major faults, none of which were identified in Appendix Z. As noted in Table 6.1, the range of ground conditions, particularly as the percentage of Poor and Very Poor ground to date is much higher than could have been anticipated at the tender stage, is much wider than could

have been anticipated. This brings not only safety concerns but, in a tunnel of this size, whether the construction methodologies are fit for purpose for all conditions.

It is informative to review the risk matrix prepared by the Owner in so far as it deals with technical tunnelling issues and this was issued in October 2014, i.e. after the tender stage but before the start of construction. These are summarised below (Table 6.2):

ID	RISK CATEGORY	RISKS DESCRIPTION	LIKELIHOOD	IMPACT LEVEL	SEVERITY
6	Execution	Serious injury to personnel and/or damage/loss to equipment/materials – Contractor controlled work areas (safety)	50% - Moderate	High	Medium
12	Site	Encountered underground conditions differ materially from expected conditions (Geotechnical)	70% - Likely	Moderate	Medium
15	Site	Risk of damage due to flood inside tunnel	50% - Moderate	Moderate	Medium
16	Site	Risk of TBM stoppage due to geological conditions	50% - Moderate	High	Medium
29	Technical?	Risk of not reliable geological data due to missed geological information, rock class distribution	70% - Likely	Moderate	Medium
35	Technical?	Risk of delays due to rock burst from face or invert	50% - Moderate	Moderate	High

Table 6.2 - Owners Risk List

In terms of the residual risks identified above this list from the Owners Risk Matrix does not appear to acknowledge in detail the geological, geotechnical and hydrogeological hazards and associated risk levels that can seriously and adversely affect both progress, costs and safety on the project. In other words they are so general that it would not be possible for a Contractor to respond to these risks until well after construction had progressed.

7. CONCLUSIONS

1. The ground conditions as encountered by the CNMJV were significantly different and generally carried higher risks for tunnelling than those predicted at the tender stage (Appendix Z).
2. Appendix Z failed to represent the significance of two major structures affecting the Volcan Tunnel:
 - The Las Cortaderas Fault
 - The El Fiero-Chacayes-El Yesillo Fault System

Both of these are differing ground conditions.

3. The stratigraphy consists of regularly alternating bands of andesitic lavas and tuffs and pyroclastics. There is no information in Appendix Z to indicate the frequency, thickness or characteristics of these bands.
4. The weaker tuffaceous and pyroclastic bands have been strongly sheared as a result of flexural slip folding. Appendix Z failed to represent the significance of these zones all of which have undergone substantial internal deformation reducing both the rock mass and effective shear strength parameters of the affected bands. All of the shear zones represent differing ground conditions since their significance was never explained and they have provided a much higher percentage of Very Poor and Poor rock mass quality.
5. No boreholes were drilled on either the Alfalfal II or the Volcan Tunnel alignments. While it would have been difficult to go to the full depth of the tunnel, nevertheless, it would have provided quantitative information on the frequency, thickness and characteristics of the weak shear zones. This meant it was not possible to understand their significance and, more importantly, no samples were available for laboratory testing to determine the intact physical properties of the multi-layered sequence forming the rock mass.
6. Rock mass quality has been estimated from surface mapping using the GSI system. These values have been reduced from the RMR system of Bieniawski. Without quantitative information from boreholes and particularly the shear zones the values presented in Appendix Z are qualitative only. Appendix Z also notes shear zones and faults were excluded from their estimate of the percentages of each rock class. This is major omission which rendered it impossible for the CNMJV to use the information to fully understand their tunnelling risks.
7. The rock mass quality information without the inclusion of data which represents the poorer ground conditions biases the assessment towards the better ground conditions. This has promoted an optimistic picture of the rock mass quality for tunnelling purposes.
8. In situ stress measurements (ratio of horizontal to vertical stresses – k value) in any strongly deformed strata in thrust belts need to be either measured or estimated. With no boreholes on the both tunnel alignments it was not possible to undertake any field measurements and no information from precedent practice has been collated in order to provide some guidance (if available).

9. Without boreholes no permeability testing has been carried out to look at the relative values of the more massive bands by comparison with the shear zones. Conclusions have been drawn from data elsewhere on the project. This indicates either zero or very low permeability values and the minimal likelihood that aquifers are present. Both of these statements are not correct. The recorded inflows into the tunnel and specific inflows related to aquifers confirm significantly differing conditions. In VA4 artesian conditions have been encountered as the tunnel (on a down gradient) intersects water bearing bands.
10. It has been possible based on current progress in the tunnels to look at the relative percentages of each rock class using the RMR system. The combined total of Poor and Very Poor classes varies from 6.8% to 21.3%. These percentages are significantly higher than could have been estimated from Appendix Z. As progress is generally dependent on the rock mass quality the poorer ground conditions have to impact on both programme and costs.
11. In addition, problems have been experienced with cave-ins. These are a direct result of squeezing and subsequent relaxation of the sub-horizontal shear zones in both the VA4 and V5 headings. In one case up to 20m³ of collapsed material. These carry a high risk level in terms of severity and depending on the amount of cover and the speed with which they occur have to be considered a serious safety concern.
12. Appendix Z has numerous gaps in information and these are documented in the report. It can only be concluded that as a geotechnical baseline the report failed to provide the CNMJV with all of the information they needed to understand the hazards and associated tunnelling risks on the project.
13. Under the ITA Guidelines for Risk Management the Owner and their representatives are required to provide a Live Risk Register at the tender stage. If this was not made available to the bidders it was a significant omission which made it impossible for the CNMJV to either fully appreciate or evaluate their tunnelling risks on the project.
14. To look at the project hazards now facing the CNMJV a summary of all of these is provided in the report. A qualitative assessment is made and the residual risk levels estimated. While these should have been assessed at an early stage of the project by the Owner and his representative, this has been done retrospectively to provide a basis on which to look at the key tunnelling risks facing the CNMJV going forward.
15. The Owner prepared a risk matrix which was presented to CNMJV in 22nd October 2014. While items 29 and 35 are geotechnical in nature there is no evidence that most of the hazards listed in Table 6.1 have been identified or assessed in accordance with the ITA guidelines.
16. Of real concern for CNMJV are the differing ground conditions at Las Cortaderas and the El Fiero Fault systems. These will challenge any TBM operation, especially an open faced gripper TBM because of the Very Poor rock mass quality which can be expected. These zones will be at least several hundreds of meters wide and give rise to hydrostatic pressures. This is a very difficult combination of conditions and there will be an impact on safety, programme and costs.
17. As tunnelling progresses in both the Alfalfal II and Volcan Tunnels the amount of cover will increase. The squeezing potential of the sub-horizontal shear zones will increase. The stress concentrations could result in strength-stress ratios of 0.3 or lower depending on the actual rock mass compressive

strength. If there are shear zones several metres or more in width in the crown these will be subject to overstressing and failure may take place rapidly. This will cause extensive overbreak and more importantly is a serious safety concern since any failure could potentially work back behind the cutterhead to personnel working areas.

18. The general conclusion from the review of Appendix Z and the actual tunnelling conditions (more than 4km has been constructed) is that the CNMJV were not in a position at the tender stage to properly understand the full extent of the risks facing them during construction. These included the following areas: construction methodologies, programme, cost and safety.

DB Powell

A handwritten signature in black ink that reads "DB Powell". The signature is written in a cursive, flowing style.

18.04.17

8. REFERENCES

- [1] ITA “A CODE OF PRACTICE FOR RISK MANAGEMENT OF TUNNEL WORKS” 2006.
- [2] ASCE “Geotechnical Baseline Reports for Construction – Suggested Guidelines”, 2007.
- [3] Appendix Z “GEOLOGY-GEOTECHNICS FOR THE UNDERGROUND WORKS IN CONTRACT AM-C0610”. (18-12-08) – Doc Ref 610-GE-INF-001
- [4] Appendix Z “GEOLOGY-GEOTECHNICS FOR THE UNDERGROUND WORKS IN CONTRACT AM-C0620”. (18-12-08) – Doc Ref 620-GE-INF-001
- [5] “GEOLOGICAL MODEL REVISION AND UPDATEALTO MAIPO HYDROELECTRIC PROJECT”, Preliminary Report, Volcan Tunnel. DEPARTAMENTO DE GEOLOGÍA FACULTAD DE CIENCIAS FÍSICAS Y MATEMÁTICAS, UNIVERSIDAD DE CHILE, March 2015.
- [6] “Folding and Fracturing of Rocks”, JG Ramsay, 1969. McGraw-Hill.
- [7] Bieniawski Z.T., “Engineering rock mass classifications”, 1989. Wiley: New York.
- [8] Barton N.R., Lien R & Lunde J., “Engineering classification of rock masses for the design of tunnel support.” *Rock Mechanics* 6(4), pp 189-239 (1974)
- [9] Hoek & Brown, “Empirical strength criterion for rock masses”. 1980. J. Geotech. Engng Div., ASCE 106(GT9), 1013-1035.
- [10] Hoek E, Kaiser P.K. & Bawden W.F., “Support of Underground Excavations in Hard Rock”. A.A. Balkema (1998).rock
- [11] International Society for Rock Mechanics “Rock characterisation, testing and monitoring – ISRM suggested methods, 1981. Oxford, Pergamon.
- [12] Hoek & Marinos, 2000a,b. “Predicting tunnel squeezing problems in weak heterogeneous rock masses”. Tunnels and Tunnelling International. Part 1 November; Part 2 December.
- [13] Hoek & Diedrichs, 2006. Empirical estimation of rock mass modulus. Int. J. Rock Mech. & Mining Sciences.
- [14] Barla, G. 1995. “Squeezing rocks in tunnels”. ISRM News JII4449.
- [15] Farmer, I. 1983. “Engineering behaviour of Rocks”. Chapman & Hall Limited.

APPENDICES

APPENDIX A – DEFINITIONS

APPENDIX A – DEFINITIONS

To help with understanding the content of the report definitions are provided below with some further explanations where necessary:

Andesites - an extrusive igneous, volcanic rock, of intermediate composition. The mineral assemblage is typically dominated by plagioclase plus pyroxene or hornblende.

Aquifer - an underground layer of water-bearing permeable rock (typically jointed volcanic rocks such as andesitic lavas and tuffs)

Aquiclude – impermeable layers of rock that form barriers to the flow of water, e.g. any of the shear zones containing clay and fine grained sheared material. Where these isolate a permeable layer the aquifer is said to be confined.

Artesian Pressures – related to a confined aquifer containing groundwater under positive pressure

Buckling – Competent layers in a multilayered sequence typically control the folding and generate classic rounded buckle folds (buckling); the less competent layers accommodate this type of folding by deforming internally

Engineer – SAM Joint Venture

Fingershield – steel support to control and contain overbreak at the cutterhead of a TBM

FLAC - numerical modeling software for advanced geotechnical analysis of rock, groundwater and ground support in two dimensions

Fracture Cleavage – typically develops around the outer layers of fold hinge due to tension but becomes pervasive as the fold tightens; sometimes termed axial planar cleavage in brittle layers where it is sub-parallel to the axial plane

Flexural Folding – Flexural slip allows folding of competent strata by creating layer-parallel slip between the layers. The fold formed by the compression of competent rock beds is referred to as a "flexure or flexural slip fold".

Hazard – Any event whether positive or negative that can affect an outcome

Hydrostatic Pressures – water under pressure due to the piezometric head and weight of water in the same zone of saturation

McNally System – a system of 8mm rebar combined with partial steel sets to control overbreak in the crown of a tunnel immediately behind the cutterhead

Overbreak – loss of material around the perimeter of a tunnel excavation due to the failure of the rock mass or individual blocks (generally in the crown)

Owner – Alto Maipo SpA

Permanent Support – the final support installed to meet the operational and functional requirements of the project

Permeability – Coefficient of permeability – hydraulic gradient expressed as the ratio of the fluid driving head and the flow path length (Farmer, 1983) [15]

Porosity – Ratio in a rock mass of the volume of voids to the combined volume of voids (Vv) and solids (Vs) (Farmer, 1983)

Primary Support – the initial support installed in a tunnel heading to maintain stability

Pyroclastics - pyroclastic deposits are commonly formed from airborne ash, lapilli and bombs or blocks ejected from the volcano itself, mixed in with shattered country rock

Residual Risk – risks which cannot be mitigated at a particularly project stage and which have to be addressed and reduced to as low as reasonably practical at a later stage

Rhyolites – Fine grained equivalent of a granitic igneous rock

RMR - Rock Mass Rating system – in the report refers to Bieniawski's classification system (1989); this is a system developed to classify rock masses for tunnelling purposes. The system separates the results into 5 different classes Very Good (80-100), Good (60-80), Fair (40-60), Poor (20-40) and Very Poor (0-20). The values in parentheses are derived from 5 parameters (1. UCS strength; 2. RQD; Spacing of Discontinuities; 4. Condition of discontinuities; 5. Groundwater inflows). The total is then adjusted for the effect of discontinuity orientation.

Shear Zone – A shear zone is a zone of strong deformation (with a high strain rate) surrounded by rocks with a lower state of finite strain (where brittle fracturing predominates)

Slaty Cleavage – fabric developed in fine grained rocks subject to strong internal deformation

Slickensiding – polished and striated surfaces formed as a result of shearing (friction) along a thrust or shear surface

Sub-horizontal – any surface or layer which is inclined at an angle of less than 15o

Sub-vertical - any surface or layer which is inclined at angle of 15o or less to the vertical

Synclinal - a geometrical form (structure) where the beds dip inwards towards the centre of a structure (usually a fold hinge)

Squeezing Ground – ground where the rock strength-stress ratio is less than 1.0

Tuffs – rocks formed of compacted volcanic fragments generally smaller than 4mm in diameter

Volcanic Breccias – a more or less indurated pyroclastic rock consisting principally of accessory and angular fragments (32mm or more in diameter) lying in a fine tuff matrix.