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Congresso Internazionale su
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International Congress on
Large Underground Openings

Congrès International sur
Grands Ouvrages en Souterrain

Firenze, Italy - 8-11 giugno 1986

Volume I

ATTI - PROCEEDINGS - COMPTES RENDUS

SITE INVESTIGATIONS FOR BIG TUNNELS. AN APPLICATION TO THE PAJARES RAILWAY TUNNEL

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1. INTRODUCTION

One of the mayor engineering structure considered for a rapid railway solution between the Centre and North od Spain (Asturias) is the Pajares Tunnel. The main topographic relief to be crossed is the Cantabrian Mountain Range. The present railway track needs more than 30 tunnels to cross that mountains. However, the new alternative consists in a unique tunnel of 21.6 Km long. Fig. 1 shows a location map, while if Fig. 2 is presented the main features of that tunnel. A service tunnel will be also constructed next to the main one. The Spanish National Railways Enterprise (RENFE) requested geological and geotechnical studies to select the best tunnel location and trace aligment. Due to the large length of the tunnel, and great thickness of the overburden (more than 1100 m in some places), as well as the geological complexity, the selection of appropiate site inestigation techniques was of first importance. Methods of site investigations, related with very long tunnels are discussed as well as the results of the geological and geotechnical works carried out.

2. METHODOLOGY

Major constraints for the general planning of site investigation programm were as follows:

- Very high relief mountain.
- Very difficult acesses for site investigation techniques.
- Average overburden thickness of more than 500 m and maximum of 1100 m.
- Climatological difficulties in accordance with high mountains.

Under this circunstances the site investigation were directed to resolve the most critical geological problems. Such as those that could be seriously affected the economy of the tunnel construction. Besides the most relevant design geotechnical parameters were also the goal of the site investigation programm. The objetives of that programm are following listed, and summarised in Fig. 3.

- To know the geological structure of the tunnel area.
- To identify the main tectonic accidents and areas of inestability.
- To estimate the state of natural stresses.
- To evaluate the hydrogeological conditions.
- To identify potential abrassive or agressive rocks, presence of gases, and high temperature gradients.
- To know the range of the main geomechanical properties.

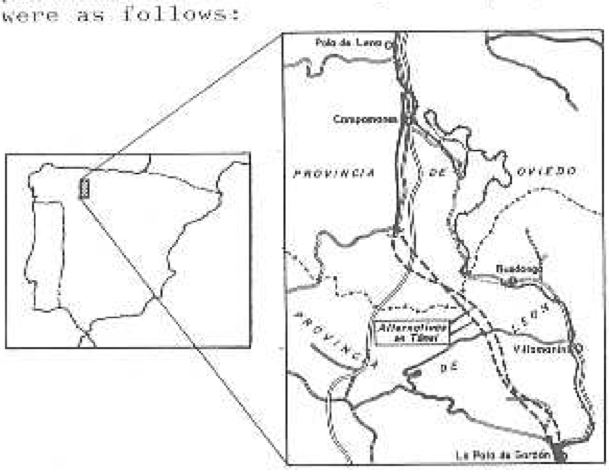


FIG. 1.-TUNEL LOCATION MAP

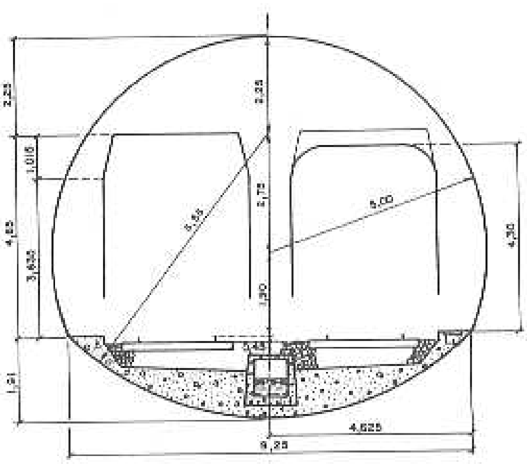


FIG. 2.- CROSS-SECTION OF THE TUNNEL

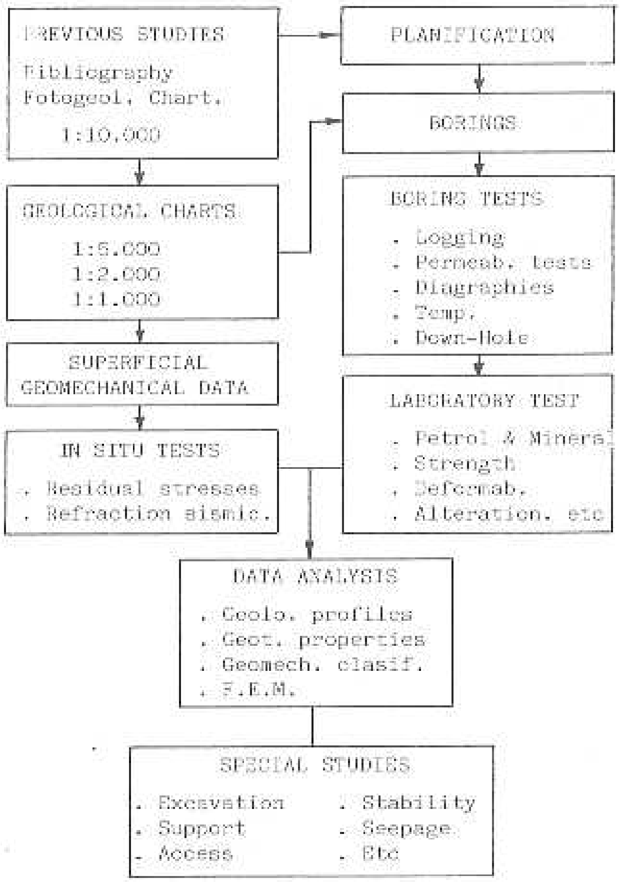


Fig. 3.- SUMMARY OF WORKS CARRIED OUT

- To classify the rock masses in accordance with geomechanical classifications.
- To obtain design parameters for stability and support design.
- To recommend alternative for the tunnel aligment, portal and access areas.

3. SITE INVESTIGATION PROGRAMM

The following site investigation techniques were carried out (fig. 3):

- Geological mapping (1/10.000 to 1/1.000 scales).
- Structural geology survey, hydrology and hydrogeological surveys.
- Checking of existing tunnels is the region.
- Borehole drilling (24 boreholes, vertical and inclinates, with a total drilling of 4300 m, length ranged from 20 m to 735 m), with RQD determination, rock samples, etc.
- Geophysical techniques: seismic refraction, down-hole, well logging with sonic logs, gamma-gamma, density, temperature and gases logs.
- Pumping test in bore-holes and packer test.
- Trenching (4 m depth and 20 m length).
- Stress release measurements by overcoring in rock out crops.

Laboratory tests included the main conventional test over 400 rocks samples (petrology, specific gravity, porosity, water content, uniaxial and triaxial strength, tensile strength, shear strength, durability, etc).

4. GEOLOGY OF THE TUNNEL AREA

The tunnel is located in the Hercynian region of the Cantabrian Mountain Range, crossing this range from south to north. There is a wide range of litological units, with absence of metamorphic and igneous rocks. The main rock formations are shales, limestones, sandstones and quartzites, from Paleozoic.

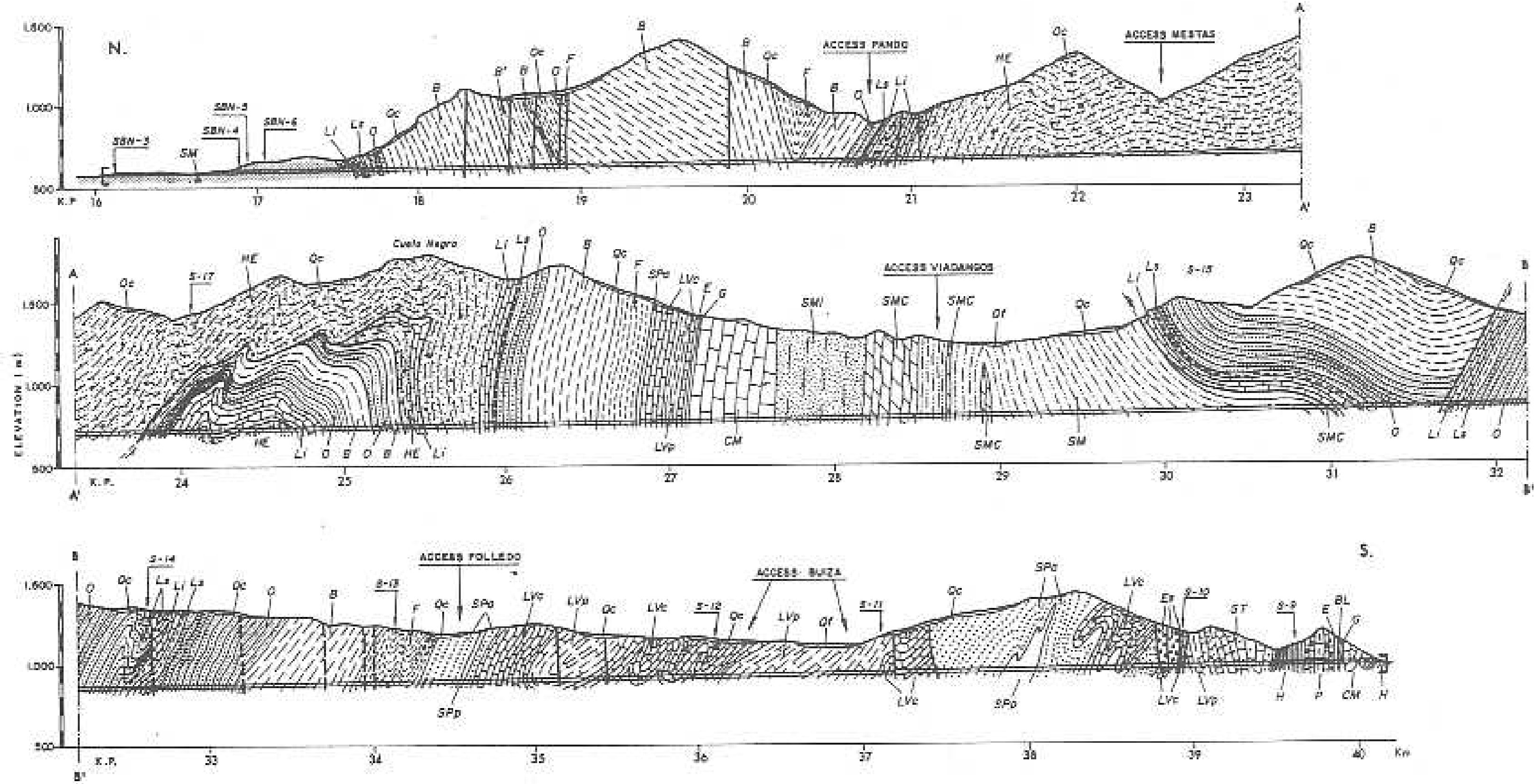
The main tectonic features of the area is a important regional thrust which separates two large geological units (the Sobia-Bodon Unit in the north, and the Somiedo-Correcilla in the south). There are also many important faults and thrust, such as the "Leon Line", and the Beberino faults.

A geological cross section is presented in Fig. 4, showing a general high dipping of the layers. The tunnel is crossing perpendicular to structures. More of 70% of the rocks affecting the tunnel are of very high durability, while the other 30% (mainly shales) are of low resistance to environmental changes. On the other hand more than 40% of the tunnel length could be on very abrassive rocks (quartz, sandstones and quartzites). The main types of rocks and their length are followed described:

- 7850 m of shales.
- 5050 m of quarcites.
- 4000 m of limestones.
- 3100 m of alternance of shales and sandstones.
- 2000 m of sandstones.

5. NATURAL STRESSES

State of natural stresses were estimated using geological criteria and empirical methods. Geological investigations shown that the main stress direction is North-South. Stress release measured by overcorings on rock outcrops confirm that residual stresses have the same main North-South direction. Observations carried out by Romana and Estefania (1983) in tunnels of the region suggest stress concentrations on the major regional thrusth. Other empirical results (Hoek and Brown, 1980) were also used. It was defined different zones



QUATERNARY

Oc — COLUVIONS
Of — ALUVIALS

CARBONIFEROUS

ES — CONGLOMENTES
SM — SHALES
SMc – LIMESTONES
SMI – SANDSTONES
CM — AND SHALES
G — LIMESTONES

DEVONIAN

BL — LIMESTONES
E — SANDSTONES
P — LIMESTONES
H — SHALES Y SANDSTONES
ST — LIMESTONES
LVp — SHALES
LVc — LIMESTONES
SPa — SADSTONES Y QUARZITAS

SILURIAN

SPp — SHALES Y SANDSTONES
F — SHALES Y SANDSTONES

ORDOVICIAN

B′ — SANDSTONES
B — QUARTZITES

CAMBRIAN

O — SHALES Y SANDSTONES
Ls – LIMESTONES
Li – DOLOMITES Y LIMESTONES
HE – CONGLOMERATES Y SANDSTONES
SBN-5 MECHANICAL LOGS
S-9
ES – LOCAL NAME OF THE FORMATION

FIG.-4 GEOLOGYCAL PROFILE ALONG PAJARES TUNNEL of initial stresses as follows:

1.- Overburden thickness (H) higher than 500 m:

. $k = \sigma_h / \sigma_v = 1.5$ in areas affected by important faults or $\sigma_c / \gamma H < 3$

. $k = 2$ if $\sigma_c / \gamma H > 3$ and faults.

2.- $H < 500$ m:

. $k = 1.5$ in compressive tectonic areas.

. $k = 2.0$ if $\sigma_c / \gamma H < 3$ or with faults.

. $k = 2.5$ for $\sigma_c / \gamma H > 3$ + faults.

where γ = is the specific unit weight

σ_c = uniaxial compresive strength

6. HIDROGEOLOGICAL CONDITIONS

The highly carstic limestones and dolomites present in some parts of the tunnel trace are the most permeable formations. Besides, the almost vertical position of the layers and the climatological conditions of a large mountain range, give a very high potential for important problems of seepage and drainage control. Other aspects to be considered in relation with the drainage is the high slope of the tunnel in only one direction, and the small service tunnel which can be used for drainage control.

7. GEOTECHNICAL CHARACTERISTICS

A large number of laboratory test were carried out as well as in situ parameters as described in fig. 3. Intact rock and rock mass properties were estimated for the main geotechnical properties. Fig. 5 shows the range of values for the uniaxial compressive strength, tensile strength and deformation modulus. Due to the vertical planes of foliation difficulties were encountered to test core samples for shear strength evaluation. Representative values of shear strength in the rock joints can see in the Table 1.

where ϕ_p is the peak friction angle; ϕ_r is the residual friction angle; c_p is the peak cohesion and the c_r is the residual cohesion.

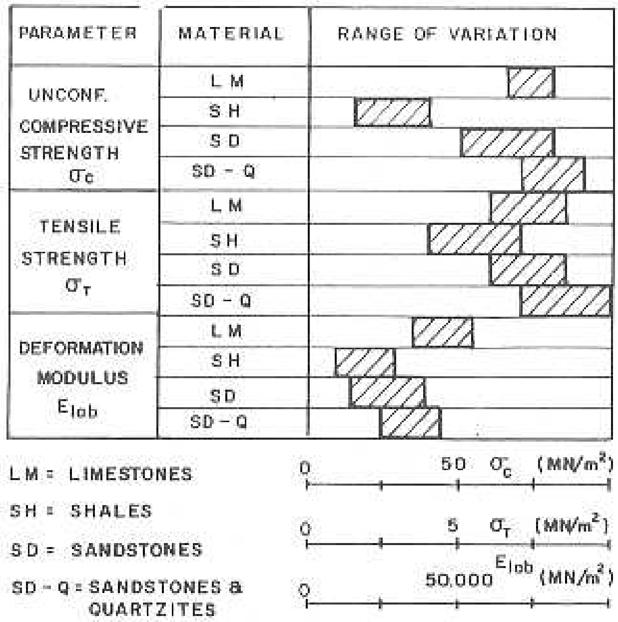


FIG.-5 VARIATION OF THE MAIN LABORATORY PARAMETERS OF THE TUNNEL MATERIALS

The geotechnical properties of rock masses has been assessed using the geomechanical classifications and analytical methods. One of the most influence parameters on the design analysis was the deformability properties of the rock mass. For a first approach the material was considered as elastic with a apparent deformation modulus E* and Poisson ratio ν*. The procedure to evaluate deformation moduli has been as follows:

a) Dynamic modulus assessed by down-hole seismic test.

b) Using RQD relations and the ? coefficient related with E_{lab} proposed by Deere et al (1966). The change of RQD with length of tunnel L_i, for a unit length L of 100 m as considered by Douglas et al (1983). This was estimated by the following expression:

TABLE 1.- SHEAR STRENGTH IN THE ROCK JOINTS

Material	Internal friction (°)		Cohesion (KN/m²)	
	ϕ_p	ϕ_r	c_p	c_r
shales	20-28	21-23	20-50	0-30
sandstones	20-35	-	40-50	-
limestones	45	30	0	0

$$E^{*} = \frac{1}{\sum_{i=1}^{n} \left(\frac{L_i}{L} \times \eta \times E_{Lab,\ i} \right)}$$

c) Applying empirical relationship between RMR, and indices and E^{*}, specially modified for this work.

Results of E^{*} are ν^{*} values are summarised in Fig. 6. Due to the important constraints to use the elastic model, it was considered a zone partial plastification around the tunnel section, where the deformation modulus were several times lower the rock mass before excavation. The thickness of this zone was taken at least one radious of the tunnel section (Fig. 7). This model was used in a finite element analysis being possible to assess strain-stresses on the supports (Oteo, 1985). The definition of the thickness of the shattered zone due to excavation wheres as follows:

MATERIAL	RQD (*)	DEFORMATION MODULI (GN/m²) DYNAMIC	STATIC MINIMUM	STATIC MAXIMUM	POISSON RATIO
LIMESTONE	66-95	9-23	3-6	7,5 - 14	0,20 0,3
SHALES	40-78	4-13	0,8-1,8	1,5-6,0	0,20 0,35
SANDSTONES & QUARTZITES	50-86	13-16	1,2-4,5	3 - 11	0,18 0,25
QUARTZITES	80-100	—	4 - 6	11 - 16	0,18 0,25
CONGLOM.	91 - 95	—	4,5 - 7,0	11 - 15	0,25

(*) AVERAGE IN THE TUNNEL ZONE

FIG.- 6 RANGE OF VARIATION OF THE ESTIMATED DEFORMATION MODULI

Parameter	Desf. bedding or poor excavation	Fav. bedding and good excavation
a	1 - 1,5 R_m	0,8 R_m
b	0,5 R_m	0,4 R_m
c	0,8 - 1,0 R_m	0,6 R_m
m	5 - 6	4

8. GEOMECHANICAL ZONING

Tunnel trace was divided in different sectors of a similar geomechanical behaviour. Geomechanical classifications were applied, specifically RMR (Bieniawski), Q (Barton), SRC (González de Vallejo, 1985) indices. The results obtained are following summarised:

Rock quality	Class	Tipical form.	Length (%)
Very good	I	Limestones	1,7
Good	II	Limestones sandstones	26,4
Fair	III	Limestones sandstones quartzites	29,3
Bad	IV	shales	40,8
Very bad	V	shales	1,8

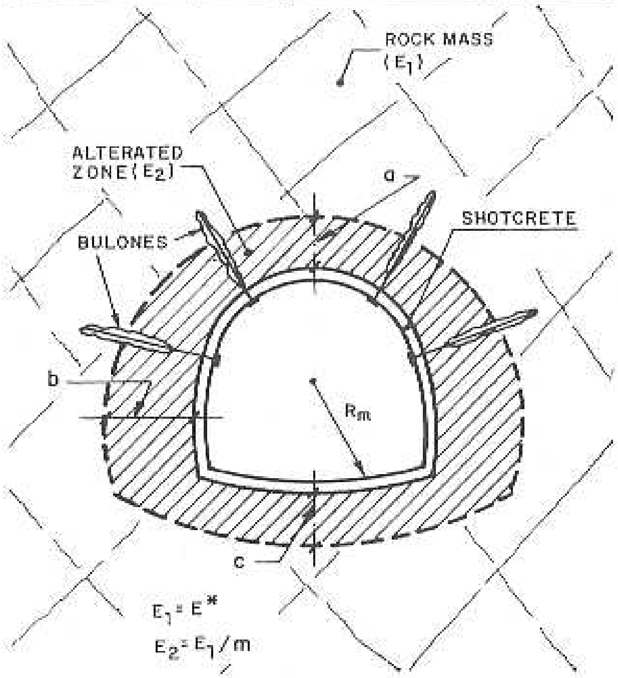
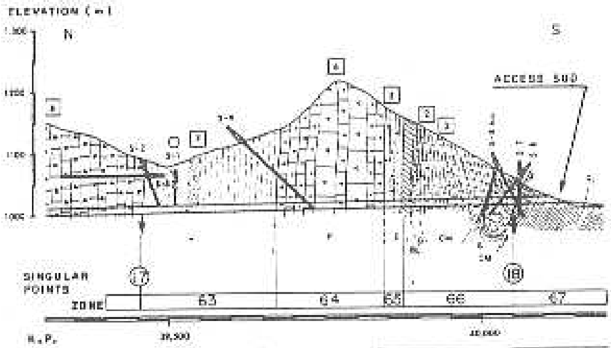


FIG.7 - LAY-OUT OF THE SIMPLIFICATED MODEL FOR DEFORMATION ANALYSIS

An example of this zoning is shown in Fig. 8 where the following parameters were included:

- length of the sector
- lithology
- hydrogeological conditions
- RMR, Q and SRC geomechanical indices
- geotechnical properties (σ_c, γ, E^{*}, ν^{*})
- k (state of stress)
- stability conditions (unsupported length and time)
- excavation methods and support reccommendations

Construction recommendations in terms of stability, support and excavations methods were assed by Mussger (1984) and related SRC indices. The New Austrian Tunnelling Method was reccomended due to the great change in geological conditions. Excavation should be carried out by drilling and blasting but the service tunnel can be excavated by TBM machine.



ZONE		63	64	65	66
LENGTH (m)		200	170	28	180
LITOLOGY		SHALE	LIMEST.	SANDST.	LIMEST.
HYDROLOGICAL CHARACTERISTICS		1	2 - 3	1	2
GEOMECH. CLASIFIC.	RMR	23	64	55	41
	Q	0,3	50	51	2
	SRC	25	86	84	38
GEOTECN. PARAMETERS	γ (T/m³)	2.65	2.70	2.65	2.80
	σ_c (MN/m²)	25	60-100	>50	40
	E^* (GN/m²)	1,8-4,5	15 - 25	4,5-11	3 - 8
	ν^*	0,25	0,30	0,25	0,30
K_o (INIT. STR.)		1	1	1	1
STABILITY CONDS.	U.S(m)	2	10 - 15	6	6
	T (DAYS)	0,4	42	0,8	1
EXCAVATION		EXPLOS. T. B.M.	EXPLOS.	EXPLOS.	EXPLOS.
(SUPPORT) (*)		4	1	1	3

(*) 1 = LOCAL BOLTS (3m) SHOTCRETE (2cm)
3 = GROUTED BOLTS (4m) SHOTC. (10 cm) WELDED WIRE FABRIC
4 = GROUTED BOLTS (6m); SHOTC (20-25 cm); WELDED WIRE FABRIC; INVERT ARCH

FIG.- 8 EXEMPLE OF THE TUNNEL ZONNING

9. CONCLUSIONS

The Pajares tunnel project gave an excellent opportunity to design and planning an extensive site investigation programm for a long tunnel under difficult geological and topographical conditions. This programm was developed during 1983 and 84, and the total cost was about 164.000 U.S.$.

Some of the main results and experiences gained in this project are following summarised:

- In situ investigations for long tunnels need a careful planning and design of the site investigation techniques and adecuate infraestructure and supporting measures. Appart from the technical problems, the logistic selection of the borehole sites, acess and climatological constraints played a very important role on the economic and technical results.
- It is of paramount importance to carried out detail and rigorous geological surveys. All the other works should by based on that geological studies. Further more, this studies represent the highest Benefic Cost ratio in relation with other site investigation methods.
- Site investigation techniques were mainly directly to resolve structural-geological problems and to characterised rock properties.
- Down-hole and well logging geophysics proved to be very useful techniques, and economic.
- The previously discussed methodology proved to be efficient when empirical, observational and analytical methods are used and compared. This procedure was also useful where great heterogeneous and uncertain geological conditions should be assessed.
- The total cost of the site investigations represents a percentage of 2-2.5% on the total projected construction cost for the tunnel.

10. ACKNOWLEDGEMENTS

The authors are greatefull to RENFE for permission to publish this paper, and the Empresa Nacional Adaro de Investigaciones Mineras under his supervision this works was carried out.

11. REFERENCES

BARTON, N.; LIEN, R. and LUNDE, J. (1974). "Engineering Classification of Rock Masses for the design of Tunnel Support". Rock Mechanics. Vol. 6. pp. 189-236.

BIENIAWSKI, Z.T. (1979). "Tunnel Design by Rock Mass Classifications". Pennsylvania State Univ. U.S. Tech. Rep. GL-7919, 131 p.

DEERE, D.N.; HENDRON, F.D.; PATTON, F. D.; PATTON, F.D. and CORADING, E.J. (1967). "Design of Surface and near surface construction in rock". Symp. Rock Mech. 8th Minnesota. USA.

DOUGLAS, T.H.; RICHARDS, L.R. and ARTHUR, L.J. (1983). "Trial excavations for underground caverns at Dinorwing power station". Geotechnique, Vol. 33, nº 44, (407-432).

GONZALEZ DE VALLEJO, L. (1985). "Tunneling evaluation using the Surface rock Mass Clasification System (SRC)". Int. Symp. ISRM. Zacatecas, Mexico.

OTEO, C. (1985). "A semi-empirical model for the design and control of tunnels in rock media". I Taller Hispano-británico on numerical models in geomechanics. CEDEX. El Paular (Spain).

ROMANA, M. and ESTEFANIA, S. (1983). "Field inventory of tunnels for clasification purposes". Int. Cong. Rock Mec. ISRM. Melbourne, D (207-210).