NATIONAL UNIVERSITY OF ENGINEERING

COLLEGE OF GEOLOGICAL, MINING AND METALLURGICAL ENGINEERING

COMPARISON OF METHODOLOGIES OF ANALYSIS AND DESIGN OF TUNNEL SUPPORTS, RMR AND Q. APPLICATION TO *SAN FRANCISCO* TUNNEL

Subject: GEOLOGY APPLIED TO CONSTRUCTIONS – GE821

Students: **Acuña Huillca Ben**

Cruz Flores Yoana

Haquima Ramos Anderson

Ríos Salas Angelo

Adviser: Graciela Gonzales Pacheco

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ABSTRACT

In underground excavations the geotechnical assessment of stability is fundamental. When an underground excavation is carried out, the redistribution of stresses originates, which correspond to the lithostatic overload, tectonic stresses and induced stresses caused by nearby excavations, which is why the stability of this should be guaranteed, which will include a geological evaluation. detail geotechnical that goes from the study of prefeasibility, feasibility, final design, operation and closure.

The families of discontinuities and the boards in some cases will be intercepted forming wedges and blocks which will tend to fall which could be contained by placing the appropriate and timely support.

Currently there are several geomechanical classifications that include support for underground excavations, two of which are the subject of this project.

CHAPTER I

INTRODUCTION

1. GENERAL

When an underground excavation is carried out, the redistribution of the efforts is originated which corresponds to the Lithostatic overload, tectonic tensions and induced tensions caused by nearby excavations, so that the stability of this must be guaranteed and will include a Geological-geotechnical assessment of detail ranging from the pre-feasibility, feasibility, final design, operation and closure studies.

The families of discontinuities and the joints in some cases are intercepted forming wedges and blocks which will tend to Drop it to be achieved by placing the appropriate and timely support.

There are currently several geomechanical classifications which include support for underground excavations, two of them being the subject of this project.

2. STATE OF THE ART OR BIBLIOGRAPHIC REVIEW/ANTECEDENTS.

Carlos Enrique Thomas Cabrera (2014) carried out his thesis on Civil engineering, which aimed at the study between the support requirements and the fortification of tunnels defined according to empirical methods of geomechanical classification versus methods Analytical and numerical.

The results of the support requirement for a horseshoe-type tunnel of 10 mtx10 are presented. Mt Approximately 90 MT2 obtained by empirical methods RMR, Q and Rmi The analytical method of stability with the software Unwedge and numerical methods of finite elements Phase 2d, in fractured rocky massifs to very fractured, dry and low to intermediate efforts, with fault mechanisms controlled by the families of discontinuities and gravity. The conclusion of the thesis is Thomas Cabrera 2014 "The results indicate that empirical methodologies are more sensitive to block volume parameters than to joints quality, that the Q method proposes lower support requirements and that the safety factors obtained with Unwedge Drastically increase with the use of Shotcrete"(p. 131)

3. APPROACH OF PROBLEM

Bieniawsky (2011) In "The last issue of Tunnels & Tunnelling International (February of 2011) enumerates not less than 41 large breaks of tunnels. The study of these cases, two of which occurred in Spain, shows that more than 85% were produced by unexpected geotechnical conditions and interpretative errors "(Pg 2).

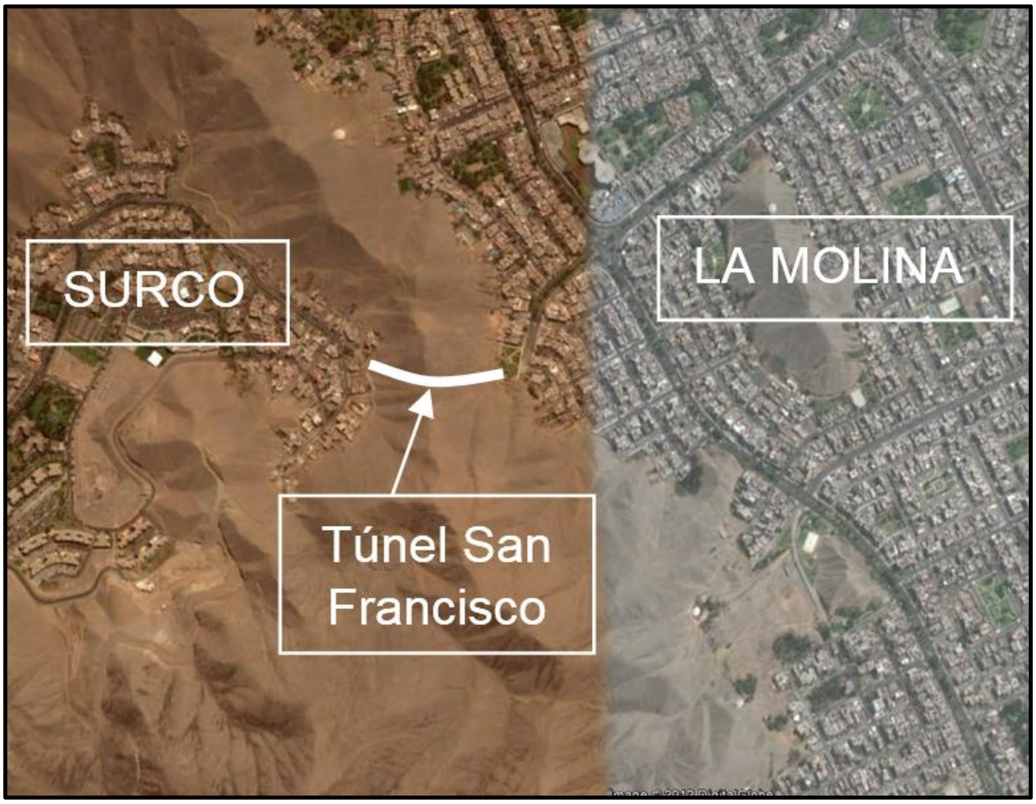
To design a tunnel should avoid choosing only a single design method, it is advisable to use the empirical method RMR or Q because there is a great experience Practice on Historical cases and the analytical method with the numerical methods of the computer and the observational method during the construction or the new Austrian method NMA with the measures of convergence and deformations.

The ratings Geomechanics RMR and Q are suitable for tunnel support design as they are Made Geological-geotechnical cartography and all data information is collected in the field, which allows us to quantify the conditions of the rocky massif and determine the most suitable sustenance. Also, these classifications are constantly being updated as the substitution of the fiber's mesh. The RMR and Q are applicable to both good quality solids and poor quality.

The project is located in the central part of the Peruvian coast in the Lima Region, between the districts of La Molina and Surco, at an altitude between 245 and 355 meters above sea level between the geographic coordinates of NORTH: 8660700 - 8661200 and ESTE: 287400 - 288,000 corresponding to the hills (witness hills in the middle of the alluvial cone of Lima) bordering the foothills of the Western Cordillera.

The rocky massif is formed by intrusive igneous rocks of the Batolito de la Costa formed by granites, granodiorites and diorites with undifferentiated contacts, of faneritic texture, holocristalina, hipidiomórfica of grain medium to thin and volcanic rocks (andesitas) of gray color of afanítica texture and Porphyritic, these rocks are covered mostly by residual soil of little thickness, the foot of the slopes is covered by colluvial deposits, the streams are covered by blocks and angular fragments fallen from the high parts with few fines.

Figure 1. Location of the San Francisco tunnel



4. JUSTIFICATION

The study would be carried out because in the rocky massifs the stability of the underground excavations is controlled by the planes of discontinuities and the gravity that causes the fall of block of rocks, therefore it is necessary the maintenance.

The project is Done because the tunnel would traverse two different lithologies; Andesitic Rocks and intrusive rocks, with certain structural conditions and with small to medium tensions, will allow us to compare the support between the RMR and Q classifications.

The study will allow us to anticipate a preliminary support in terms of the type and quantity of maintenance in tunnels of dimensions and similar conditions of lithologies, structural and stresses.

5. OBJECTIVE

- To evaluate the probable support to be placed in a tunnel using two RMR empirical geomechanical classifications (BIENIAWSKY) and Q (BARTON) and compare their results.

Specific objectives

- Sustaining a tunnel with the application of the Geomechanical classification Rmr
- Sustaining a tunnel with the application of the Geomechanical classification Q
- Analyze the results of the two methods.
- Explain the differences in case they were very different

6. HYPOTHESIS

With both methods the results will be similar.

CHAPTER II

THEORETICAL FOUNDATIONS/CONCEPTUAL FRAMEWORK

For the construction of tunnels it is essential to make the classifications of the Rocky massif, to determine the design of the support, which will be applied in the tunnel project that crosses the San Francisco Hill, located between the districts of the Molina and Santiago de Surco, this project has been developed in the same area where the research project called "Analysis of stability in the construction of the tunnel in the San Francisco hill" presented to the Research Department of the Faculty, prepared by teachers; Nora Revolle Alvarez, Esteban Manrique Zuñiga and Graciela Gonzales Pacheco in the year 2012 and this year the students of the geology course applied to the constructions have developed their field practice enriching the data and information.

This rocky outcrop is formed by intrusive and extrusive magmatic rocks which will be crossed by the tunnel in mention and according to these geomechanical classifications it will be suggested the possible support to guarantee stability and the non-occurrence of Fatal accidents, the field work is an empirical methodology and have assumed values of some parameters for the possible design, so there must be differences between the projected design and the installed since the investigations should be carried out Corresponding to this engineering work.

Key words

Geomechanical classifications RMR, Q, San Francisco, sustaining

1. THEORETICAL FRAMEWORK.

The support refers to the structural elements of subjection of the land, applied immediately after the excavation of the Tunnel In order to secure their Stability during the construction and after it, as well as guarantee the safety conditions

The coating is placed after the support and consists of applying a layer of concrete or other structural elements, in order to proportional long-term resistance to the tunnel and give a regular finish, improving its Functionality (Aerodynamic conditions, impermeability, luminosity, and to ensure the aesthetics of the project).

2. PROBLEMS WITH EXCAVATION IN FRACTURED MASSIFS

In relatively superficial tunnels and excavations it is very common to drop blocks from the ceiling and walls. These blocks are formed by the intersection of geological structures.

Types of blocks:

I. Key or Critical block (critical block): It would fall with great probability in the absence of reinforcement.

II. Potential key BLOCK: Removable But with high probability of being subject in place by friction.

III. Safe removable block: safe under gravitational conditions.

IV. "Sharp" BLOCK: You cannot move without pushing your neighbors.

V. Infinity BLOCK: Has face Free But by its extension it is not removable.

VI. Fracture BLOCK: No tieNE face free in digging.

Type I, II and III blocks are removable, type IV, V and VI are non-removable.

3. STABILITY OF UNDERGROUND EXCAVATIONS

Miguel Angel Berrocal Mallqui October 2015 Mining Safety

The engineer Miguel Angel Berrocal Mallqui, author of the book "stability of Underground excavations", presents practical guidelines for measuring the tensions in situ of the Rocky massif using the method of detonation of drills (DT).

This procedure provides, in real time, the direction of the main voltage, the parameter k, and the value of the tensions, according to the interpretation of the elliptical orientation of the major main stress.

This knowledge is necessary to design sections and forms of underground excavations Freestanding On the contour of the cutting line, such as: trunk, horseshoe and circular, adapted to the stresses in situ to the excavation.

Also, by interpreting the cutting line, unstable rock operating or artificial wedges can be identified and controlled. In any scenario, and in cases of instability of the underground excavations, the whole of the support can be directed towards that of major main tension.

This will maintain the balance between the support and the tensions of the place, before an excavation in the contour of the line of cut, for each condition of massive rock or structural.

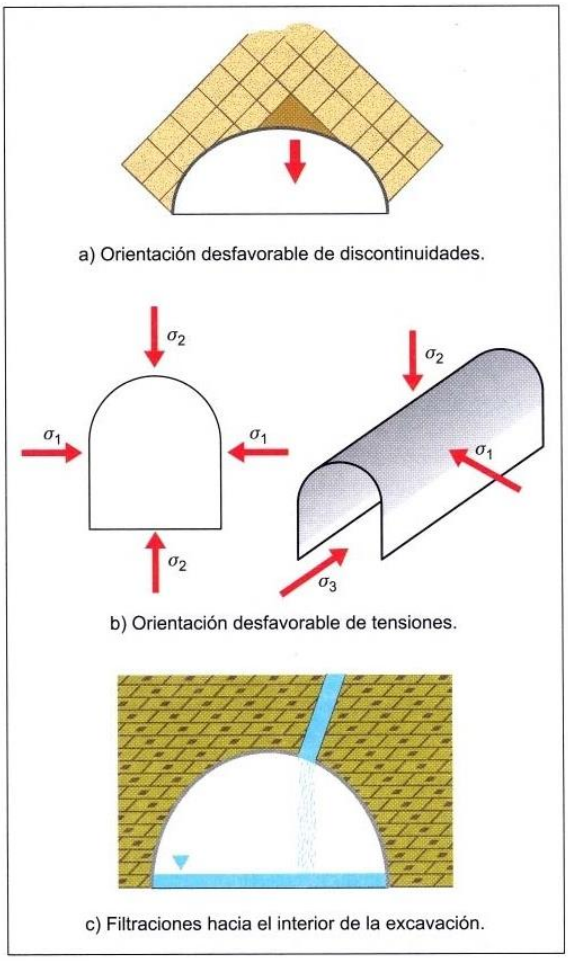


Figure 2. Influence of the efforts in the tunnels.

4. CLASSIFICATION ROAD ACCORDING TO THE MTC'S ROAD MANUAL

First Class roads. Are roads with an IMDA Between 4,000 and 2,001 veh/day With A Road of Two Rails at least 3.60 m wide. can have crossings or vehicular steps at level and In urban areas it is advisable to have pedestrian bridges or in the absence of road safety devices, That allow for operating speeds, More secure. The rolling surface of these Roads must be paved.

CHAPTER III

METHODOLOGY OR ENGINEERING PROCESS ACCORDING TO THE NATURE OF THE RESEARCH

1. DETERMINATION OF THE RMR (BIENIAWSKI, 1979)

Developed by Z. T. Bieniawski during the years 1972-73, and has been modified in 1976 and 1979, based on more than 300 real cases of tunnels, caverns, slopes and foundations. To determine the RMR index of rock quality is made use of the six parameters of the terrain, which is listed below:

- Resistance to simple compression of the material
- RQD (Rock Quality Designation)
- Spacing of discontinuities
- discontinuities status
- Presence of water
- Orientation of discontinuities

The RMR is obtained as a sum of scores corresponding to the values of each of the six parameters listed (table 1).

To then The table of quality scales of the RMR classification is presented.

Table 1. Determination of the RMR (Bieniawski, 1979)

1	Strength of intact rock material	Point load strength index	>100 Kp/cm^2	40-80 Kp/cm^2	20-40 Kp/cm^2	10-20 Kp/cm^2	For this low range uniaxial compressive test is preferred		
		Uniaxial comp. strength	>2500 Kp/cm^2	1000-2500 Kp/cm^2	500-1000 Kp/cm^2	250-500 Kp/cm^2	50-250	10-50	<10
	Rating		15	12	7	4	2	1	0
2	RQD		90%-100%	75%-90%	50%-75%	25%-50%	<25%		
	Rating		20	15	10	8	5		
3	Spacing of discontinuities		>2m	0.6-2m	0.2-0.6m	0.06-0.2m	<0.06m		
	Rating		20	15	10	8	5		
4	Conditions of discontinuities	Discontinuity length	<1m	1-3m	3-10m	10-20m	>20m		
		Rating	6	4	2	1	0		
		Separation	None	<0.1mm	0.1-1.0mm	1-5mm	>5mm		
		Rating	6	5	3	1	0		

		Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensides
		Rating	6	5	3	1	0
		Infilling	None	Hard filling<5mm	Hard filling >5mm	Soft filling <5mm	Soft filling >5mm
		Rating	6	4	2	2	0
		weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
		Rating	6	5	3	1	0
5	Ground water	Inflow per 10m tunnel length (l/m)	Nulo	<10	10-25	25-125	>125
		Relacion Joint wáter press/major principalσ	0	0.0-0.1	0.1-0.2	0.2-0.5	0.5
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing
	Rating	20	15	10	8	5	

Rating adjustment for discontinuity orientations

Strike and dip orientations		Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-2.5	-50	-60

Rock Mass Classes determined from total ratings

Class number	I	II	III	IV	V
Description	Very good rock	good rock	Fair rock	Poor rock	Very poor rock
Rating	100-81	80-61	60-41	40-21	<20

Meaning of rock classes

Class number	I	II	III	IV	V
Average stand-up time	10 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass	>4 Kp/cm^2	3-4 Kp/cm^2	2-3 Kp/cm^2	1-2 Kp/cm^2	<1 Kp/cm^2
Friction angle of rock mass	>45°	35°-45°	25°-35°	15°-25°	<15°

Effect of discontinuity strike and dip orientation in tunneling

Strike perpendicular to tunnel axis				Strike parallel to tunnel axis		Dip 0°-20° irrespective of strike
Drive with dip		Drive against dip				
Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	Dip 45°-90°	Dip 20°-45°	
Very favourable	Favourable	Fair	Unfavourable	Very favourable	Fair	Unfavourable

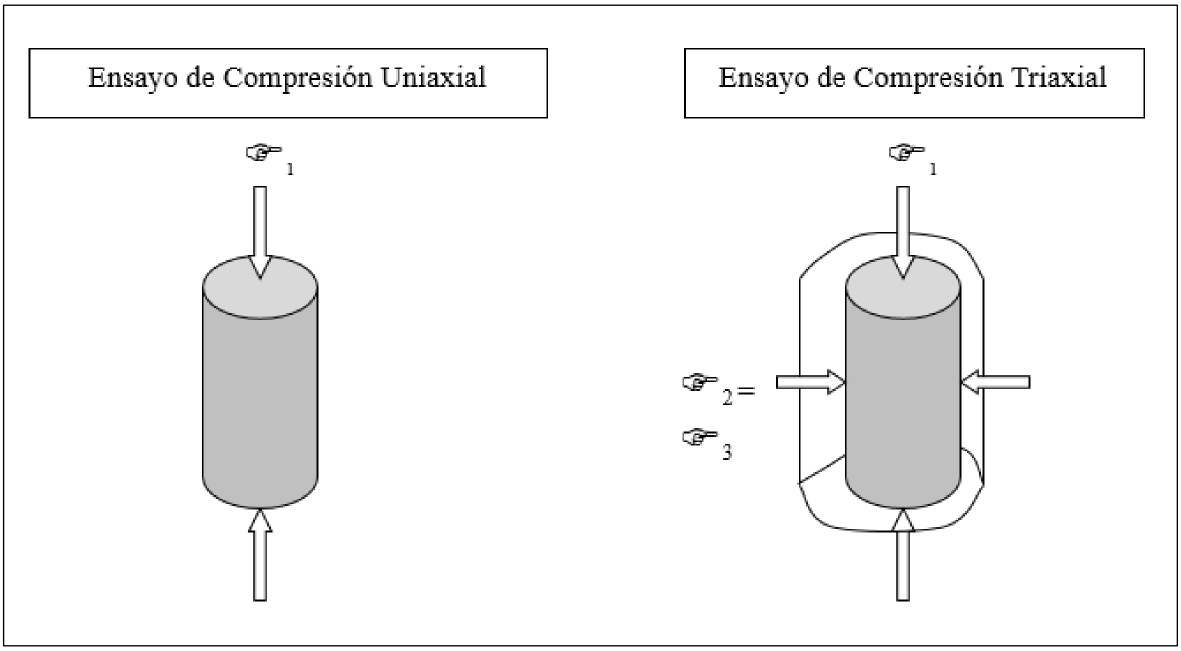
2. RESISTANCE OF THE ROCK MATRIX

They allow to determine the resistance to compression (elastic limit) and the deformation of the rocky matrix as the efforts are applied. (Fig. 3)

Laboratory tests = > resistance test to compression Simple or uniaxial.

Triaxial assay (lateral confinement by pressure fluid).

Figure 3 Resistance of the Rocky matrix



Geological factors That affect these trials:

a) Lithology.

b) Grain size and cementation (sedimentary rocks)

c) Porosity.

d) Microcracking

e) Degree of weathering or alteration

The assessment of the first parameter, Rock matrix Resistance intact, depends on the method used.

Table 2. Rock matrix Resistance

1	**Strength of intact rock material**	**Point load strength index**	**>100 Kp/cm^2**	**40-80 Kp/cm^2**	**20-40 Kp/cm^2**	**10-20 Kp/cm^2**	**For this low range uniaxial compressive test is preferred**		
		Uniaxial comp. strength	**>2500 Kp/cm^2**	**1000-2500 Kp/cm^2**	**500-1000 Kp/cm^2**	**250-500 Kp/cm^2**	**50-250**	**10-50**	**<10**

	Rating	**15**	**12**	**7**	**4**	**2**	**1**	**0**

3. INDEX RQD (ROCK QUALITY DESIGNATION)

Defines the degree of fracturing of the Rocky massif from the analysis of **cores** recovered In **Probes** ($\Phi \geq 54.7$ mm).

The Estimation Of Index RQD can Also be performed from the frequency of discontinuities, λ, by the following Expression that provides the value Theoretical Minimum RQD:

$$RQD \approx 100e^{-0.1\lambda}(0.1\,\lambda + 1)$$

2	**RQD**	**90%-100%**	**75%-90%**	**50%-75%**	**25%-50%**	**<25%**
	Rating	**20**	**15**	**10**	**8**	**5**

4. SEPARATION OR SPACING BETWEEN JOINTS.

Distance between two consecutive discontinuity planes and of the same family, measured in a perpendicular direction to those planes. Spacing defines the size of he Matrix block Rock as well as a reference to the degree of fracturing.

Figure 4. Separation or spacing between joints

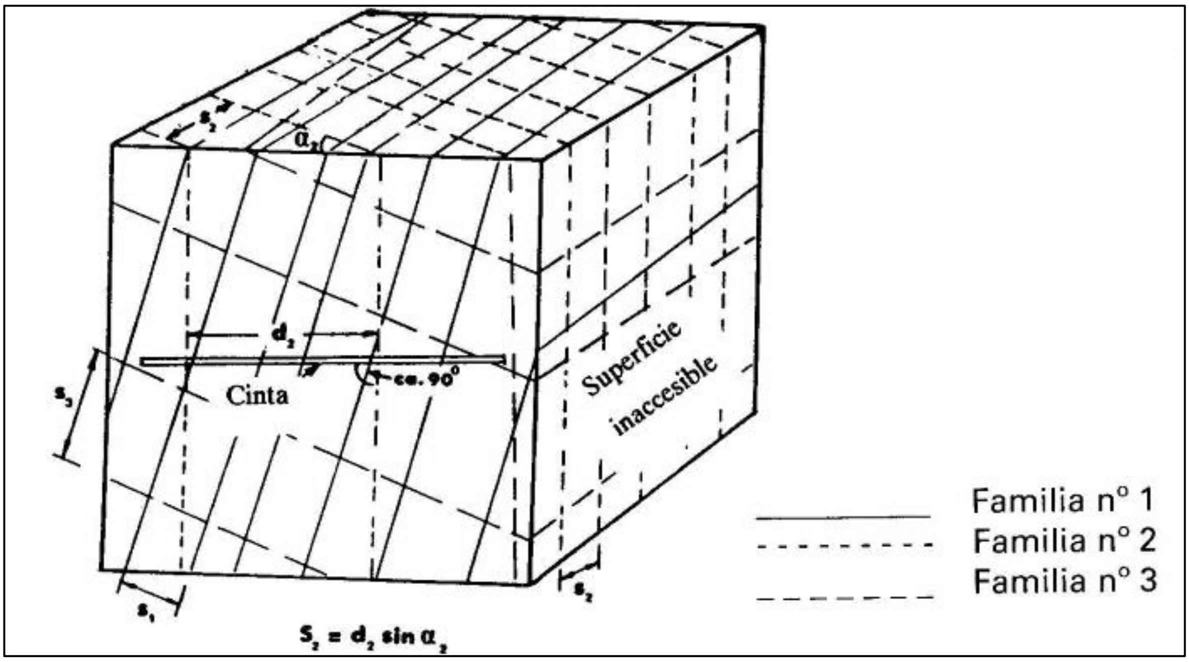


Table 3. Table of Separation or spacing between joints

3	**Spacing of discontinuities**	**>2m**	**0.6-2m**	**0.2-0.6m**	**0.06-0.2m**	**<0.06m**
	Rating	**20**	**15**	**10**	**8**	**5**

5. STATE OF THE JOINTS.

Roughness + opening + absence/presence of filler

Most unfavourable situation ⇔ Smooth discontinuity, with great openness and with presence of expansive clayey filling.

Figure 5. Typical-roughness-profiles-and-corresponding-range-of-JRC-

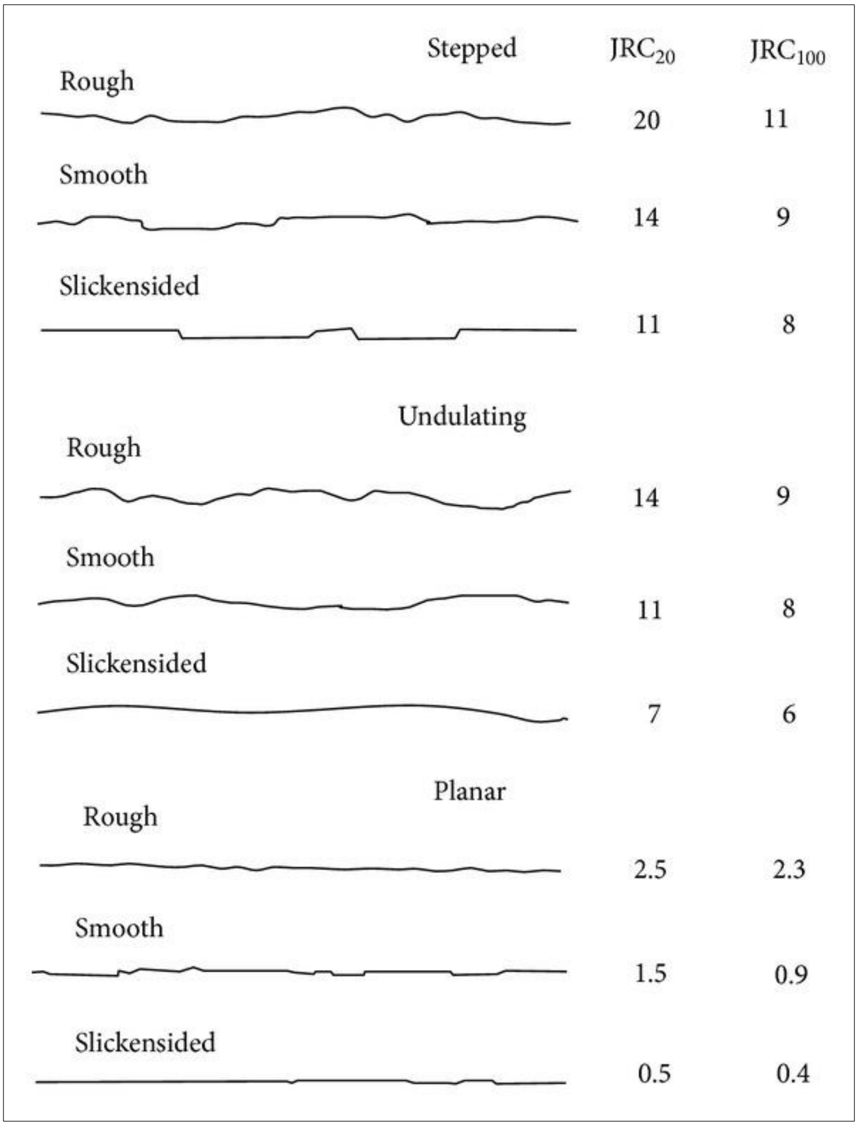


Table 4. Description of the openings

CLASS	DESCRIPTION	ABERTURA
I	Very closed	<0.1mm

II	Closed	0.1-0.25mm
III	Partially closed	0.25-0.5mm
IV	Open	0.5-2.5mm
V	Moderately broad	2.5-10mm
VI	Wide	>1cm
VII	Very wide	1-10cm
VIII	Extremely wide	10-100cm
IX	Cavernous	>1m

whose valuation for the calculation of the RMR is as follows:

Table 5. State of the joints

4	**Conditions of discontinuities**	**Very rough surfaces Not continuous No separation Unweathered Wall rock**	**Slightly rough surfaces Separation <1mm Slightly weathered walls**	**Slightly rough surfaces Separation <1mm Highly weathered walls**	**Slickensided surfaces or Gouge <5mm thick or separation 1-5mm continuous**	**Soft gouge >5mm thick or separation >5mm continous**
	Rating	**30**	**25**	**20**	**10**	**0**

6. CIRCULATION OF WATER THROUGH DISCONTINUITIES. GROUNDWATER.

Indicative of the weathering suffered by the Rock As well as the lubricating effect of the water. Depending on the type of work, this parameter can be interpreted and calibrated in a different way.

Table 6. Water circulation through discontinuities

5	**Ground water**	**Inflow per 10m tunnel length (l/m)**	**Nulo**	**<10**	**10-25**	**25-125**	**>125**
		Relacion Joint wáter press/major principalσ	**0**	**0.0-0.1**	**0.1-0.2**	**0.2-0.5**	**0.5**
		General conditions	**Completely dry**	**Damp**	**Wet**	**Dripping**	**Flowing**
	Rating	**20**	**15**	**10**	**8**	**5**	

7. CORRECTION BY ORIENTATION OF THE DISCONTINUITIES.

Sliding in favor of the Stratification.

a. In the event of an Angle of friction between layers of 30 °, the shaded part is Move,

b. In the same case as the dip less than 30 ° no Would Sliding.

Figure 6. Correction by orientation of the discontinuities

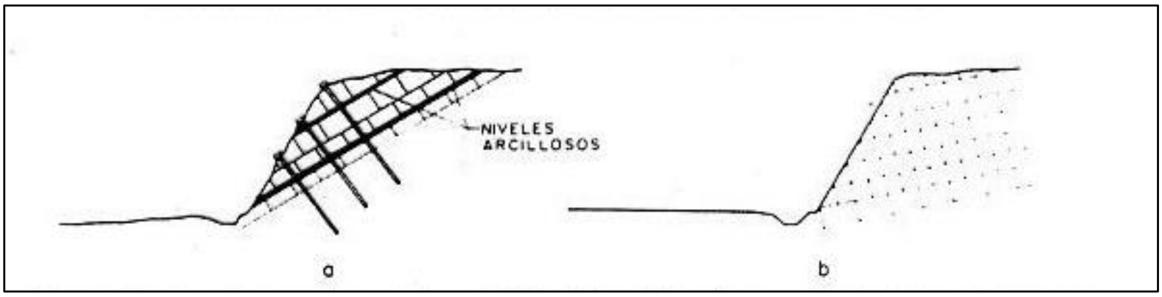


Table 7. Rating adjustment for discontinuity orientations

Strike and dip orientations		Very Favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-2.5	-50	-60

8. RESISTANCE TO CUTTING DISCONTINUITIES. CLASSIFICATION OF THE MASSIF ROCKY FROM THE RMR

Based on the values of RMR for a given structure, the Rocky massif is classified within one of the five categories named as very good, good, medium, poor and very poor as described in the table.

i. CLASSIFICATION OF THE ROCKY MASSIF FROM ROCK MASS RATING, RMR

Table 8. Classification Rock Mass Rating, RMR

Properties of the parameter	ROCK MASS RATING				
	100-81	80-61	60-41	40-21	<20
Description	Very good rock	good rock	Fair rock	Poor rock	Very poor rock
Average stand-up time	10 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass	>4 Kp/cm^2	3-4 Kp/cm^2	2-3 Kp/cm^2	1-2 Kp/cm^2	<1 Kp/cm^2
Friction angle of rock mass	>45°	35°-45°	25°-35°	15°-25°	<15°

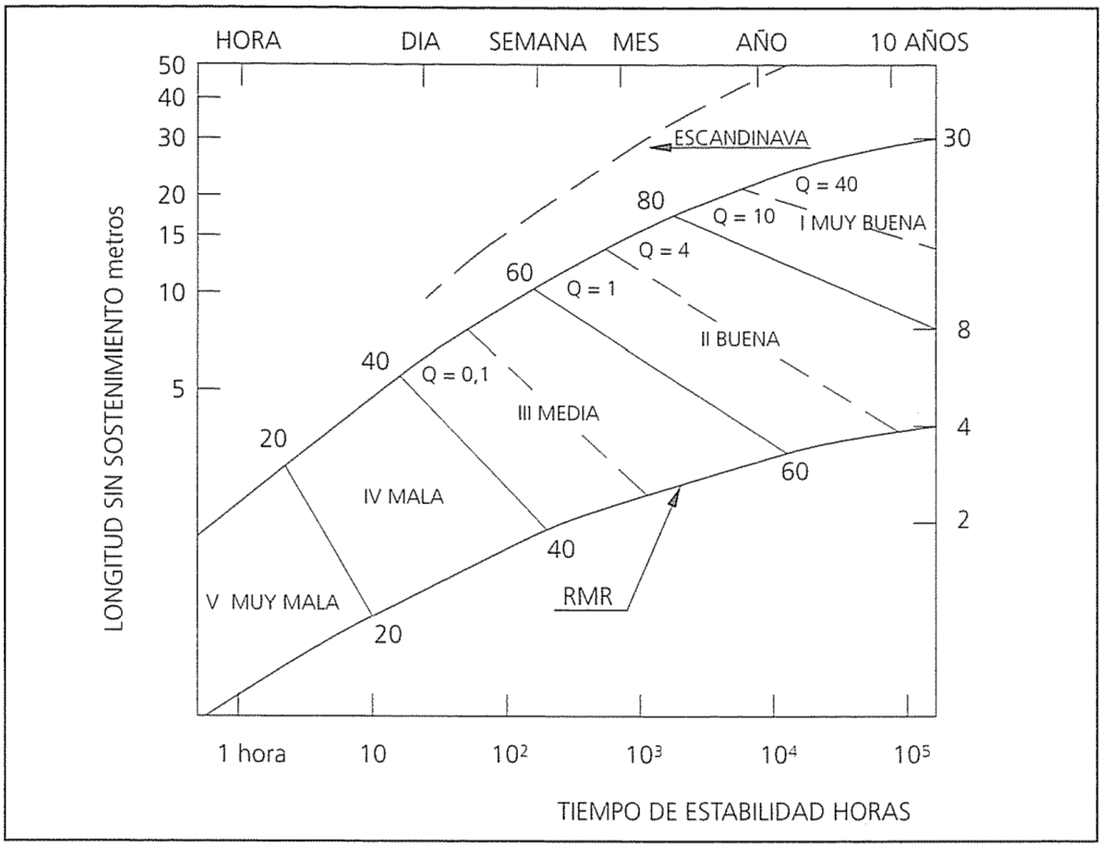
The cohesion value is an order of magnitude greater in the case of tunnels because the discontinuities are relatively narrower and less spaced between them.

9. TIME OF SELF-SUPPORT TUNNELS.

For arch-shaped tunnels the self-support time will be significantly larger than those with flat-roofed tunnels. For tunnels with arc-shaped ceilings, the stand-alone time is related to the RMR classification category, shown in the table Previous. is important to highlight that should not be delayed Unnecessarily the ceiling support, in the cases of rocky massifs with high self-support time, since this can lead to a deterioration of the rock massif.

Self-support time versus tunnel light for various kinds of rocky massifs and their classifications according to RMR. (Bieniawski, 1989).

Figure 7. Time of self-support tunnels



10. SUPPORT REQUIREMENT ACCORDING TO THE VALUE OF RMR.

Bieniawski (1989) provided a guide for determination of the Support requirements for a tunnel, with 10m free light, excavated according to the conventional method or Drill and Blast. The Guide is applicable Depending on factors such as depth From the surface (to have Present overload or stresses problems in situ), tunnel size and Digging method. The amounts of support present in The Table correspond to the permanent support and not to the temporary or primary support.

Table 9. Guide to digging and supporting tunnels according to Rock Mass Rating, RMR (Bieniawski, 1989)

Rock mass class	Excavation	Support		
		Rock bolts (20 mm diam., fully bonded)	Shotcrete	Steel sets
I-Very good rock RMR: 81-100	Full face: 3 m advance	Generally no support required except for occasional spot bolting		
II-Good rock RMR: 61-80	Full face: 1.0-1.5 m advance; Complete support 20 m from face	Locally bolts in crown, 3 m long, spaced 2.5 m with occasional wire mesh	50 mm in crown where required	None
III-Fair rock RMR: 41-60	Top heading and bench: 1.5-3 m advance in top heading; Commence support after each blast; Commence support 10 m from face	Systematic bolts 4 m long, spaced 1.5-2 m in crown and walls with wire mesh in crown	50-100 mm in crown, and 30 mm in sides	None
IV-Poor rock RMR: 21-40	Top heading and bench: 1.0-1.5 m advance in top heading; Install support concurrently with excavation - 10 m from face	Systematic bolts 4-5 m long, spaced 1-1.5 m in crown and walls with wire mesh	100-150 mm in crown and 100 mm in sides	Light ribs spaced 1.5 m where required
V-Very poor rock RMR < 21	Multiple drifts: 0.5-1.5 m advance in top heading; Install support concurrently with excavation; shotcrete as soon as possible after blasting	Systematic bolts 5-6 m long, spaced 1-1.5 m in crown and walls with wire mesh. Bolt invert	150-200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert

11. Q CLASSIFICATION SYSTEM OF BARTON (1974)

Determines the quality of the massif and is applied in definition of maintenance requirements in underground excavations.

Based on a large amount of data associated with tunneling projects, (Barton et al, 1974) of the Norwegian Geotechnical Institute (NGI) developed the Geomechanical classification system Q to estimate the need for support and fortification of tunnels. The value of Q is defined by six parameters combined in the equation 1.

In logarithmic scale, from 0.001 to 1000.

Defined from 6 parameters:

$$\boxed{Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}} \quad \text{(Equation 1)}$$

Where:

RQD = Rock quality designation.

Jn = Index According to the number of fracture systems

Jr = Index According to the roughness of the surface of the fractures.

Ja = Index according to the alteration in the surface of the fractures or their filling.

Jw = Water inflow.

Srf = (Stress Reduction Factor) coefficient dependent on the tensional state of the Rocky massif.

- The first quotient accounts for the size of the blocks that are formed;
- The second quotient is an indicator of the resistance between the blocks, controlled by the resistance in the discontinuities;
- The third quotient, called "active effort", considers the effect of water pressures, degrees of confinement or relaxation.

The traditional application of the value of Q depending on its 6 parameters is for the selection of a competent combination of Shotcrete and bolts for the support and fortification of Rocky massifs, i.e. for the estimation of the permanent cladding for tunnels or caverns in rock.

- Software application Dips, to determine the families of discontinuities.
- Analysis of the stability of the slope in the portals.

12. THE RQD PARAMETER

It was developed by Deere (Deere et al, 1963) to provide a quantitative estimate of the quality of the rocks as a function of witnesses. It is defined as "the percentage of pieces intact longer than 100mm over the total length of the control." The RQD is a relatively simple and fast measuring tool, occupied in the records by witnesses and is often the only method used to measure the degree of fracturing in the area of interest.

With respect to the other parameters, it is established that, Jn is the index for the number of joints families in the area of interest, Jr It is the index for roughness of the joints family more unfavorable or discontinuity stuffed, Ha It is the index for the degree of alteration or filling of clay for the family of joints more unfavorably, Jw It is the index for the influx of water and pressure effects, which can cause emptying of the filling of the joints, and SRF is the index for the fault, for resistance/solicitation reasons in massive hard rocks, for Piston structure or for Swelling.

To Then A sequence of tables is presented to estimate the values propitious for our purposes, calculation of the Factor Q.

Table 10. Table to define the 6 parameters of the Q

DESCRIPTION	VALUE	NOTES
1. ROCK QUALITY DESIGNATION	***RQD***	
A. Very poor	0 - 25	1. Where *RQD* is reported or measured as $\leq$ 10 (including 0),
B. Poor	25 - 50	a nominal value of 10 is used to evaluate *Q*.
C. Fair	50 - 75	
D. Good	75 - 90	2. *RQD* intervals of 5, i.e. 100, 95, 90 etc. are sufficiently
E. Excellent	90 - 100	accurate.
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use ($3.0 \times J_n$)
G. Three joint sets plus random	12	
H. Four or more joint sets, random,	15	2. For portals use ($2.0 \times J_n$)
heavily jointed, 'sugar cube', etc.		
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact		
b. Rock wall contact before 10 cm shear		
A. Discontinuous joints	4	
B. Rough and irregular, undulating	3	
C. Smooth undulating	2	
D. Slickensided undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is
E. Rough or irregular, planar	1.5	greater than 3 m.
F. Smooth, planar	1.0	
G. Slickensided, planar	0.5	2. J_r = 0.5 can be used for planar, slickensided joints having
c. No rock wall contact when sheared		lineations, provided that the lineations are oriented for
H. Zones containing clay minerals thick	1.0	minimum strength.
enough to prevent rock wall contact	(nominal)	
J. Sandy, gravely or crushed zone thick	1.0	
enough to prevent rock wall contact	(nominal)	

DESCRIPTION	VALUE		NOTES
4. JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
a. Rock wall contact			
A. Tightly healed, hard, non-softening, impermeable filling	0.75		1. Values of ϕr, the residual friction angle,
B. Unaltered joint walls, surface staining only	1.0	25 - 35	are intended as an approximate guide to the mineralogical properties of the
C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2.0	25 - 30	alteration products, if present.
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	20 - 25	
E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)	4.0	8 - 16	

4, JOINT ALTERATION NUMBER	J_a	ϕr degrees (approx.)	
b. Rock wall contact before 10 cm shear			
F. Sandy particles, clay-free, disintegrating rock etc.	4.0	25 - 30	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick)	6.0	16 - 24	
H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick)	8.0	12 - 16	
J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water.	8.0 - 12.0	6 - 12	
c. No rock wall contact when sheared			
K. Zones or bands of disintegrated or crushed	6.0		
L. rock and clay (see G, H and J for clay	8.0		
M. conditions)	8.0 - 12.0	6 - 24	
N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening	5.0		
O. Thick continuous zones or bands of clay	10.0 - 13.0		
P. & R. (see G.H and J for clay conditions)	6.0 - 24.0		
5. JOINT WATER REDUCTION	J_w	approx. water pressure (kgf/cm^2)	
A. Dry excavation or minor inflow i.e. < 5 l/m locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates; increase J_w if drainage installed.
D. Large inflow or high pressure	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure	0.1 - 0.05	> 10	
6. STRESS REDUCTION FACTOR		*SRF*	
a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated			
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock any depth)		10.0	1. Reduce these values of *SRF* by 25 - 50% but only if the relevant shear zones influence do not intersect the excavation
B. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth < 50 m)		5.0	
C. Single weakness zones containing clay, or chemically dis-tegrated rock (excavation depth > 50 m)		2.5	
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)		7.5	
E. Single shear zone in competent rock (clay free). (depth of excavation < 50 m)		5.0	
F. Single shear zone in competent rock (clay free). (depth of excavation > 50 m)		2.5	
G. Loose open joints, heavily jointed or 'sugar cube', (any depth)		5.0	

DESCRIPTION	VALUE			NOTES
6. STRESS REDUCTION FACTOR			***SRF***	
b. Competent rock, rock stress problems				
	σ_c/σ_1	$\sigma_t\sigma_1$		2. For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
H. Low stress, near surface	> 200	> 13	2.5	
J. Medium stress	200 - 10	13 - 0.66	1.0	
K. High stress, very tight structure (usually favourable to stability, may be unfavourable to wall stability)	10 - 5	0.66 - 0.33	0.5 - 2	
L. Mild rockburst (massive rock)	5 - 2.5	0.33 - 0.16	5 - 10	
M. Heavy rockburst (massive rock)	< 2.5	< 0.16	10 - 20	3. Few case records available where depth of crown below surface is less than span width. Suggest *SRF* increase from 2.5 to 5 for such cases (see H).
c. Squeezing rock, plastic flow of incompetent rock under influence of high rock pressure				
N. Mild squeezing rock pressure			5 - 10	
O. Heavy squeezing rock pressure			10 - 20	
d. Swelling rock, chemical swelling activity depending on presence of water				
P. Mild swelling rock pressure			5 - 10	
R. Heavy swelling rock pressure			10 - 15	

ADDITIONAL NOTES ON THE USE OF THESE TABLES

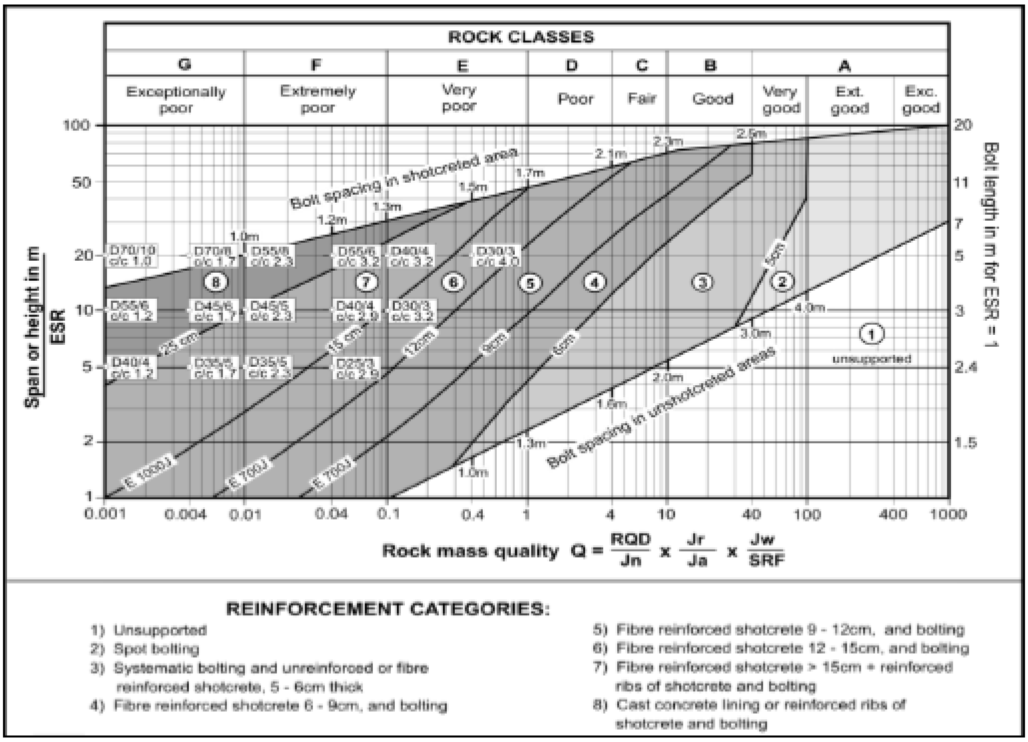
When making estimates of the rock mass Quality (*Q*), the following guidelines should be followed in addition to the notes listed in the tables:

1. When borehole core is unavailable, *RQD* can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relationship can be used to convert this number to *RQD* for the case of clay free rock masses: $RQD = 115 - 3.3\ J_v$ (approx.), where J_v = total number of joints per m^3 ($0 < RQD < 100$ for $35 > J_v > 4.5$).
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed, these parallel 'joints' should obviously be counted as a complete joint set. However, if there are few 'joints' visible, or if only occasional breaks in the core are due to these features, then it will be more appropriate to count them as 'random' joints when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the weakest significant joint set or clay filled discontinuity in the given zone. However, if the joint set or discontinuity with the minimum value of J_r/J_a is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating *Q*. The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.
4. When a rock mass contains clay, the factor *SRF* appropriate to loosening loads should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent, the strength of the intact rock may become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.
5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to the present and future in situ conditions. A very conservative estimate of the strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

13. Q SUPPORT GRAPH

In this graph, the equivalent dimension, De, versus the value of Q, is determined, with the support categories defined in the graph originally published by Barton. This graphic was later updated to directly provide support for the conditions defined by the classification. Finally, Grimstad and Barton (1993) made one of the latest updates reflecting the increase in the use of steel fibers for shotcrete reinforcement in underground excavations.

Figure 8. Support graph for Q. (Palmstrom A., 2009)



CHAPTER IV

RESULTS: CHARACTERIZATION OF MATERIALS/ PROCESS SIMULATION/ ECONOMIC STUDY OR PRE-FEASIBILITY OF THE PROJECT

The tunnel will cross intrusive and extrusive igneous rocks (Figures 3, 4).

Figure 9. Hornblendic granite of pinkish hue, crystals of fine grain of 1mm, slightly altered

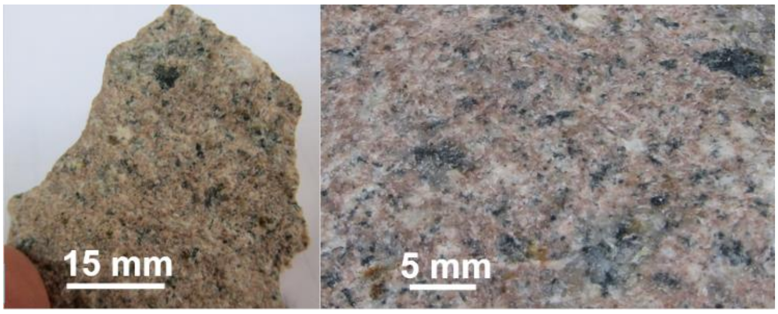


Figure 10. Hornblendic porphyritic. Andesite with a blackish gray color.

The rock samples were tested for resistance to compression in the FIGMM-UNI rock mechanics laboratory, obtaining the following results:

Table 11. Results of the compression resistance test of eight samples

Sample	Lithology	σ_c (Mpa)
M-1	Andesite	203.37
M-2	Andesite	183.23
M-3	Andesite	190.15
M-4	Granodiorite	121.04
M-5	Granodiorite	90.45
M-6	Granodiorite	214.92
M-7	Granite	218.29
M-8	Granite	234.96

For the Following procedure, a number of stations have been determined to find the measures of the discontinuities and start doing the calculations.

For this project Analyzed a Possible stroke of the tunnel (A1, A2 and A3) comprising 2 stages.

Figure 11. Typical section of the San Francisco tunnel project

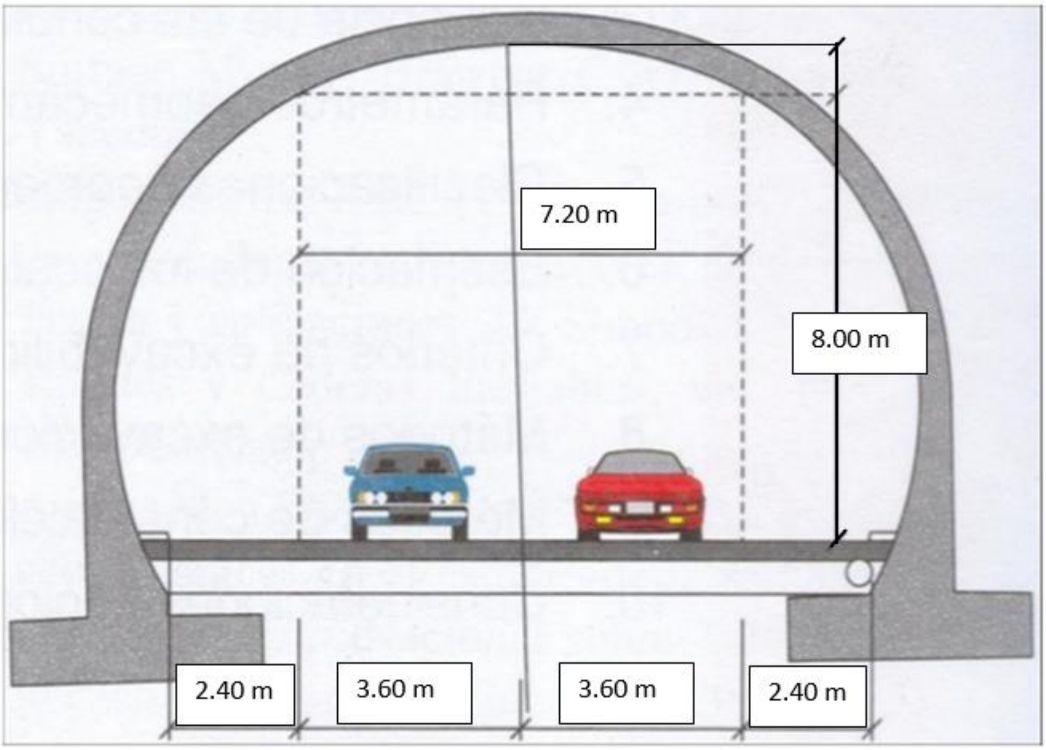
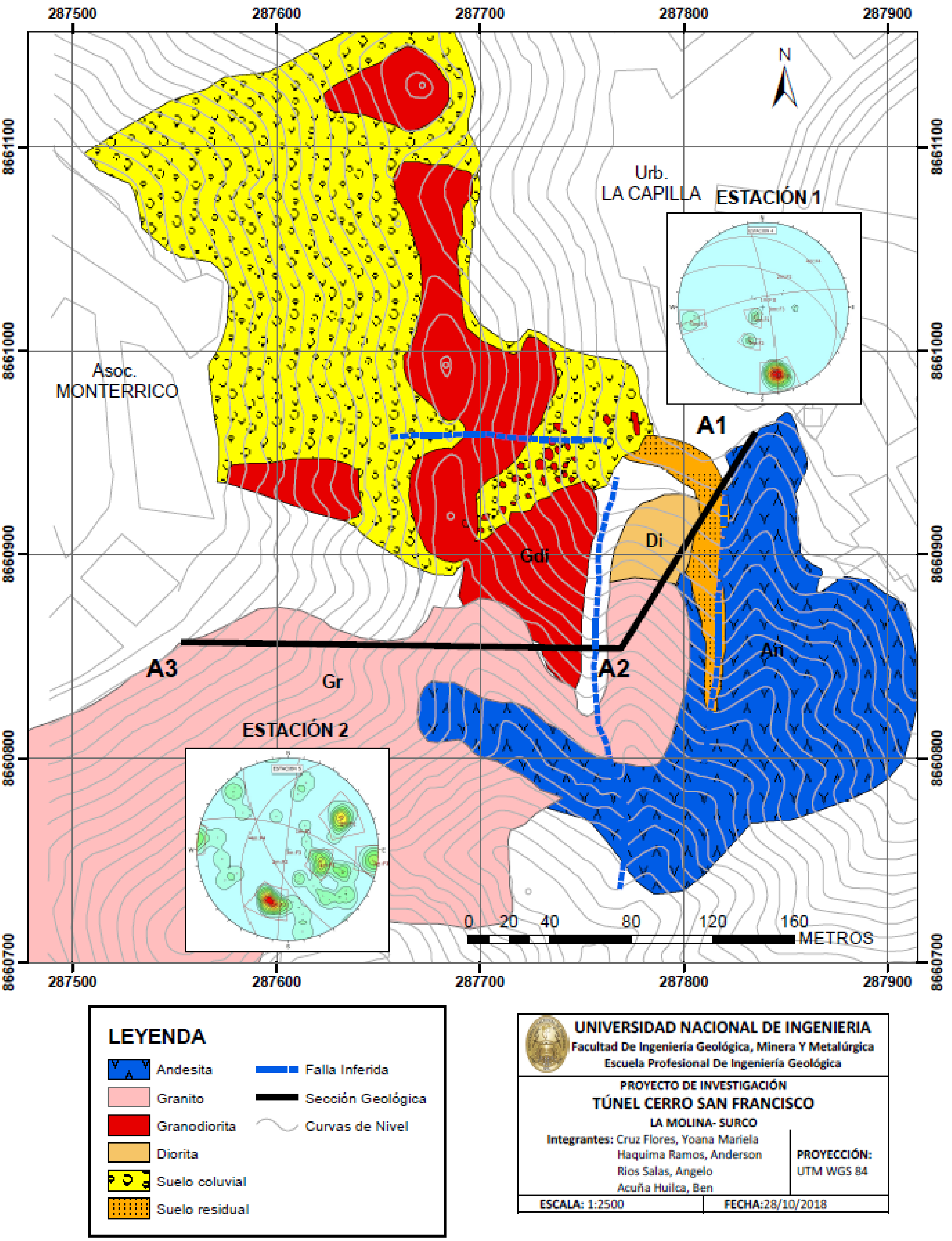


Figure 12. Structural geological map of the work area. (Geological plane with the alternative stroke for the train)



a. FIRST STAGE:

- Recognition of the work area
- Geological and geotechnical mapping.
- Selection of the structural domains and location of the stations of measurement of the discontinuities.
- Field data collection: Rocky matrix for petrographic and petrological studies, Rock Mechanics tests (resistance) and annotation of discontinuities characteristics (spacing, persistence, continuity, roughness, resistance of Walls, opening, filling and water filtrations).

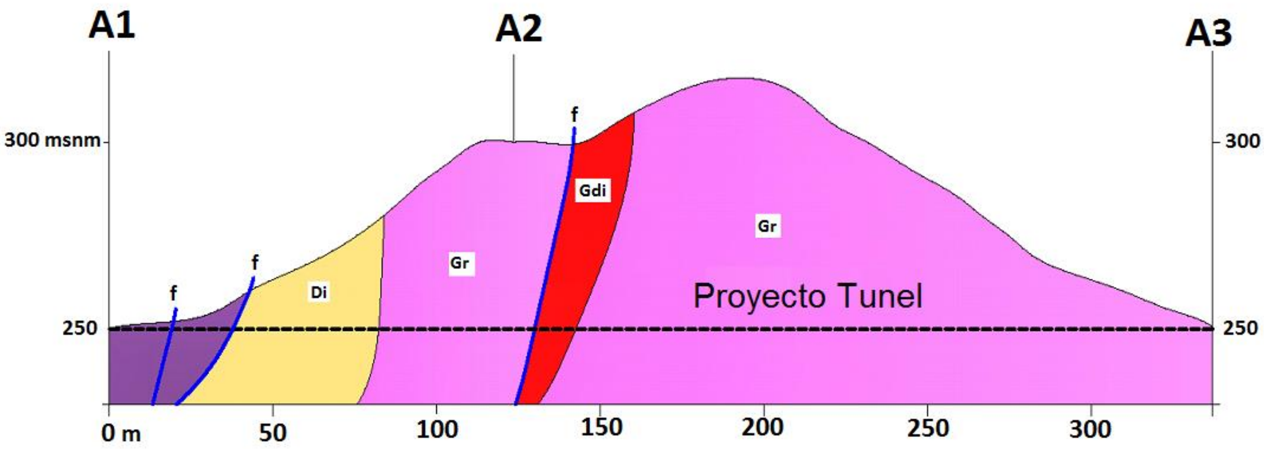
b. SECOND STAGE:

Classification of the Rocky Massif RMR, Q, stability analysis of the portals, analysis of stability of wedges and calculation of support.

It should be noted that the area studied presented poorly accessible places that limited field work, and special for data taking.

As worked in the field and in the cabinet are presented the following values used for the RMR and Q Barton.

Figure 13. Section of the possible stroke of the tunnel.



c. GEOMECHANICAL CHARACTERISTICS OF THE TUNNEL LINE

In this alternative a tunnel of 340.00 m in length has been proposed, which would be built in two sections: first section of 130 m, of S30 ° W direction; Second section of 210 m, from W.

The A1 portal would be located in the La Molina district at the end of Av. De los Cóndores and the A3 portal in the Surco district, which would be linked to Av. Los Constructores.

Figure 14. Entrance portal A1, La Molina, geomechanical station 1

- **Portal A1 and progressive tunnel 0 + 090 to 0 + 220 excavation direction N 80 ° E**

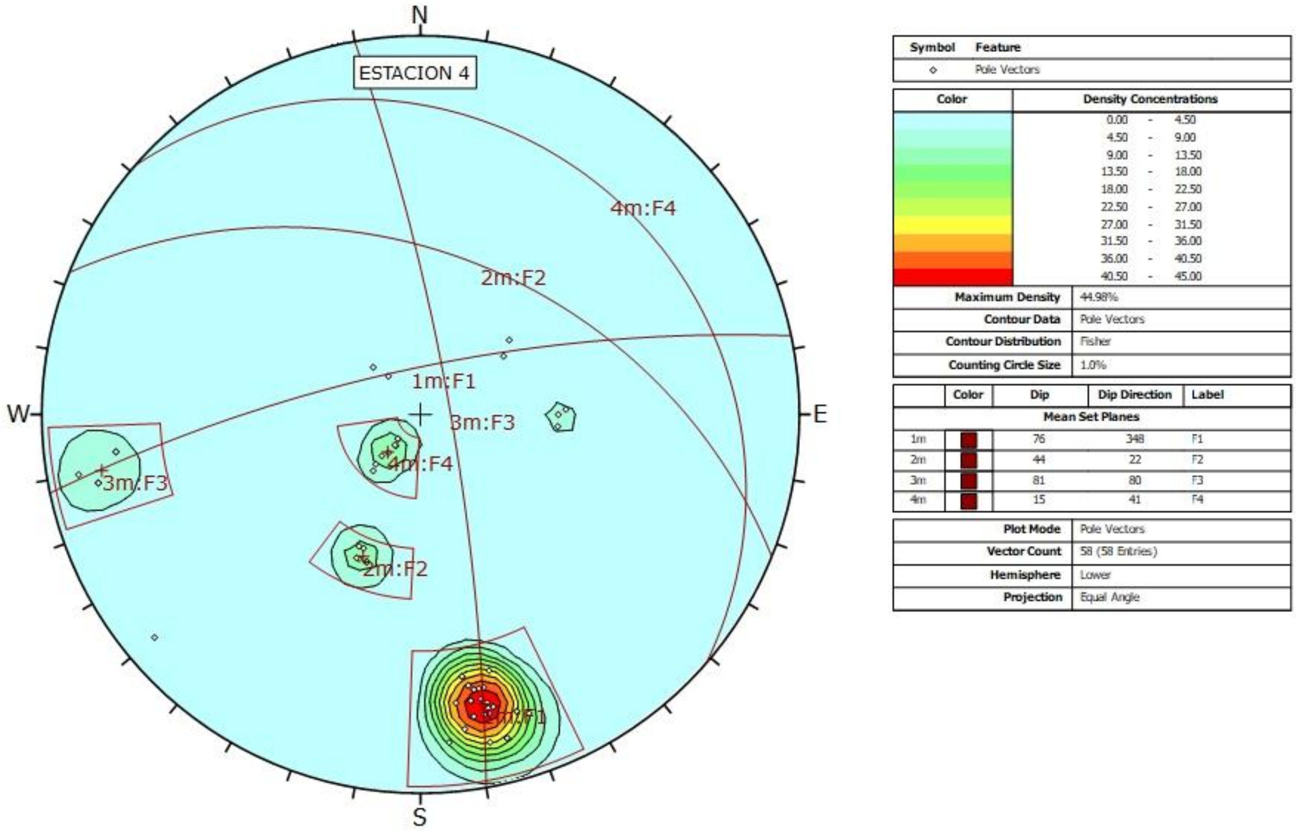
  Four main and one sporadic fracture systems were determined, the orientations are as follows:

  - F1: N 78 ° E / 76 ° NW
  - F2: N50 ° W / 15 ° NE
  - F3: N68 ° W / 44 ° NE
  - F4: N10 ° W / 81 ° NE

  According to the analysis of stability of this portal of orientation N20 ° W / 30 ° SW is stable.

  With these fracture systems and their respective parameters and with the use of the Unwedge program, 4 wedges with a high safety factor were determined, which is why it is stable.

Figure 15. Portal A1



- **Portal A3 and progressive tunnel 0 + 049 to 0 + 130**

  Three main and one sporadic fracture systems were determined, with the following orientations:

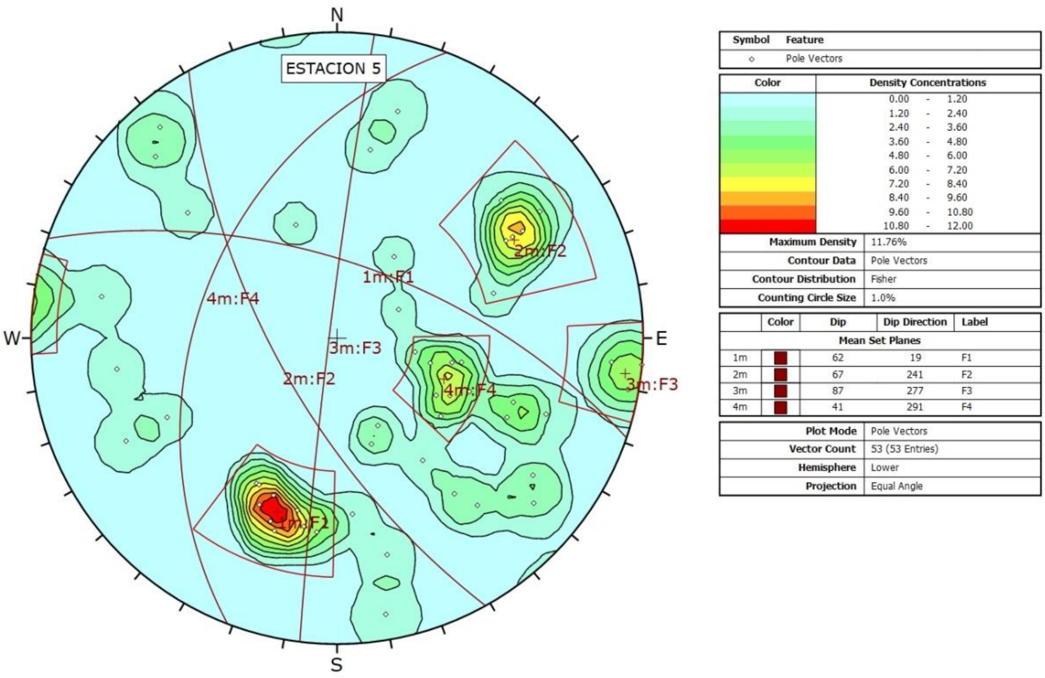
  - F1: N 76 ° W / 62 ° NE
  - F2: N30 ° W / 69 ° SW
  - F3: N21 ° E / 48 ° NW
  - F4: N7 ° E / 87 ° NW

  According to the slope stability analysis of this orientation portal N55 ° E / 45 ° NW, 2 wedges are presented.

  With the Unwedge program, wedges with a high safety factor were determined, which is why it is stable between 0 + 049 to 0 + 130 and in the excavation direction S30 ° W

  Between the progressive 0 + 130 there are 2 wedges with high safety factors, which is why it is considered stable.

Figure 16. Portal Slope A3 of address N55 ° E / 45 ° NW.



d. CALCULATIONS

For the following Results, the carefully analyzed field data has been used to determine the Value that best fits the characteristics of the rock we have studied.

Table 12. Calculations of the RMR

Progressions	0+000 - 0+049	0+049 – 0+078	0+078 – 0+130	0+130 – 0+201	0+201 – 0+297
Q	1.22	4.87	1.43	4.87	1.22
RQD	73	73	64.5	73	73
Jn	24	9	9	9	18
Jr	1	1.5	3	1.5	1.5
Ha	1	1	3	1	2
Jw	1	1	1	1	1
Srf	2.5	2.5	5	2.5	2.5

Table 13. Calculations of the Q system

Progressions	0+000-0 + 049	0+049 – 0+078	0+078 – 0+130	0+130 – 0+201	0+201 – 0+297
RMR	65-10= 55	65-10= 55	61-10 = 51	66-10 = 56	66-10 = 56
Long. DISC.	4	4	4	4	4
Opening	1	1	1	1	1
Roughness	3	3	3	3	3
Filling	2	2	0	2	2
Alteration	5	5	3	6	6
Spacing	10	10	10	10	10
Moisture	15	15	15	15	15
Resistance	12	12	12	12	12
RQD	13	13	13	13	13

CHAPTER V

DISCUSSION OF THE RESULTS

Making the respective comparatives for the analysis of the support for the method of calculating the RMR and Q BARTON, the FF values were obtained:

Table 14. Table of results.

PROGRESSIONS	Q BARTON	RMR
0 + 000-0 + 049	1.22	55
0 + 049 – 0 + 078	4.87	55
0 + 078 – 0 + 130	1.43	51
0 + 130 – 0 + 201	4.87	56
0 + 201 – 0 + 297	1.22	56

We can appreciate that the Q values represent with best detail the rock characteristics to determine the right characteristics to find the appropriate support in the sections presented in our tunnel profile.

In almost everything the work area has a rock of average quality, why the requirements for the support in both methods are similar in their characteristics.

Table 15. Comparison of the results for the support

PROGRESIONES	Q Barton	RMR
0+000 - 0+049	• Fibre reinforced shotcrete 9.00-12.00 cm and bolting. • Bolt length = 3.5 m spacing = 2.3 m	• **Bolts:** Systematic bolting 4.00 m length and spacing 1.50 a 2.00 m with occasional mesh. • **Shotcrete:** 5.00-10.00 cm on the roof y 3.00 cm on the walls
0+049 – 0+078	• Systematic bolting and unreinforced or fibre reinforced shotcrete, 5-6cm thick. • Bolt length = 3.5 m spacing = 2.5 m	• **Bolts:** Systematic bolting 4.00 m length and spacing 1.50 a 2.00 m with occasional mesh. • **Shotcrete:** 5.00-10.00 cm on the roof y 3.00 cm on the walls

0+078 – 0+130	• Fibre reinforced shotcrete 6.00-9.00 cm and bolting. • Bolt length = 3.5 m spacing = 2.3 m	• **Bolts:** Systematic bolting 4.00 m length and spacing 1.50 a 2.00 m with occasional mesh. • **Shotcrete:** 5.00-10.00 cm on the roof y 3.00 cm on the walls
0+130 – 0+201	• Systematic bolting and unreinforced or fibre reinforced shotcrete, 5-6cm thick. • Bolt length = 3.5 m spacing = 2.5 m	• **Bolts:** Systematic bolting 4.00 m length and spacing 1.50 a 2.00 m with occasional mesh. • **Shotcrete:** 5.00-10.00 cm on the roof y 3.00 cm on the walls
0+201 – 0+297	• Fibre reinforced shotcrete 9.00-12.00 cm and bolting. • Bolt length = 3.5 m spacing = 2.3 m	• **Bolts:** Systematic bolting 4.00 m length and spacing 1.50 a 2.00 m with occasional mesh. • **Shotcrete:** 5.00-10.00 cm on the roof y 3.00 cm on the walls

CONCLUSIONS

So, with these pictures Presented Detailing the Desired parameters, we demonstrate that both methods are Comparable and even similar.

Better detail is achieved with the Parameter Q. This Difference is because the method "Q" Consider efforts to be an underground excavation.

The RMR method considers corrections for discontinuities, the water FACTOR and the states of these, which propitiate a different point of analysis.

With these methods we have been able to estimate the quality of the Rocky massif, the stability of the analyzed zones, and form the basis for the establishment of costs.

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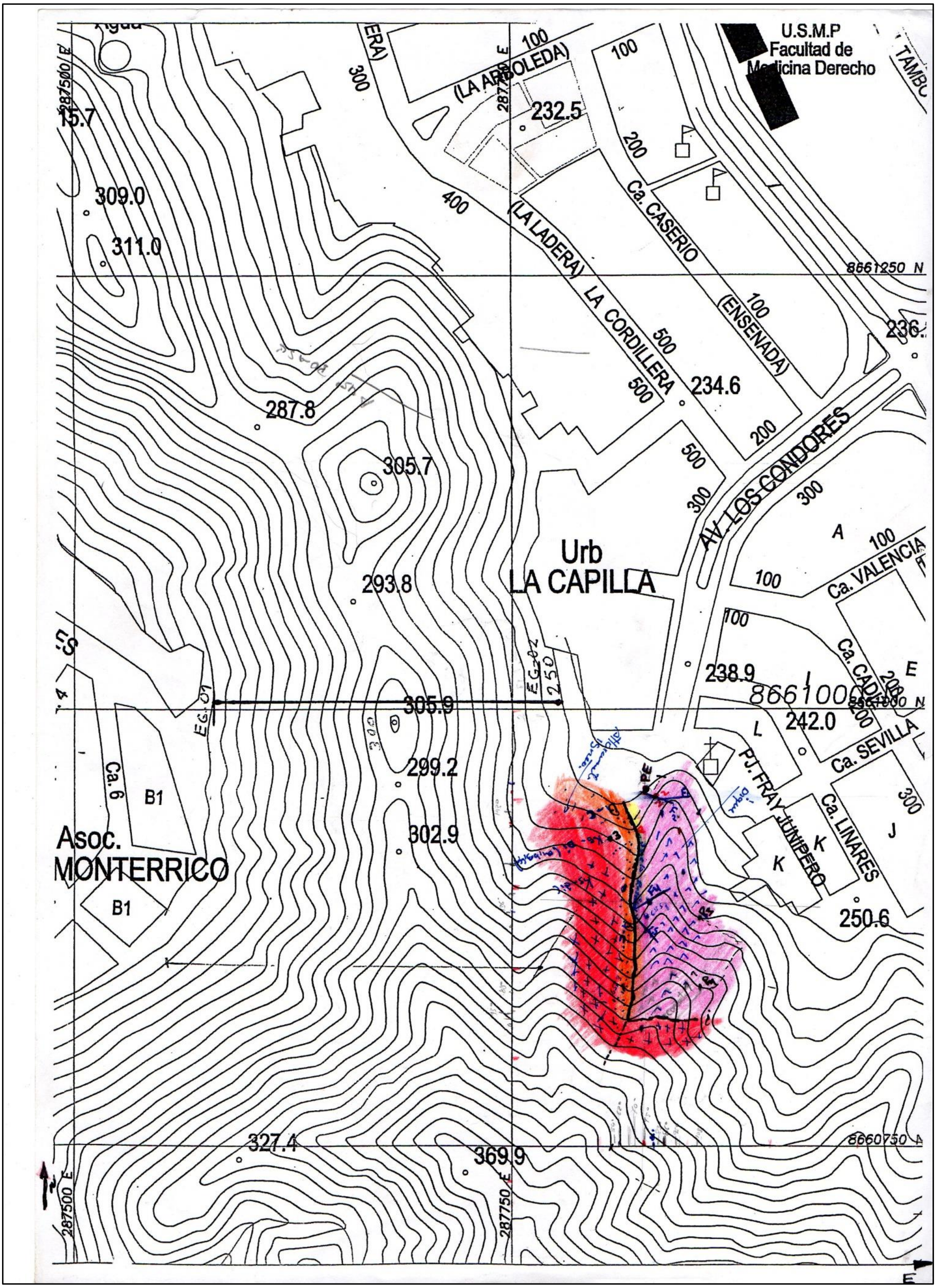
Map of the Field



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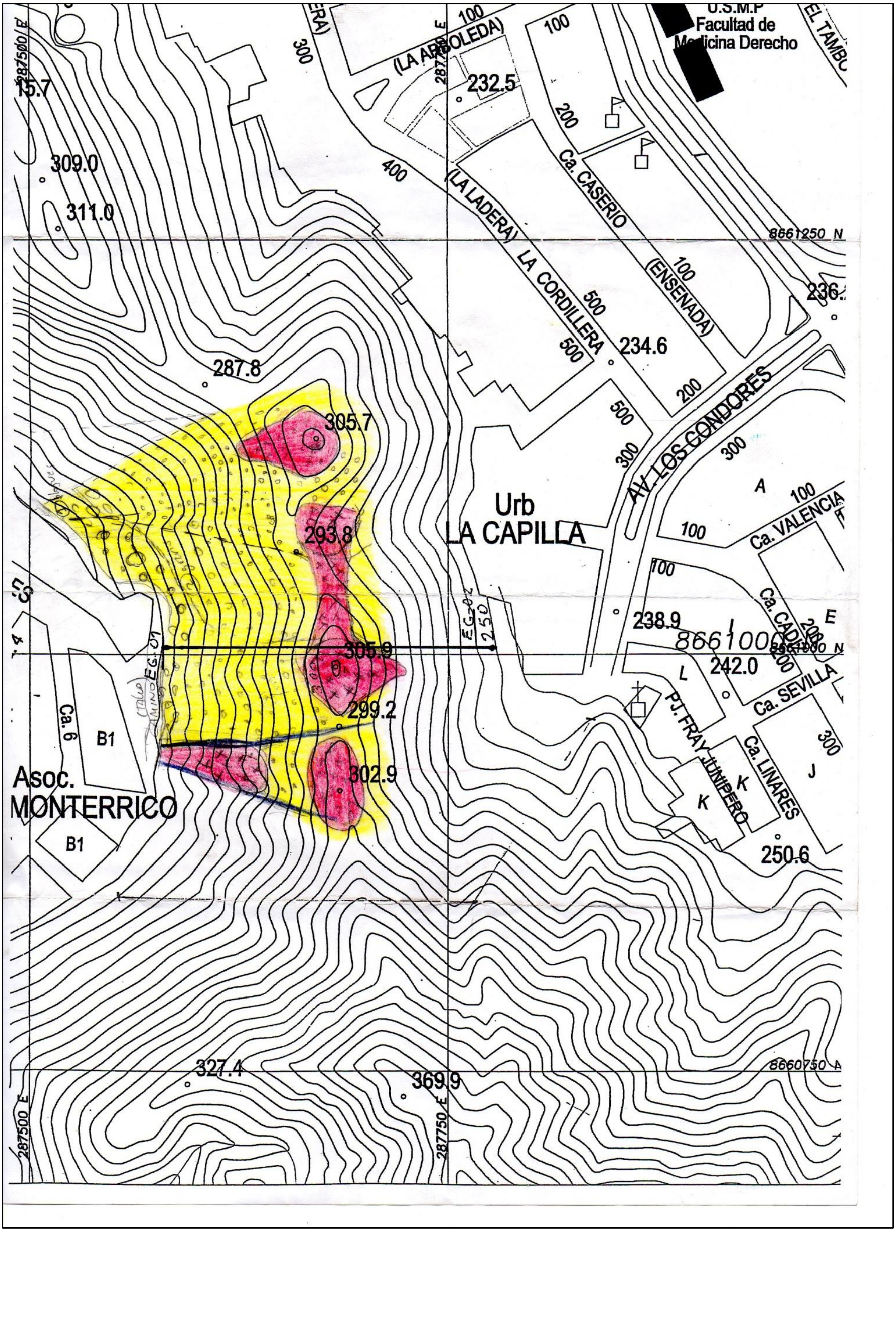

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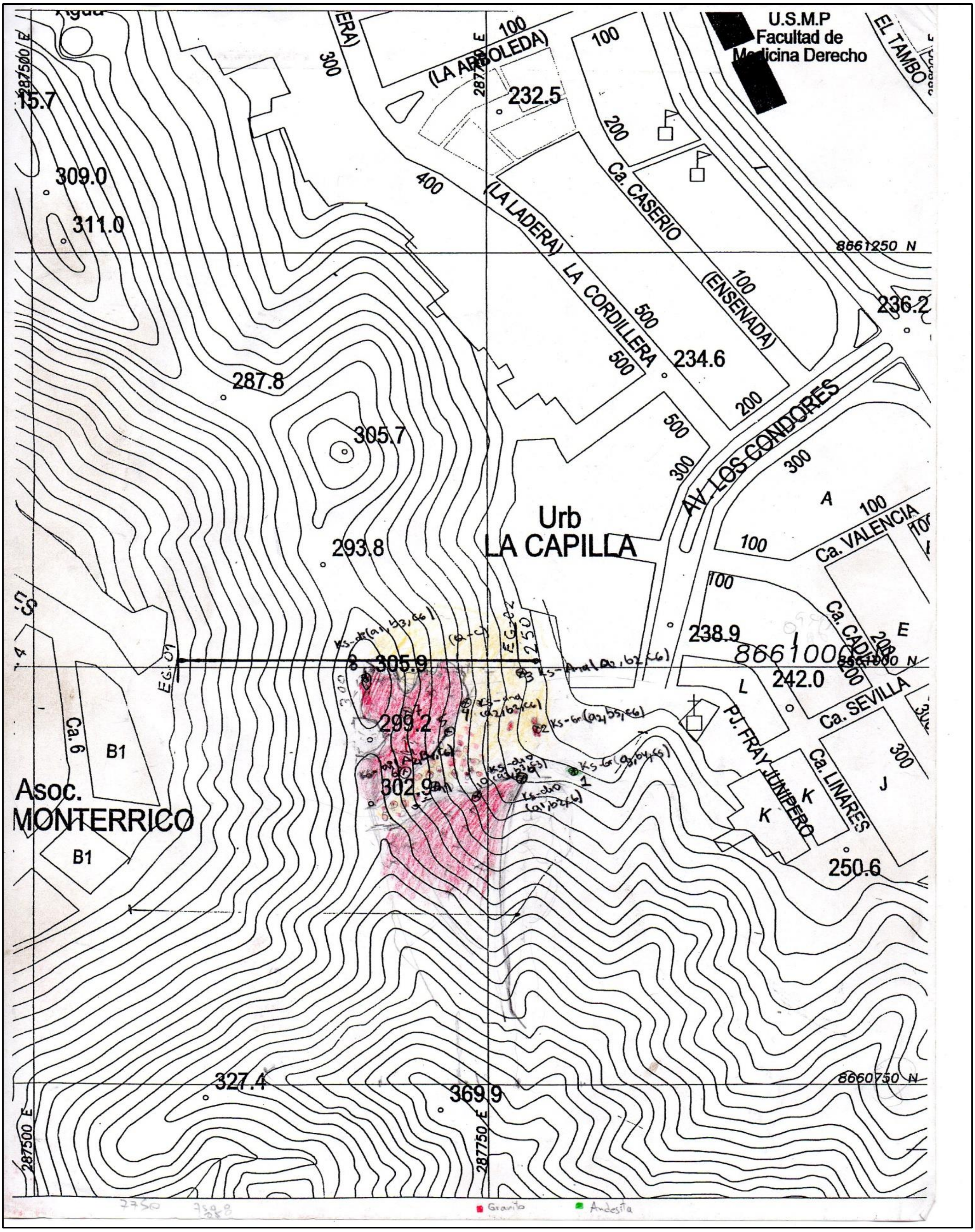
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APPENDIX A

Engineering Standards Applied in the Project

ISO 14689:2017

Identification, description and classification of rock

This standard specifies the rules for the identification and description of rock material and mass on the basis of mineralogical composition, genetic aspects, structure, grain size, discontinuities and other parameters. It also provides rules for the description of other characteristics as well as for their designation. The standard applies to the description of rock for geotechnics and engineering geology in civil engineering. The description is carried out on cores and other samples of rock and on exposures of rock masses.

ASTM D2487 - 17e1

Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)

This standard classifies soils from any geographic location into categories representing the results of prescribed laboratory tests to determine the particle-size characteristics, the liquid limit, and the plasticity index.

The various groupings of this classification system have been devised to correlate in a general way with the engineering behavior of soils. This standard provides a useful first step in any field or laboratory investigation for geotechnical engineering purposes.

Slope Stability Evaluation and Acceptance Standards

Document No.: P/BC 2017-049

Department of Building and Safety

This standard is to provide uniform requirements for evaluation of and standards for acceptance of stability of slopes. These requirements include consideration of pertinent engineering geologic and soils engineering factors of the critical field conditions that may reasonably be expected at the project location. These requirements include documentation and recommendations needed to determine if the site as proposed to be developed has an acceptable level of stability.

Geotechnical Design Standard – Minimum Requirements

Manual

Department of Transport and Main Roads

General requirements. Performance requirements. Unreinforced embankments. Reinforced embankments. Cut slopes. Unreinforced cuts. Reinforced cut slopes. Deep foundations. Retaining structures. Embedded retaining walls. Reinforced concrete cantilever retaining walls. Soil nailed walls. Reinforced soil structure RSS walls. Gabion retaining walls. Boulder retaining walls.

ISO 18674-2:2016

Geotechnical monitoring by field instrumentation - Measurement of displacements along a line: Extensometers

This standard specifies the measurement of displacements along a line by means of extensometers carried out for geotechnical monitoring. The standard presents general rules of performance monitoring of the ground, of structures interacting with the ground, of geotechnical fills and of geotechnical works.

The standard is applicable to: monitoring the behavior of soils, fills and rocks; - checking geotechnical designs; deriving geotechnical key parameters; evaluating stability ahead of, during or after construction (e.g. stability of natural slopes, slope cuts, embankments, excavation walls, foundations, dams, refuse dumps, tunnels).

ASTM STP984

Rock Mass Classification and Tunnel Reinforcement Selection Using the *Q*-System

This standard provides an overview of the *Q*-system and documents the scope of case records used in its development. A description of the rock mass classification method is given using the following six parameters: core recovery (RQD), number of joint sets, roughness and alteration of the least favorable discontinuities, water inflow, and stress-strength relationships. Examples of field mapping are given as an illustration of the practical application of the method in the tunneling environment, where the rock may already be partly covered by a temporary layer of shotcrete. The method is briefly compared with other classification methods, and the advantages of the method are emphasized.

ASTM D5924 - 96

Standard Guide for Selection of Simulation Approaches in Geostatistical Site Investigations

This standard covers the conditions that determine the selection of a suitable simulation approach for a site investigation problem. Alternative simulation approaches considered here are conditional and nonconditional, indicator and Gaussian, single and multiple realization, point, and block. The standard describes the conditions for which the use of simulation is an appropriate alternative to the use of estimation in geostatistical site investigations.

ISO 19434:2017

Mining — Classification of mine accidents

This standard establishes a classification of mine accidents by their origin or causes, by the type of accident, and by their results or consequences. The latter includes only the accidents resulting into consequences on people, not equipment or machinery.

Different categories of causes, types and consequences of mine accidents are briefly defined, and a 3-digit code is assigned to each category. These can be combined to ultimately allocate a unique 15-digit code to each type of mine accident. This code can then be used in statistical analysis. Similarly, an allocated code clearly shows to which categories of causes, type of accident and resulting consequences the mine accident belongs to. The standard is applicable to all surface and underground mines.

APPENDIX B

Multiple Constraints, Restrictions and Limitations Considered in the Project

Geological and Geotechnical Constraints

Several technical concerns and constraints are considered in the project: stability of the opening during excavation, tunnel-induced displacement field, constraints for alignment, thrust zone, shear zone, fault zone especially when water-bearing, shallow overburden, existence of nearby structures, foreign objects inside the ground, complex geometry, recent global formations, fills, weathering, groundwater. These concerns and constraints must be carefully taken into account for a successful project.

Availability of Geological and Geotechnical Data

Part of the required soil and rock information was obtained by laboratory testing of soil and rock samples, as well as from in-situ analysis. Remaining information was obtained from other sources such as reports from technical reports Peruvian Geological Institute, Geological Engineering Chapter of Peruvian Engineers Association, Geological, Mining and Metallurgical Peruvian Institute IGMMP. All required information and data was finally found and made available, and applied to the project.

Uncertainties and Risks

No geotechnical project is risk free. Risk are managed, minimized, shared, transferred or accepted, but they cannot be ignored. Geological and geotechnical concepts with respect to structure could be uncertain. On the other hand, economic evaluations have uncertainties related to cost estimation, changing conditions in economically viable sites, changes in geotechnical technology, fluctuations in costs and market conditions, political situation, community relations, etc. All these issues must be carefully analyzed in order to ensure the profitability of the project for the most conservative economic conditions and diversity of scenarios. In this project, all these issues have been considered from a conservative scenario and criteria.

Safety Considerations

Geological and geotechnical activities present diverse safety issues that must be taken into account in the development of the project. It is important to comply with safety standards pointing to satisfy proper safety levels considering their impact in the project budget. Care of human life, well-being and safety is an important issue to take into account throughout the different stages of the project and its life-cycle.

Environment and Sustainability

Geological and geothecnical activities face diverse and broad environmental issues at both local and global levels which could affect the project sustainability. The project considers environmental issues such as potential effluents spills, soil, air and water pollution, habitat protection and biodiversity. The project also considers community relations with local people as an important stakeholder of the project.

Schedule

The project must be completed in one academic semester. It is estimated the project requires an average of 150 hours of teamwork with 4-5 students per team. Considering that, besides the senior design project course, students are enrolled in 3-4 additional courses in the academic semester, students have to plan ahead in order for identify all required activities, distribute the tasks among all team members and, finally, integrate all partial tasks to configure the final project.