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SITE INVESTIGATION, ITS COST AND BENEFIT IN PORTUGAL

— 2 volumes —

volume I

by

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To my children

GONÇALO
and
MARIANA

ABSTRACT

This thesis comprises two volumes. Vol. I contains the account of the research and Vol. II a record of the site details upon which the research was based.

In Vol. I the most common exploration and testing methods used in rock mass site investigations, specially those for dams and tunnels, are reviewed. These methods are discussed rather than described, both in terms of (a) the benefits that their utilization can bring towards the knowledge of site conditions required for design purposes and (b) their costs. When appropriate, special account is given of the Portuguese practice in this respect (Chapters I, II and III).

Twenty six projects (8 concrete arch dams, 6 concrete gravity dams, 6 earth dams and 6 tunnels) which were designed in Portugal during the last decade were used to assess the costs and the benefits of site investigation. In order to assess the benefits a special approach was devised that permitted benefits to be quantified in a subjective way. For this, the professional geotechnical staff responsible for each site investigation studied were personally interviewed and asked to assign marks for both the benefit of each type of work and for the benefit of the investigations overall for each of the sites studied. The costs of each site investigation work performed are real costs normalized to 1980 prices, as provided and assessed either by the contractors, consultants and the owners of the projects themselves or from data supplied by them.

Benefits and costs are separately discussed (Chapters IV and V) and lead to pertinent conclusions on a wide range of aspects important to owners (clients), their engineers (consultants) and

contractors.

Finally some relationships between the benefits and the costs have been studied (Chapter VI) which reflect and define this aspect of present Portuguese site investigation practice.

In Vol. II, the 26 case histories are presented. Each comprises data about the site conditions and lists all the works performed during the site investigation, taking into account their characteristics and quantities, the time when they have been performed, and their costs. Relevant information which can be useful to future site investigations in Portugal, like the results of mechanical characterization tests, is quoted. The subjective appraisal of benefits provided by the geotechnical staff who were interviewed is also included in total in this volume.

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CHAPTER I

INTRODUCTION

1-1 - GENERAL

The number of important civil engineering structures that have been designed in Portugal during the last 10 years can be considered as unusually great. This has inevitably led to the collection of an equally large volume of information from the site investigations undertaken, concerning site conditions, as well as information about the approach used in conducting these investigations.

The people interested in site investigations in Portugal, mainly those who are either directly or indirectly involved in this task, are aware of the disadvantages of so great much information lying spread out and silent in files of project owners, consultants and contractors. If this information could be retrieved and assimilated it would enable an assessment to be made of what has been undertaken and of how it has been carried out, thus allowing an evaluation of the Portuguese approach to site investigations. Such an assessment would constitute a basis for discussing and eventually improving the approach. Another advantage lies in the fact that all the information collected could provide significant help when planning and undertaking investigations for new sites.

This task is not an easy one, regardless of the country under consideration; the effort required to acquire the information is great and the lack of understanding of owners, consultants, contractors, etc. often makes them reluctant to give access to the relevant information.

The benefits of site investigation are a matter which frequently give rise to controversial discussion as they are difficult to assess. Within this thesis an attempt is made to investigate this

problem and at least to give the subject a better basis for discussion.

The cost of site investigations, on many occasions condition the extent and quality of the studies undertaken. Any discussion of the benefits of site investigations loses much of its usefulness if the inherent costs are not considered as well. It must be said that the costs of site investigations are very seldom quoted and this problem is, once again, due to the fact that obtaining the data depends upon the willingness of those who hold the information.

The difficulty in obtaining information can explain, in part, the reason why a research of this type is only just being carried out now, despite its usefulness.

The access to the information required for this work was gained, after explaining to those involved what it was intended for; the author would like to state that as a result of this the holders of the information - owners, consultants, contractors, etc.- to whom it was necessary to appeal, without doubt understood the usefulness of the job.

1.2- RESEARCH OBJECTIVES

From the above the main research aims can be summarised as follows:

i) To illustrate the Portuguese approach to site investigations for rock masses.

ii) To assess the benefits of site investigations under the Portuguese approach.

iii) To assess what the costs are for the benefits derived.

To achieve the proposed objectives it was decided to restrict the research to dams and tunnels studied over the last decade. This is due to the fact that dams and tunnels are presently the most common important structures available, providing a reasonable number of case histories to deal with. Further, by considering only projects from the last ten years, the risk of giving an unreliable picture of actual Portuguese practice would be, at least, minimized. This has enabled the author to include in this study nearly all the projects designed during this ten year period and as a result problems related to project selection have been avoided.

Within this thesis it was possible to consider 26 independent projects.

1.3 - DEVELOPMENT AND PRESENTATION OF THE RESEARCH

A - A review of existing exploration and testing methods (Chapters II and III)

In chapters II and III an analysis will be made of the exploration and testing methods most commonly used in the site investigations of rock masses for dams and tunnels.

A general assessment of the benefits of the main methods will be given in terms of information, both in type and quality, together with applicability of these methods within the different phases of the studies. Attention will also be paid to the cost and time factors related to the use of each method: the necessary personnel involved will be indicated where appropriate.

Since the methods and techniques used in site investigations are generally known, their description is considered unnecessary.

Consideration of site investigations during the construction phase of schemes is beyond the scope of the present work. The phases considered which form the basis for the development of the design phases are those shown in Table 1.I.

UK (BS 5930, 1981)	PORTUGAL	PHASE OF DESIGN
Desk Study Site Reconnaissance	Gathering existing information and reconnaissance	Feasibility
Examination for design (incl. ground investigation and specialist studies)	Preliminary S.I.	Pre-design
	Design S.I.	Detailed design
Construction follow-up * Performance Appraisal	Construction * and Performance	Construction *

* Not considered in this thesis

TABLE 1.I - Phases of site investigations and corresponding phases of design

In Chapter II, geophysical exploration methods will at first be considered followed by direct mechanical exploration methods which include trenches, shafts and galleries. Boreholes will be considered subsequently, enhancing the importance they normally possess in site investigations.

With regard to testing (Chapter III), attention will be focussed first on the main tests currently used to determine the mechanical characteristics of both the rock material and the rock masses. These tests are most important for the design of dams and tunnels in rock masses as they are related to rock mass strength and deformability. This is followed by particular reference to the index tests associated with mechanical characterization. Consideration is also given to the scale effect relating to mechanical characterisation as this is one of the major problems in the study of rock mass geotechnical behaviour. Finally, Chapter III deals with the more common tests for assessing the "permeability" of rock masses - the Lugeon or Packer tests.

B - Assessing the benefits of site investigations (Chapter IV)

A special approach had to be devised for the assessment of the benefits derived from site investigations for the 26 projects studied. This required the collaboration of the people (referred to as assessors here) who were involved in the studies undertaken for each site.

This approach is explained in Chapter IV which deals with the benefits of site investigations, where the results obtained are discussed and appropriate conclusions are drawn.

C - Assessing the costs of site investigations (Chapter V)

The costs of site investigations for each of the 26 projects are discussed in Chapter V. In order to make a comparison of the costs for each of the different types of site investigation work performed for the same project, and to make a similar comparison between different projects, it was necessary to convert the costs to a common basis. To do this, it was decided to convert the given prices to

1980 equivalent costs.

The discussion of the costs led to some relevant conclusions which are also presented within this chapter.

D - Relationships between benefits and costs (Chapter VI)

Although not initially one of the main aims of this research, some relationships between the benefits and the costs for the different types of works performed during the site investigations are described in Chapter VI.

A benefit - cost graph has been constructed in which different areas dealing with the different types of works used in the site investigations for the 26 projects studies are shown. Subsequently, a coefficient (Return Coefficient - RC) relating the benefits arising from site investigations and the costs of the site investigations was established in order to enable a comparison to be made between site investigation methods. The aim of this coefficient is to show the return provided by each type of work in terms of the knowledge of the site conditions each work provided for the money invested in doing the work. Further, the variation, for a given type of work, of this return coefficient as the site investigation progresses is also discussed.

E - Presentation

This work is presented in two volumes. Volume I comprises Chapters I to VI and Volume II consists of three Appendices (A, B and C) dealing with the factual information which has been gathered, worked out and presented in an easy to use manner.

Appendix A includes 26 sets of data, each dealing with one project. Here, the history of related site investigations is presented. The site conditions are summarized, including a geological sketch of the site, and the works performed are quoted in a chronological and detailed manner. The results of mechanical characterisation tests are also included due to the important reference they provide for other studies. These sets of data are arranged according to project type: concrete arch dams, concrete gravity dams, earth dams and tunnels.

Appendix B deals quantitatively with the marks given by the assessors for the benefits of both each type of work performed and the site investigations overall, for each site: note that nowhere in this work have benefits been described in terms of money.

Finally, in Appendix C, the costs of each type of work performed during the site investigations, are presented in an individual systematic way both in terms of their actual costs when performed and their 1980 equivalent prices. The construction budget and the ratio (cost of site investigation ÷ cost of construction) relationship is also indicated in each case.

A detailed explanation of the information included in Volume II is given within its introduction where the steps required to obtain this information are also described.

Dealing with rock site investigations, this research reflects the fact that more than $\frac{3}{4}$ of the Portuguese territory is composed by rock masses either outcropping or close to ground surface.

In very simple words, the main steps normally followed in the type of site investigations included in this study are as follows (see also table 1.I):-

1- Gathering and studying existing information followed by site reconnaissance which involes a first geological mapping.

2- A first phase of ground exploration and testing, the aim of which is to lead to the geotechnical zoning of the rock mass and so assist pre-design decisions. In this phase it is common to use such techniques as geophysical exploration, trenches, boreholes, galleries and sometimes, laboratory tests. Of the quoted works, trenches and galleries are generally not used for tunnel site investigations. It is generally expected that at the end of this phase either the type of dam or the tunnel position be established.

3- A second phase of exploration and testing works, to assist design, the objectives of which are to complete the information obtained previously, to solve problems revealed by the first phase, to better the geotechnical zoning of the rock mass and to undertake mechanical characterization. Any of the already used side investigation techniques can be used again if necessary, and "in-situ" tests are also performed in this phase, when applicable.

The execution of site investigations in three phases as

illustrated above is not a rigid practice. Due to particular circumstances they can sometimes be extended to four phases or shortened to two phases.

In particular situations short programmes of either trenches and geophysical exploration can prove very useful to assist the reconnaissance.

CHAPTER II

EXPLORATION METHODS

2.1 - GEOPHYSICAL METHODS

Geophysical methods are very commonly used in Portugal both for general exploration purposes and for helping with the mechanical characterization of rock masses by using their results to derive quality indices.

These methods can be considered as indirect methods for obtaining information in site investigations. Their introduction in engineering geology studies dates back about fifty years (ROBERTSHAW and BROWN, 1955), and since then their utilization has grown constantly due to improvements in the adaptation of the methods to different situations in site investigations, and to the accuracy of results required. The short time spent, the easiness of transport of the necessary equipment and the small teams of personnel involved, are the main attraction of these methods.

The methods based on the velocities of the elastic waves and on the measurement of the electrical apparent resistivity are those of utmost interest, being the ones more frequently used in studies of rock masses for dams and tunnels.

In Table 2.I, a summary of these methods is presented and in Table 2.II the type of information is recorded.

The seismic reflexion and gravimetric methods have not been included in Table 2.I because they are less frequently utilised in common site investigations. Other particular methods (down hole logging methods) are referred to in section 2.3 when dealing with the boreholes.

SEISMIC REFRACTION METHOD - Seismic Refraction Profiles	
DIRECT SEISMIC METHODS	- Measurement of wave velocities within: a) Points inside the rock mass (v.g. boreholes) b) Points inside the rock mass and points at surface c) Cross-hole method
ELECTRICAL RESISTIVITY METHODS	- Apparent resistivity profiles - Geoelectrical sounding - Down hole resistivity logging

TABLE 2.I - Summary of the main geophysical methods for site investigation in rock masses.

2.1.1 - GEOPHYSICS AS A QUALITY INDEX

Quality indices for rock masses obtained by other means, have long been employed for geotechnical zoning purposes. In this case, the RQD, the percentage of recovery, the joint frequency etc., can be mentioned, the RQD (DEERE et al., 1966; DEERE, 1968) being the most frequently used.

The quality indices derived from geophysics, to which the travelling characteristics of seismic waves have been used, especially the velocity of the longitudinal waves (Vp), present the advantage of being easy and economical to obtain which has caused interest in their establishment and application.

ONODERA (1963) proposed a quality index based on dynamic elastic modulus determined "in-situ" (e_d) and in the laboratory (E_d) calculated by using equations (1) or (2)

$$E_d = V_p^2 \, \rho \, \frac{(1 + \nu)(1 - 2\nu)}{1 - \nu} \qquad (1)$$

$$E_d = V_s^2 \, \rho \, 2 (1 + \nu) \qquad (2)$$

* Thickness of deposits over the bed rock, depth of weathering/decompression of the rock mass, detection of underground cavities, location of buried services.

* Geological structure of the rock mass including position of bedding, veins, major defects, boundaries.

* Location of water table level.

* Ripability of the formations.

* Obtaining quality indices for the rock material and rock masses.

* Porosity (resistivity method).

* Forecast advance rating for tunnelling.

* Obtaining indices to be correlated with the mechanical characteristics of the rock masses.

* Assessment of grouting efficiency.

* Determination of dynamic mechanical parameters:

  Ed - Dynamic elastic modulus

  d - Dynamic Poisson coefficient

  G - Rigidity modulus

TABLE 2.II - Summary of the main types of information for which the geophysical methods are used.

where: S.I. units

E_d - Dynamic Young modulus of the material (g/cm. sec^2) [N/m^2]

V_p - Longitudinal wave velocity (cm/sec) [m/sec.]

v_s - Transversal wave velocity (cm/sec.) [m/sec.]

ρ - Density of the material (g/cm^3) [m/sec.]

ν - Dynamic Poisson ratio (dimensionless)

This quality index, $\frac{ed}{E_d}$, can be reduced to the simplest expression $\left(\frac{V_p \text{ field}}{V_p \text{ lab}}\right)^2$, assuming equal value for the other variables in equation (1).

In ONODERA's opinion, the decrease of e_d relative to E_d results from the presence of bedding, lamination, foliation, schistosity, jointing and faulting, i.e., discontinuities.

It must be said that although those features generally lead to lower seismic velocities in the case of rock masses than in the case of the rock material, that is $\frac{Vfield}{Vlab} < 1$, this relationship can sometimes be greater than unity due to a high state of stress on the rock mass, a feature not reproduced when determining Vlab. This has been verified, for example in the case of the Capanda dam site, by works carried out by LNEC - LISBON (LNEC, 1972).

Based on $\frac{e_d}{E_d}$ values, ONODERA (1963) proposed a soundness classification for rock masses as per table 2.III, together with geological diagnostics of rock mass conditions to each grade considered.

DEERE et al. (1966) correlated $\left(\frac{V_p \text{ field}}{V_p \text{ lab}}\right)^2$ with RQD - Fig. 2.1, and in subsequent works DEERE and MERRIT (1969) and COON and MERRIT (1970) quoted by RODRIGUES (1979) have statistically shown that these two quality indices are equivalent measures of the quality of a rock mass, especially of igneous nature.

Symbol	Grade	Soundness (e_d/E_d)
A	Excellent	0.75-
B	Good	0.50-0.75
C	Available	0.35-0.50
D	Deficient	0.20-0.35
E	Bad	-0.20

GEOLOGICAL DIAGNOSTICS

A: Fresh, with no alteration; almost no joint, crack etc.

B: More or less jointed or cracked, but with only slight parting; more or less weathered only on the surface along the parting, but inner part is fresh and compact.

C: More or less parted by joint or crack with or without minor amount of interstitial clayey matter, but rock itself is rather fresh with its surface weathered.

D: Parting is rather open and wide, which usually contains clay or muddy materials accompanied by fissure water; rock itself may be partly hard, but generally somewhat weathered or altered and jointed or cracked.

E: Weathering advanced, rock materials altered or weathered as a whole; conspicuously jointed, cracked or crushed.

TABLE 2.III Soundness classification of rock masses according to ONODERA (1963).

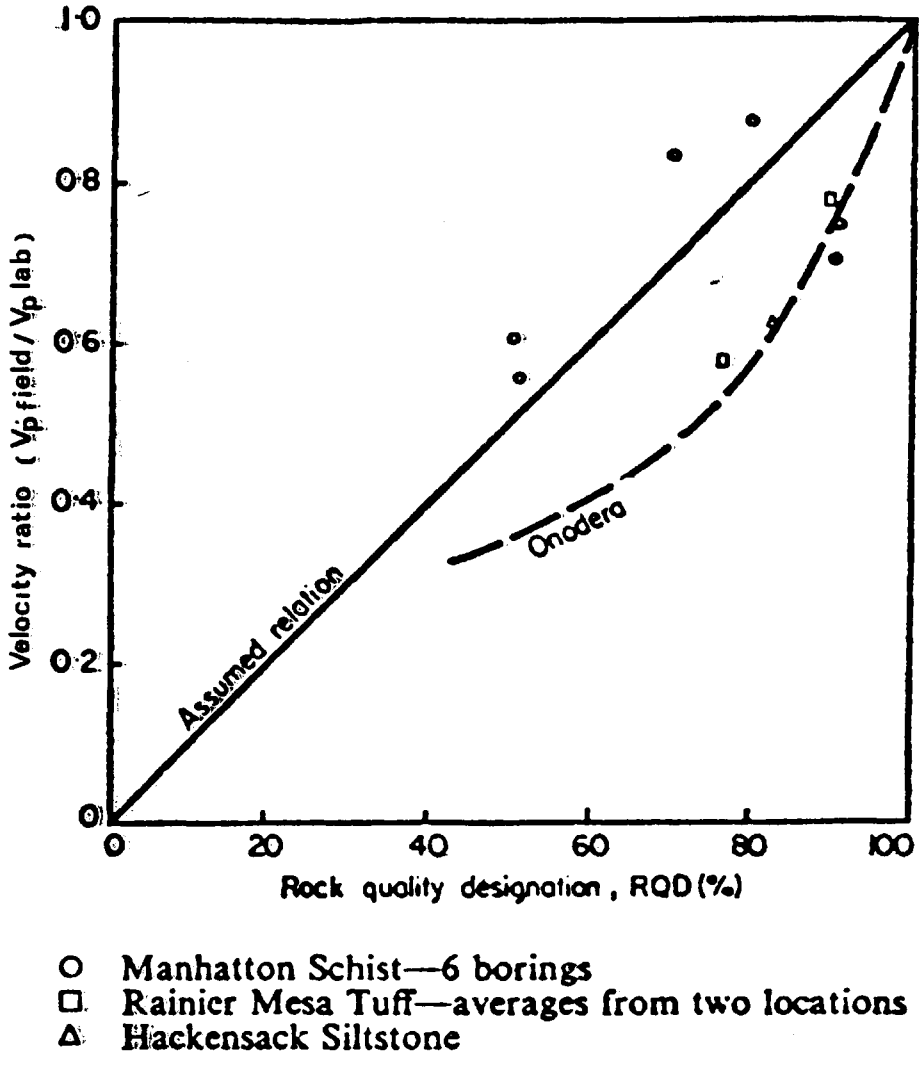


FIG. 2.1 - Correlation of rock quality as determined by velocity ratio and RQD, according to DEERE et al. (1966).

Sjøgren et al. (1979) have correlated the values of V_p, determined by seismic refraction method, with both RQD and joint frequency - Fig. 2.2 - on several types of unweathered igneous and metamorphic rocks at five Scandinavian sites. A divergence of seismic velocities can be expected in similar materials when their overall seismic velocity is less than 3,500 cm/sec. The role played by weathering and alteration in this type of correlation was pointed out by the above quoted authors.

The employment of V_p obtained by seismic refraction as a quality index for assessing engineering classification grades, was reported for example by GRAIGER et al. (1973) for chalk, the results being shown in Table 2.IV. Indentical application was also reported by KNILL and JONES (1965).

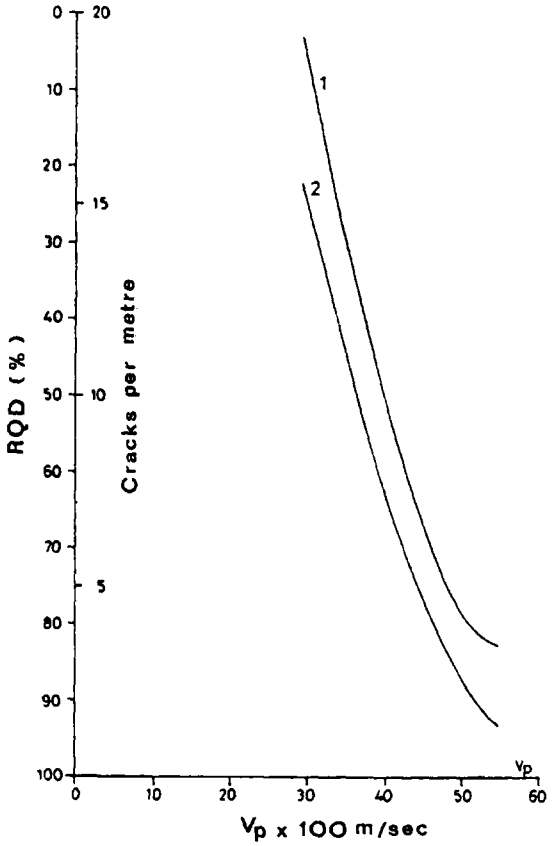


FIG. 2.2 - Correlation between V_p and joint frequency (1) and RQD (2) for several unweathered igneous and metamorphic rock (SJØGREEN, et al., 1979).

RANGE OF V_p VALUES (Km/sec)	MEAN VALUE (km/sec)	CHALK GRADE*
0.65 - 0.75	0.7	V
1.0 - 1.2	1.1	IV
1.6 - 1.8	1.7	III
2.2 - 2.3	2.3	II

* For description of different chalk grades see page 131.

TABLE 2.IV - Correlation between V_p and engineering chalk grades, according to GRAINGER et al. (1973).

Seismic wave velocity values have also been found applicable as an index in the assessment of efficiency of grouting and consolidation works, though determinations made prior to and after the performance of such works. KNILL (1969) refers to the use of the relationship $\frac{V_p \text{ field}}{V_p \text{ lab}}$ as an index in the prediction of grout take.

The attenuation degree of the seismic waves may also be a measure of mass quality and so it is applicable as a quality index (e.g. POLAK, 1971). Generally, the quick attenuation of the amplitude of waves is characteristic of poor quality rock masses. RODRIGUES (1979) quoting other authors points out that the attenuation parameters are more sensitive indicators of rock mass quality, than RQD or travelling seismic velocities, especially in less jointed rock masses.

To obtain attenuation parameters by conventional seismic techniques is, however, difficult and so examples of their employment as quality indices are scarce.

Finally it should be said that as seismic wave velocities are determined for well defined directions, they lose significance as quality indices when dealing with strongly anisotropic rock masses.

2.1.2 - GEOPHYSICAL QUALITY INDICES IN MECHANICAL BEHAVIOUR

It can be said that all the quality indices mentioned earlier can be used as indices of the mechanical behaviour of rock masses. In fact, the variation of quality in a rock mass leads, as a rule, to a variation in the same direction of their general mechanical characteristics.

The velocity of longitudinal seismic waves (V_p) is largely used in the assessment of rock mass ripability or excavatability, bearing in mind the lithology of the geological formations that are present. Empirical correlations based on vast accumulated experience have been used to produce ripability charts as the one shown in Fig. 2.3. These charts make possible a preliminary ripability assessment early in site investigations, some times taking advantage of seismic methods employed for other purposes, thus without additional costs.

The static deformability modulus of the rock masses is one of the most important mechanical parameters that should be known

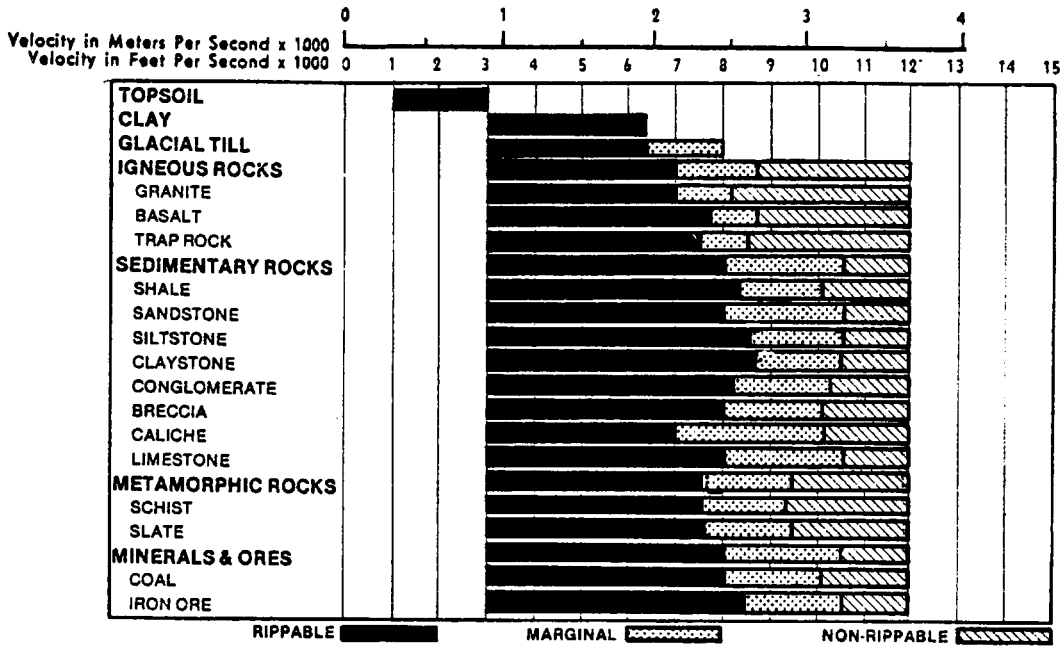


Fig. 2.3 - D9H Ripper performance chart, estimated by seismic waves velocity (CATERPILLAR TRACTOR CO., 1977).

due to its influence in the design either of tunnels or of concrete dams, especially in the case of arch dams. Its determination by using traditional static methods is prolonged and troublesome, requiring particular preparatory works and the use of generally heavy equipment. Furthermore, the costs involved are very high. These have led to several investigations in the search for quality indices which could be correlated with that parameter, essentially with two aims:

1 - to enable a rapid and economical assessment of variations and the establishment of homogeneous zones, regarding rock mass deformability, during that phase of site investigations involved with geotechnical zoning.

2 - to reduce the number of "in-situ" traditional tests (flat jacks, plate load tests etc.) necessary for deformability characterisation, normally carried out during the design site investigation phase, if possible to a minimum for obtaining and checking correlations between their results and the values of the selected quality indices.

The employment of quality indices may, in these cases, lead to benefit/costs relationships drastically lower than the ones normally obtained through the exclusive use of traditional tests. This is due to the low costs in obtaining the needed information and the possibility of increasing information volume also for costs.

In the specific case of geophysical quality indices, the parameter e_d (dynamic Young modulus "in-situ") involving only the determination of longitudinal wave velocities (V_p) has been the most used through empirical correlation with the "in-situ" static deformability modulus (E_{field}) of the rock masses.

Recently, RODRIGUES (1979) gave attention to this subject based

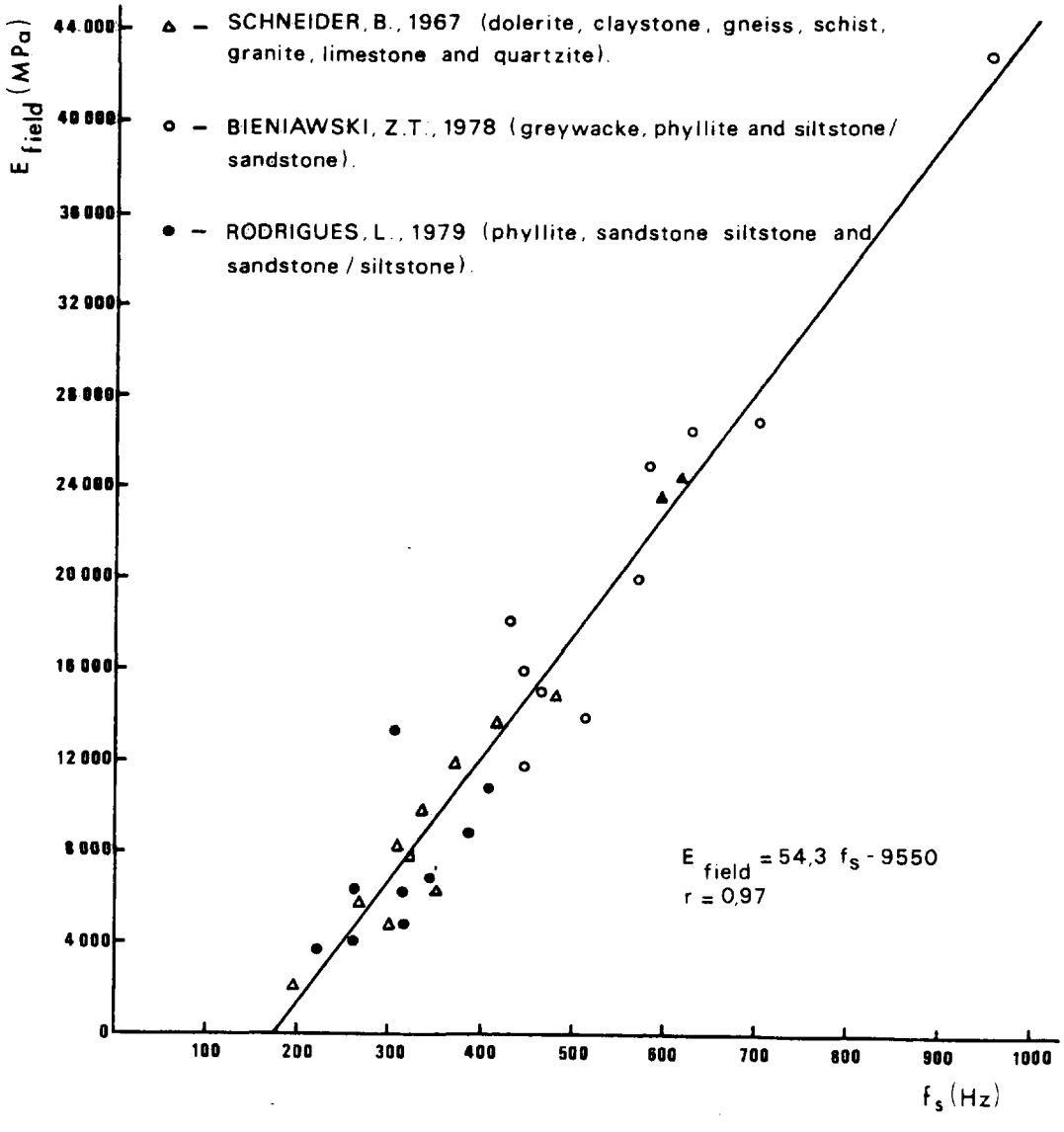


FIG. 2.4 - Correlation between shear wave frequency (f_s) and static deformability modulus "in-situ" (E_{field}); RODRIGUES, 1979 (modified).

on existing bibilography and on results from his own research. He concluded that the relationship between e_d and E_{field} will be valid only for a determined rock mass. The Young's dynamic modulus, acting as a quality index will allow the extrapolation of the results of the static tests performed in a given zone to other zones of the same rock mass.

The shear wave frequency (fs) determined by the so called "Petite sismique" technique (SCHNEIDER, 1967) seems to be gaining importance as a quality index in terms of the benefit it can bring for geotechnical zoning by enabling to foresee the static deformability modulus and for assessing this parameter for design purposes.

BIENIAWSKI (1978) presented a rather close correlation between those parameters with results, some of which had previously been obtained by SCHNEIDER (1967), derived from different rock masses.

Subsequently, RODRIGUES (1979) confirmed the already mentioned close correlation between f_s and E_{field}. More results obtained in two different sites with different lithology, using flat jacks to determine E_{field} were added to existing ones, leading to a correlation expressed by the equation

$$E_{field}\ (MPa) = 5.43\ f_s - 9550$$

with a correlation coefficient r = 0.97 - (Fig. 2.4.)

2.1.3 ACCURACY VERSUS BENEFIT

When thinking to include geophysical methods in site investigation programmes, one should consider not only the applicability of these methods but also the accuracy of the information they can offer. The benefit provided by a method will increase as the level of accuracy which it makes possible approaches that required for the information sought.

As an example, it can be referred (GRIFFITS and KING, 1969; WEST and DUMBLETON, 1975) that in evaluating the depth of a contact between materials with different apparent resistivities, the expected error is of $\pm$ 20%. On the other hand, the accuracy limit for assessing the depth of bedrock by seismic refraction method will be $\pm$ 10%, this figure increasing as the depth increases; the last authors referred to furthermore state that for depths shallower than 1.5 to 3 m, results obtained by using this method are not reliable.

The seismic refraction method can also not be applicable when there is not enough velocity contrast between the overburden and the bedrock. The same applies when the depth of a contact between different geological layers is required and the lower layer does not exhibit a sufficiently higher velocity than that of the overlying one; in this case, the thickness of the upper layer will include the thickness of the lower one.

It seems logical to conclude that by using geophysical methods for the type of information which have been referred, a major benefit will be obtained, in principle, in an early stage of site investigations when the accuracy required is smaller and problems which will have to be clarified later have to be revealed. SAAYMAN (1978), for example, stated that "the proper time to execute a seismic survey is before any other work is undertaken. Execution of geophysical work on a relative undisturbed site is as important as the realisation that the results are most effective in providing a guide towards planning the site investigations". Besides, the above presented examples are sufficient to demonstrate the need for checking and adjustment of the results by means of more accurate exploration methods.

Concerning benefits, one can say that geophysical exploration methods are found to be useful in all phases of site investigation including the construction phase, which will not be dealt with here. In a general way the benefits that these methods can bring depend upon the type of information and the accuracy required in each phase and upon the existing correlatable information obtained by other means.

During the early stages of site investigations, the seismic refraction method and the methods based on electrical apparent resistivity, performed from the surface, are the most used, either applied individually or complementing each other. In this way, a first picture of the rock mass may be obtained, mainly as regards the depth of bedrock the depth of weathering or decompression, the geological structure, the position of water table, and the excavatability of the formations. They can supply valuable information in conjunction with a rather reduced volume of direct exploration works, in certain cases even when these are not available, bearing in mind that a great accuracy is not required at that time.

A good prior knowledge of the regional geological conditions and also of the conditions at the site, are major factors for the

expected benefit.

In Portuguese site investigations the employment of geophysical methods which have been referred to above, is frequent early in the studies: for example to help produce a first comparative assessment of dam sites and tunnel alignments.

It can be summarized by stating that beyond the acquisition of a first model for the rock mass, it is often possible to foresee problems which will have to be solved in further phases of a site investigation and to gain a basis for planning further works in order to obtain the most favourable benefit/cost relationship.

In later stages of the studies the usefulness of electrical apparent resistivity and seismic refraction methods depend upon the ease with which the results can be interpreted either by the experience already gained or by correlation with the other information gathered in the meantime, in accordance with the increasing level of knowledge required. Based on the experience obtained by their employment, these methods can frequently be used successfully, even when applied in isolation.

The direct seismic methods, as they involve velocity measurement of the elastic waves travelling directly through a certain volume of the rock mass, require access to the interior of the rock for generating and/or detecting the seismic signals. This is normally reached through boreholes and/or galleries which, it must be said, are generally not available at the initial phase of the studies.

In the phase of site investigation related to pre-design (or in the first stages relative to design if this document directly follows the feasibility studies) a remarkable benefit may be gained from the geophysical methods from their contribution towards the geotechnical zoning of the rock mass using quality indices.

In table 2.V, the main possible benefits which can be obtained by using the geophysical exploration methods for preliminary and detailed site investigation purposes are shown.

The benefits obtained by using geophysical methods are, even today, put in doubt by many people who are possibly influenced in a negative way either by their personal experience or by their knowledge of cases of failure when these methods had been applied. It is true that such failure may occur mainly when the conditions are not

PRELIMINARY INVESTIGATIONS	* Assist to assess a first model for the rock mass conditions: - Depth to bed rock - Depth of weathering or decompression - Depth to water table * Help to detect buried services * Preliminary assessment of excavatability * Preliminary assessment of the mechanical characteristics of rock masses * Anticipate problems to be solved later * Help in geotechnical zoning * Help planning further investigations
DETAILED INVESTIGATIONS	* Help to better the model of the rock mass conditions and investigation of particular zones based on previous knowledge and experience * Help to assess mechanical behaviour of rock masses * Anticipate advance rating for tunnelling * Obtain dynamic mechanical characteristics of the rock masses * Evaluation of grout efficiency

TABLE 2.V - Main possible benefits of the geophysical exploration methods for preliminary and detailed site investigation purposes.

appropriate for their use, the methods selected are unsuitable or the results are not interpreted in a critical and convenient manner.

When referring to seismic methods, BURTON (1976) stated that "the interpretation of seismic records is very much a question of skill and experience of the interpreter". It is agreed that skill and experience are relevant not only in this case but to all that concerns site investigations and it is important to underline that geophysical methods should not be taken as substitutes for the traditional mechanical exploration methods as can sometimes happen. The advantage that can generally be gained is a decrease in the quantity of the other works needed for the acquisition of a certain type and level of information, hence a decrease in their costs and therefore in the total cost of site investigations.

On Table 2.VI, the main reasons that may lower the success or bring failure in the employment of geophysical methods are presented, the first six points having already been quoted by WEST and DUMBLETON (1975).

* Geology or site conditions unsuitable for geophysical methods.
* Poor initial specification for the geophysical survey.
* Inadequate liasion between client, consulting engineer and geophysical contractor.
* Use of incorrect geophysical method, equipment or field technique.
* Wrong time of geophysical survey in the site investigation programme.
* Bad or inadequate interpretation or misleading presentation of geophysical results.
* Interpretation and acceptance of results without the evidence of enough direct trial methods.
* Disregard of accuracy level provided by the methods.

TABLE 2.VI - Reasons that may lower the success or bring failure in the employment of geophysical methods.

2.1.4 - GEOPHYSICAL METHODS: GENERAL DISCUSSION

A knowledge of the factors that influence the benefits from using geophysics helps an assessment of the factors that should be considered when assessing the cost of using these methods. The time and costs associated with the most commonly used geophysical methods in Portugal are here considered.

Seismic refraction profiles

The necessary time and costs for the execution of seismic refraction profiles vary, in principle, with local conditions. Adverse topography and vegetation can substantially reduce work performance.

Investigations at great depths[1] will imply longer lines (distance between the shot point and the last geophone of the array). This increase, if considerable, will lead to longer time to lay down an equal number of geophones, thus increasing slightly the final costs.

In order to give an idea about execution time it is advisable to consider a real situation. Consider a profile with a length of 60 m, using a 12 geophone array at sites where topographic and vegetation conditions are not adverse, and using the direct and reverse shots technique. The execution time will vary between 1.5 and 2 hours. That enables a well trained team to carry out 5 to 7 of those profiles in the same area per complete working day.

The cost of one of such profiles, including their interpretation varies (considering 1980 current prices in Portugal) between 2.3×10^3 and 3×10^3 Escudos, i.e., a cost close to that of drilling 1 metre of NX borehole [2].

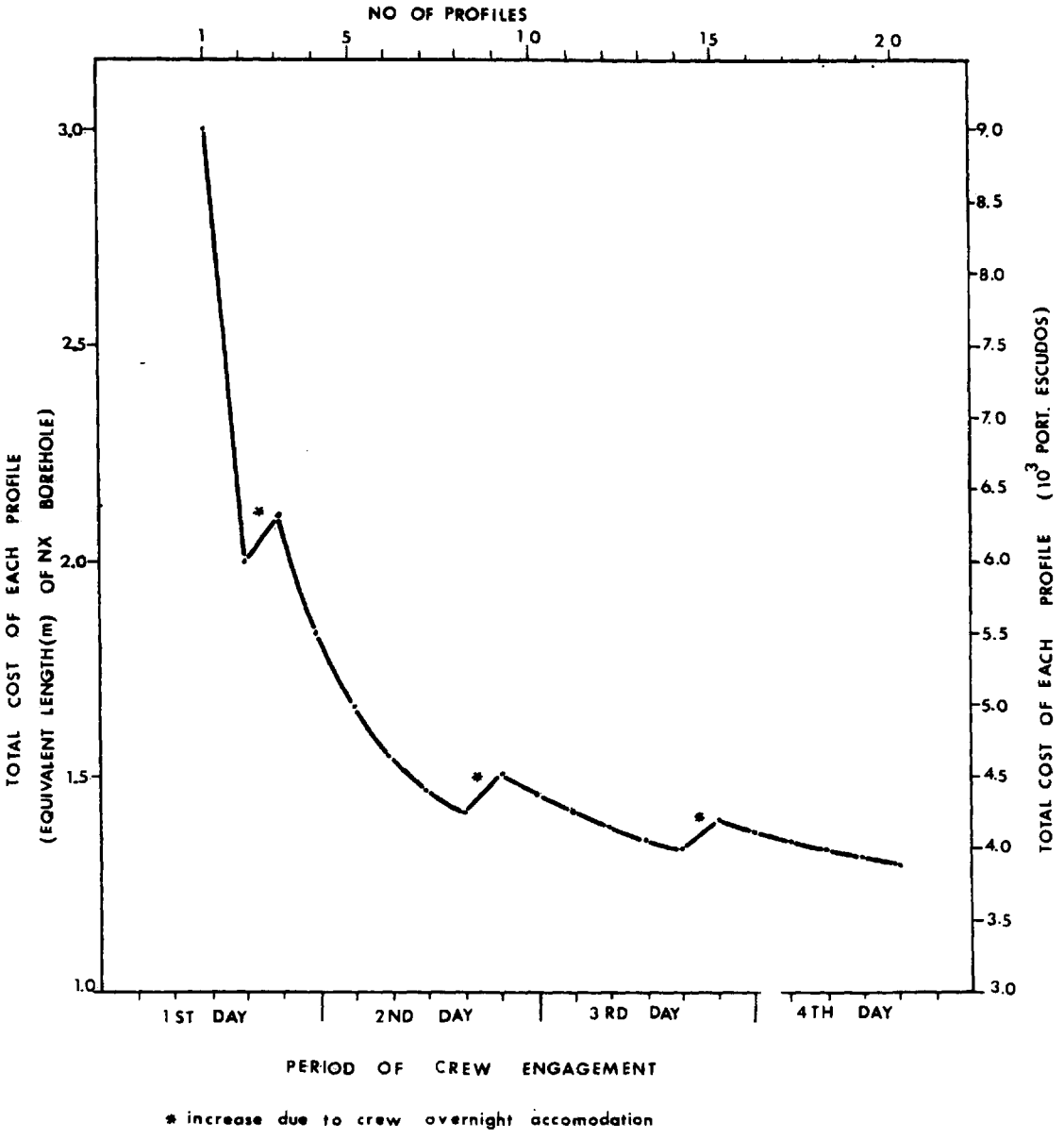
When attempting cost optimization, which is common to all site investigation methods in which the data acquisition is clearly cheap

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(1) The depth investigated by seismic refraction method is about 1/4 of the length of the line.

(2) The cost of 1m of NX borehole used as a comparative term throughout the text refers to only the cost of the drilling operation (2.8×10^3 - 3×10^3 Portuguese Escudos at 1980 current prices). This comparison has been deliberately chosen because so many clients mistakenly compare prices in this way.

when related to the travelling and permanency costs of the crew and/or equipment, which are generally fixed for each site, advantage must be taken of the crews and equipment in the field being able to obtain the greatest possible volume of useful information.



USED VALUES:

TRAVELLING EXPENSES (RETURN)	– 6×10^3 ESC.
CREW OVERNIGHT OCCOMODATION	– 4×10^3 ESC.
EXECUTION OF EACH PROFILE	– 3×10^3 ESC.
COST OF 1m NX BOREHOLE	– 3×10^3 ESC.
TRAVELLING TIME (ONE WAY)	≡ TIME OF 2 PROFILES

FIG. 2.5 – Number and unit cost of seismic refraction profiles made in a period from 1 to 4 days of crew engagement.

The theoretical example in Fig. 2.5 seems to justify the attention that this aspect should merit from anyone who plans this type of work. The graph shows the cost of each profile, after a given number of them which can be made in a period varying from one to four days of crew engagement in a site 200 km distant from a base. Note that in the above example geophysics includes the cost of crew and travel whereas the cost of borehole metres is only that of drilling.

If, for example only one day of work is considered, the travel to and from site will consume a time equivalent to performing four profiles; thus, only two can be made, each costing the equivalent of 2 m of NX borehole. Notice that the performance of a third profile will imply the crew staying overnight. After four working days, the total assessed number of executed profiles will rise to twenty and the cost of each will be 40% lower than the one previously indicated; this is the equivalent to 1.3 times the cost of drilling one meter of NX borehole. The graph also shows that in this case the reduction in unit cost of profiles lessens from the complete third day of work onwards.

Seismic fans

The execution of seismic fans in order to determine direct longitudinal wave velocities along several directions in a given rock mass volume implies, as a rule, the existence of previous exploratory works such as boreholes or galleries. Because such excavations are normally created for other purposes and then later utilized for seismic fans, their cost will not be included here.

1.5 to 2 hours can be taken as the average time, involved in performing one seismic fan, using a 12 geophone array. Nevertheless it must be pointed out that by increasing the number of fans whilst maintaining geophone locations the time needed to perform the next fans may be shortened. For example, after the first fan had been executed, it may be possible to make another two in the following hour.

Based on the above, the cost of a series of seismic fans depends beforehand upon the existence or non existence of a variation in geophone location for the different fans to be carried out. Taking that aspect into consideration, the unit costs of seismic fans and their interpretation vary, according to 1980 Portuguese current prices, between 2×10^3 and 3×10^3 Escudos, this being from .7 to 1 times the average cost of 1 m of NX borehole.

Cross-hole Method

This method implies the availability of duly spaced boreholes. Besides the determination of longitudinal wave velocity (V_p) the method also makes possible, through the use of adequate techniques and equipment, the determination of shear wave velocity (V_s) and its frequency, the importance of which has been mentioned previously.

The use of special equipment and techniques means that this method is not as quick as the ones already considered. A variable to be accounted for, in this case, is the depth of the boreholes and the test frequency along them. Systemmatic measurements one metre apart, using three boreholes 25 metres in length (one for placing the wave generator and the remainder for placing the geophones), carried out after the drilling is finished, will consume about 3 working days according to LNEC-LISBON experience.

Based on the conditions described in the previous paragraph and considering only the expenses of the geophysical crew, the cost will be about 1×10^3 Escudos at Portuguese 1980 current prices, being equivalent to around $\frac{1}{3}$ of the cost of 1 m of NX borehole, per metre of depth tested by the cross-hole method.

If boreholes are sunk purposely in order to perform the method, as very often happens, their cost will constitute an item to be added.

Finally, it must be noted that in the case where the wave generation is made at the bottom of the hole (not considered above) it will be necessary to take into account that the test execution time will constitute an unproductive time to the crew engaged in drilling operations. On the other hand, after a test execution the geophysical team will have to wait for the boreholes to be resumed and the required depth for the next test to be reached.

Electrical apparent resistivity profiles

Also in this case topography and vegetation, if adverse, influence time taken and execution costs. For the apparent resistivity profiles, the measuring stations are normally 5 m apart, implying the movement of the entire line between stations.

The time involved depends on the number of readings (stations) required which is related to the total length of the resistivity

profile. A 100 m profile, for example, involving 20 readings will take, under normal conditions about 1 hour to complete. A well trained geophysical team will be able to accomplish around 10 profiles of this length, in the same area, during one day.

The cost of apparent resistivity profiles is better assessed by giving an average value for the cost of each reading, because the final cost will depend upon the number of those. At 1980 Portuguese current prices each reading costs about 50 ESC. Thus, in the preceding example of a 100 m long profile, the cost will be approximately 1×10^3 ESC. This means the execution of three of those profiles, in a given area will cost as much as 1 m of NX borehole.

Geoelectrical soundings

The time needed to perform geoelectrical soundings, as well as their cost, are related to the depth investigated. In practical terms this can be taken as $\frac{1}{3}$ of the maximum separation between the current electrodes A and B (v.e. ROBERTSHAW and BROWN, 1955; ZARUBA and MENCL, 1976).

Time spent in each geoelectrical sounding depends on the number of readings required which depends, by its turn, upon the depth to be investigated.

On a sounding characterised by a maximum $\overline{AB} = 100$ m, involving 13 readings, it will be expected that a geophysical crew performing reasonably will be able to accomplish a number of six per working day in the same area.

In the case of the preceding example (max. $\overline{AB} = 100$ m) and considering the average 1980 prices in Portugal (80 ESC. per reading), each geoelectrical sounding would cost about 1×10^3 ESC., similar to the cost for a 100 m long resistivity profile. Therefore, three of those soundings would cost the same as 1 m of NX borehole.

Table 2.VII summarizes the costs and execution times for the different geophysical methods, using the examples quoted in the text. The number and category of the personnel involved in each case is also quoted. For this, a scale of categories was established as follows:

METHOD	COST AS METRES OF NX BOREHOLE	EXECUTION TIME	PERSONNEL INVOLVED	
			NUMBER	CATEGORY
1 Seismic Refraction profile, 60 m long, (using a 12 geophone array, with direct and reverse shots)	0.3	1.5 - 2 hours	1 1 1-2	A C/D F
1 seismic fan using a 12 geophone array)	0.7 - 1	1.5 - 2 hours	1 1 1	A C/D F
Cross-hole method (systematic measurements 1 m apart, using three 25 m long boreholes) carried out after the drilling is finished	8	3 days	1 1 1	A C/D F
1 electrical apparent resistivity profile, 100 m long	0.3	1 hour	1 1 1	A C/D F
1 geoelectrical sounding (using a max. AB = 100 m, involving 13 readings)	0.3	1.5 hour	1 1 2	A C/D F

TABLE 2.VII - Costs, execution times, and personnel involved, for geophysical exploration methods

A - Graduate senior
B - Graduate junior (less than 5 years experience)
C - Experienced technician
D - Junior technician (less than 5 years experience)
E - Specialized worker
F - Local worker

As before, the costs for the geophysical methods are given as metres of NX borehole (cost of drilling operation) and they include only the costs for the crew to perform the method and for the interpretation of the results, based on LNEC - Lisbon 1980 current prices.

The teams usually involved in the execution of the geophysical methods are relatively small. The greater number of local workers in the case of the apparent resistivity profiles is due to the constant change of electrode position between the different measuring stations. For geoelectrical soundings, because the potential electrodes remain fixed or vary very little, only two workers are needed, one to move each of the current electrodes.

2.2 - TRENCHES, SHAFTS AND GALLERIES

In the next pages, the mechanical exploration methods which lead to the direct observation of the rock masses - trenches, shafts and galleries - will be reviewed focussing particularly on their employment in studies for dams and tunnels in Portugal.

Boreholes, will be dealt with separately, in 2.3, because of their importance, being the more versatile form of mechanical exploration and also the one most frequently used.

Excepting trenches, the mechanical exploration methods constitute also an indispensable mean of access to the interior of the rock mass to perform "in-situ" tests. If these methods are carried out at the end of general site studies, a great part of their benefit can be wasted.

In any particular phase of investigation, the establishment of a plan of works that considers the information needs and which has enough flexibility to enable any modification suggested or imposed by the development of the works to be carried out, should be attempted. This emphasises the need for knowledge, training and skill on the part

of the people who are responsible for the site investigations and the need for close supervision of the work. A good knowledge of the structural behaviour of the proposed engineering works is also required and close cooperation between the engineering geologist and the structural engineers and other specialists must be maintained.

2.2.1 - TRENCHES AND SHAFTS

Investigation using these types of works is generally carried out to expose the rock mass that is reasonably close to the ground surface. In trenches, the depths do not currently exceed 3 to 4 metres and a width of 1 to 2 metres is most commonly used. Widths of 3 metres are frequent when bulldozers are used. Shafts can reach greater depths, nevertheless they are seldom deeper than a 12 metres.

The essential difference between these two exploration methods is as follows:

a) trenches enable a linear investigation along the terrain permitting both easy changes of direction and lateral branches when these are suggested by the conditions encountered. The common limitation to their performance or extension in terms of practicality is that caused by adverse topographic conditions.

b) shafts allow greater depths to be reached but deal with restricted areas in relation to the overall area usually involved by the studies. Any lateral investigation from the shafts generally proves to be of very little practical use and expensive, the increase cost being due to the necessary safety of the excavations.

Safety considerations can limit, in economic terms the depth of excavation. In soils, timber lining must be required for depths greater than 3 meters (OLIVEIRA, 1975). In rocks, the excavation becomes more difficult as the depth increases, due to the frequent observed improvement of material quality.

Using back actor excavators, the depths of trenches will be fixed by the characteristics of those machines and the light equipment most currently used for this purpose does not reach more than 3 to 4 metres. Sometimes, trenches for dam site studies call for the use of blasting.

The dimensions of a shaft will be the minimum required for its excavation by the selected methods and the tests to be performed.

Refering to the employment of shafts in superficial deposits for determining their nature and the depth to bedrock , RICHEY (1964) stated that "a shaft 1.8 m (6 ft) by 0.9 m (3 ft) or so, sufficient for a man to work in, may be sunk to a depth of 9 m (30 ft) and more. The depth will depend upon costs and also the pressure of the soil upon the timber lining". The ISRM (1975) suggested rectangular shafts of 1.2 x 1.8 m as having provided greatest use per cost, while SZECHY (1960) refers to section as large as 3 x 1.5 m to 3 x 2 m.

2.2.2 - GALLERIES

The exploration galleries enable investigation along well defined directions inside the rock masses, presenting versatility similar to trenches (direction change, lateral branches). They are the most practical way to reach deep zones where "in-situ" mechanical characterization tests have to be performed. Generally, galleries are horizontal or slightly inclined towards the portal in order to allow easy drainage, but can have any inclination.

The length of galleries is determined by the local conditions and by the extension of the stresses induced by the proposed structure. Economic reasons very frequently restrict the performance of such type of exploration works, especially their number.

As the length progresses the excavation cost increases due to the longer time required for mucking, reaching the working face, and for the provision of ventilation (ISRM, 1975). When poor quality rock masses are to be excavated, further costs have to be faced in order to provide safe support.

Dimensions for gallery cross sections are, as a rule, the minimum which enable the work progression. The ISRM (1975) quotes dimensions about 1.5 x 2 m^2 as being most often the optimum cost/utility size. For galleries a few tens of metres in length excavated by traditional methods, cross sections of about 1.8 in height by 1.2 m in width, are the most currently used (OLIVEIRA, 1975). These dimensions have to be increased for longer galleries due to the necessity for ventilation and mechanization of much removal. WAHLSTROM (1973), stated that "miners discovered that the minimum size tunnel which will permit working space at the heading is one that is, at least, 2.1 m (7 ft) in height and 1.2 m (4 ft) wide, clear of

the rock walls or tunnel supports. For excavations made by using mechanical means, STEVENS (1973) stated that "probably the smallest drift to drive economically is approximately 1.8 m (6 ft) x 2.4 m (8 ft.).

Fig. 2.6 shows the increase in volume to be excavated for several dimensions of galleries of rectangular cross section, as a percentage of that to be excavated for a gallery 1.2 x 1.8 m^2 in cross sectional area. It can be seen that, for example, a cross sectional area of 1.5 x 2 m^2 will imply an increase in volume of about 40% and such increases have to be paid for.

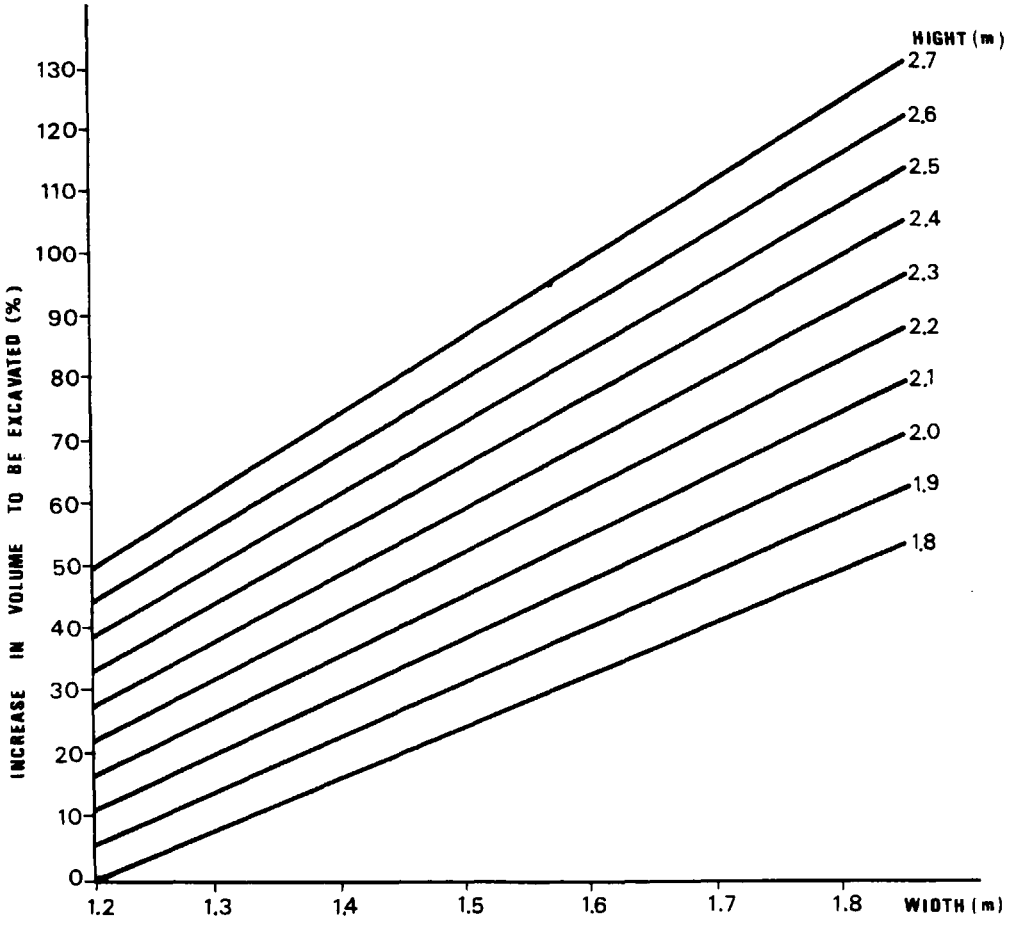


FIG. 2.6 - Increase in volume to be excavated for several dimensions of rectangular section galleries, as a percentage of that for a 1.2 x 1.8 m^2 cross-section gallery.

2.2.3 - RECORDING AND REPORTING INFORMATION

As a rule, the information obtained from the previously mentioned mechanical exploration methods must be reported both in a

descriptive (text) and in a graphical form.

The "interest" on the capital invested in these works is the value of the information they produce. Frequently this works do not provide good value for money due to deficiency in the type, volume and quality of the information obtained or due to a lack of good reporting.

It is important to stress that sometimes a local reinspection of the performed works is not possible later, and that the information obtained has to be examined by different people, either directly or not directly involved in the studies. Therefore, the investigation must embrace all the items which, in the light of present state of knowledge, contribute to the understanding of the actual conditions of the rock mass relevant to the design. On the other hand a broadly known and accepted terminology must be used when reporting the information, rather than any personal terminology.

Regarding shafts and galleries, the data reported must be in line with, if possible, the items shown on table 2.VIII, as suggested by the ISRM (1975).

For the terminology to be used it is recommended that what is suggested by ISRM (1975, 1977, 1980) or by the Geological Society of London (1977) should be adopted.

* Situation drawing (with coordinates and elevation at some interesting points of the gallery), orientation and slope of the floor, dimensions, etc.
* All geological information concerning the rock mass, such as the description of the rock; system, spacing, orientation and nature of joints, faults, etc.;
* the equipment employed for the opening of the shaft or gallery;
* situation, orientation, and references of the samples obtained from the walls, floor or roof,
* situation, orientation and references of "in-situ" tests performed;
* the hydrogeological aspects observed during the opening and maintenance of the excavation, such as moisture, water discharge and its variation with time;
* devices employed for the maintenance of the shaft or gallery.

TABLE 2.VIII - Items to be considered in reporting data from shafts and galleries, according to ISRM, 1975.

Fig. 2.7 is an example of how to record the information in a graphical manner. In this case a gallery using a simple single wall log is shown (ISRM, 1975), this being only accepted for very homogeneous rock masses. Other methods can be used including those considering both side wall logs, as well as the roof or , which is much less practical, the gallery floor (e.g. OLIVEIRA, 1977: TIMUR, 1974; KENDORSKI and BISHOFF, 1976). Nevertheless, a standardization must be attempted.

Similar recording methods must be used for trenches and shafts for the items applicable.

For cylindrical openings, graphical recording is less easy; however, their execution for site investigation purposes is not frequent. Oliveira (1977) suggest a method to apply when dealing either with horse shoe shaped or cylindrical shaped galleries. Other methods can be adapted from the several existing ones dealing with the mapping of circular shaped tunnels (v.g. COOPER, 1968; JACK, 1969).

2.2.4 - USE VERSUS BENEFIT

The benefit from each mechanical exploration method in terms of type and quality of information, the ability to sample and execute "in-situ" tests and other exploratory like geophysics, are greatly variable.

Table 2.IX is a summary of the degree of utilisation of each of those methods for several real situations. The three adopted degrees of use (high, moderate, low or without utilization) can also be taken as equivalent to the benefit provided by each method for each item in the first column. Boreholes were also included in order to present a comparison of all exploration methods currently used.

Trenches

Trenches are of greatest value when used early in an investigation. They make geological mapping easier and permit a preliminar visual assessment of the rock mass characteristics near the surface. They can also assist the interpretation and understanding of geophysical results and the planning of further exploration works. The type of information and the accuracy required in this phase of an investigation means that trenches generally have a high benefit/cost ratio.

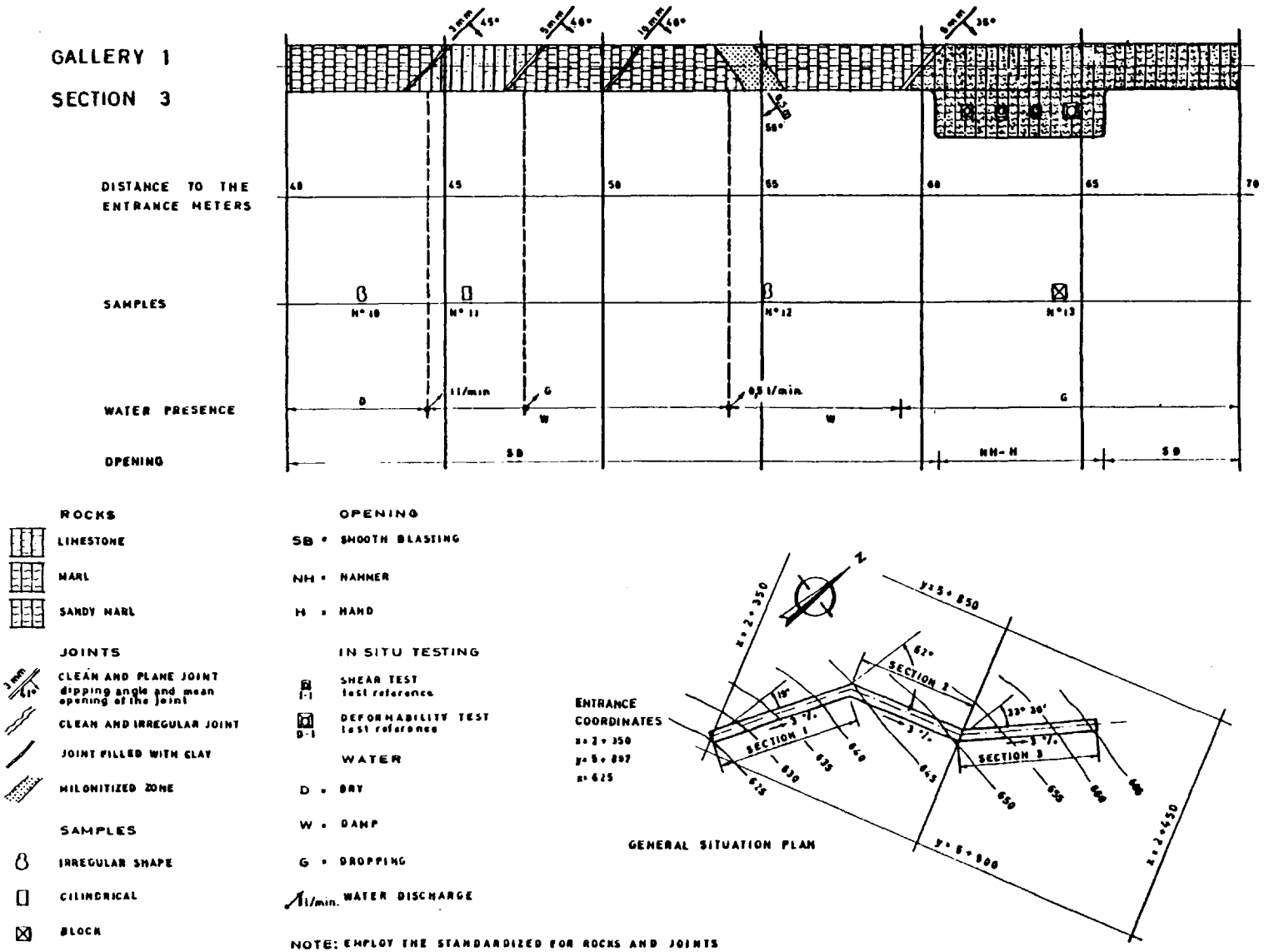


FIG. 2.7 - Method for graphical recording of information from galleries (ISRM, 1975).

Situation	Trenches	Shafts	Galleries	Bore-holes
* Depth of superficial deposits	++	+	-	++
* Lithological boundaries	+	+	+	++
* Depth of weathering or de-compression of rock mass	+	+	+	++
* Assessment of lateral vari-ation of the state of weathering	++	-	++	++
* Study of joint character-istics near the surface	++	+	-	-
* Study of joint character-istics in depth	-	+	++	++
* Study of sliding surfaces	-	++	++	+
* Evaluation of faults and crushed zones characteristics	++	++	++	++
* Attitude of bedding and schistosity	++	-	++	++
* Hydrogeological information	-	+	++	++
* Location of dissolution cavities	-	+	++	++
* Study of geological structure of the rock mass	+	-	++	++
* Assessment of grouting efficiency	+	-	+	++
* Execution of "in situ" tests for mechanical characterization	-	+	++	++
* Execution of geophysical exploration	+	+	++	++

Key: ++ Highly used; + moderately used
- little or not used

TABLE 2.IX - Degree of utilization of mechanical exploration methods for several situations.

The excavation of trenches for dam site studies can reach several hundred of metres (see Table 2.X). In Fig. 2.8 a relationship between total length of excavated trenches and dam axis length for some dam sites is presented. Although a correlation had not been expected, it is clear that there is a pattern. However, statistical checks between the parameters shown in Table 2.X demonstrate there is no sensible correlation. This reflects the differences in site conditions and approach of the investigations.

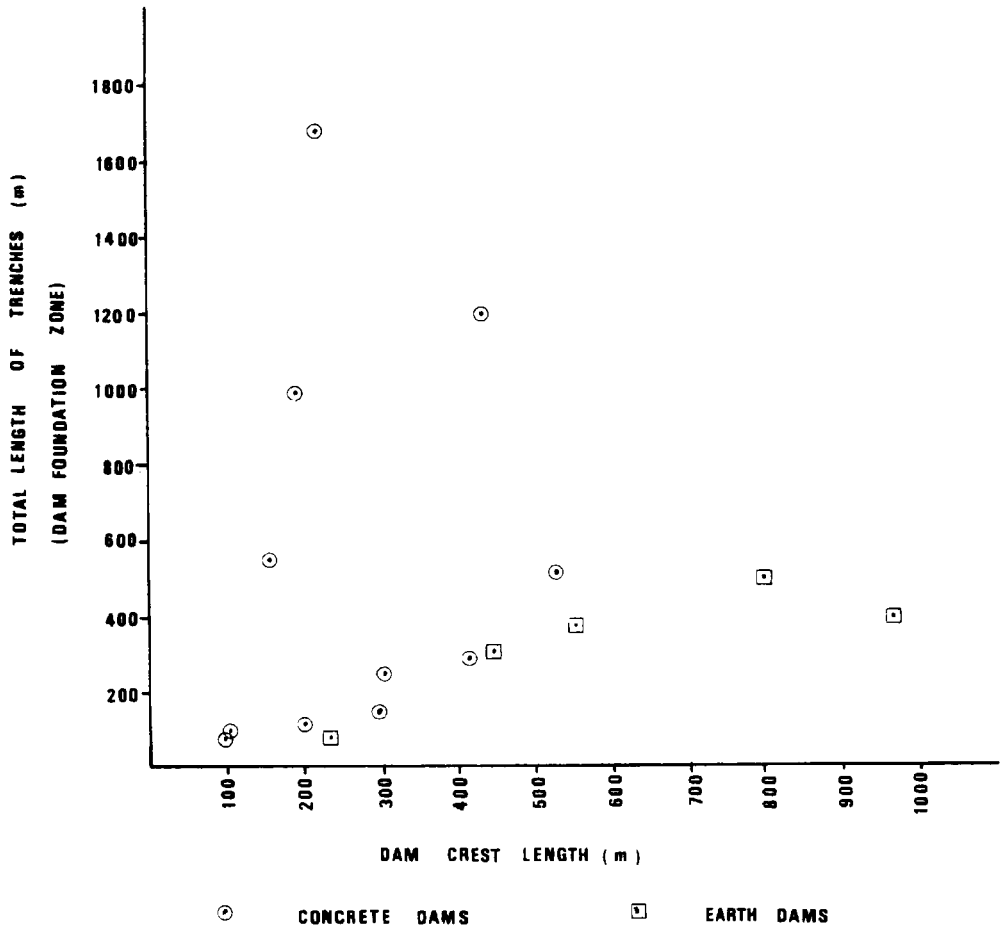


FIG. 2.8 - Relation between dam crest length and total length of trenches for the projects studied.

In the case of site investigations for tunnels, trench excavation is much less used,this being due to the following factors:

a) the investigation depths allowed by trenches usually lie very much above the rock mass zones to be excavated by the tunnel, as opposed to what happens in the case of dams in which excavations begin from the surface.

DAM SITE	TYPE C-Concrete E-Earth	CREST LENGTH (m) (A)	MAX HEIGHT ABOVE FOUNDATIONS (m)	TOTAL LENGTH OF TRENCHES (B)	$\frac{B}{A}$
ALTO LINDOSO(*)	C	297	110	145	0.5
ALVIDE	C	100	25	95	1.0
ATROZELA	C	95	24	74	0.8
FOZ-CÔA	C	443 SPILLWAY :	145	1189 430	2.7
FRONHAS	C	190	62	980	5.2
FUNCHO	C	142 SPILLWAY :	43	283 53	0.7
TORRÃO	C	218	69	1663	7.7
RAIVA	C	200	34	110	0.6
TOUVEDO	C	155	60	543	3.5
VALEIRA	C	300	48	250	0.8
VALHELHAS(**)	C	525	60	510	1.0
AZIBO	E	551 SPILLWAY:	56	370 430	0.7
CAPINHA	E	231 SPILLWAY:	18	75 60	0.3
MEIMOA	E	796 SPILLWAY:	65	491 644	0.6
ODELOUCA	E	440 SPILLWAY:	80	302 240	0.7
SABUGAL	E	960	59	395	0.4

(*) Only in the right bank (**) Only in the left bank

TABLE 2.X - Total length of trenches excavated for dam sites (see Appendix A, vol. II). Quoted lengths concern trenches in the dam foundation zone, except otherwise stated.

b) the length of most tunnels means that rock mass outcrops are generally available from which the necessary geological information sought during the reconnaissance can be extrapolated. Trenches are only considered in order to clarify singular aspects in very localized zones, namely at the portals.

As the site investigation of an area progresses the benefits provided by trenches begin to decrease as account is taken of the information from the previous phases and that proceeding from other works that are usually being performed. At this stage the excavation of trenches can be considered only to clarify problems that have been identified by other investigations. This applies more particularly to dam axis zones, for example to expose areas where faults or shear zones are expected.

It must be appreciated that the investigation of a site does not progress in a uniform way. Hence in a dam the investigation may reach a point where trenches are no longer economic for the main foundation, but are economic for spillway location which may not have been decided by that stage.

Shafts

For assessing the benefits which can be derived from the use of exploratory shafts, these fundamental aspects have to be considered:

1) they accurately reveal variations with depth but are of limited use for assessing lateral variations.

2) they constitute the only method available that allows a first class continuous vertical inspection of the rock mass quality beyond the depth to which trenches go.

3) they permit sampling and "in-situ" tests to be conducted.

The first of these aspects means that, currently, shafts are not employed during reconnaissance, when the aim is to acquire general information about the rock mass. This does not apply to small trial pits which can be very useful in this stage .

Shafts are best used during preliminary studies to solve localized problems identified from the preceeding phase of investigation, and to indicate the reliability of the information both acquired and required.

For studying the ground to be excavated for structures associated with dams (spillways and certain type of power plants), shafts can be employed, but they are seldom used for dam foundation studies. The excavations for spillways sometimes involve great depths, beyond the possibility of trenches, and the use only of boreholes and geophysical methods can prove unsatisfactory.

In the particular case of semi buried power plants, the shafts are, in general, the best method for the study of the foundations and have the advantage of being sometimes incorporated in their excavation, which represents a pay back of part of the spent money.

Considering site investigations for tunnelling, shafts are seldom used and even so, only for tunnels of great importance in which the safety requirement is very high. In general these shafts coincide with accesses or other facilities like ventilation, with their excavation being incorporated, in the final excavation.

The main fact against using shafts only for investigation in tunnels is their costs, bearing in mind the depth that would be necessary and the difficulty of extrapolating the information. Nevertheless, the zones near the portals, which concern lower overburden, constitute particular zones which frequently cause the greatest problems during construction and in these locations the excavation of shafts can be considered of value in terms of benefit/cost.

The costs of shaft excavation (always expensive) increase excessively with depth and for depths greater than 60-90 m it is almost prohibitive (ISRM, 1975).

Large diameter auger piling rigs can now excavate man size shafts to depths of over 30 m (WILSON, 1975). With the very heavy on-site costs for that equipment, this is only economical where a large number of shafts is contemplated. For rock masses, the auger technique is only applicable when the rock is soft or highly weathered.

The rotary drilling technique can be used as in mining works. As a rule, its advantage over shaft sinking by conventional mining procedures increases as the hole diameter becomes smaller or the formation become softer (COLE, 1973). In that author's opinion, penetration rates can vary from as low as about 30 cm to more than about 60 cm per drilling hour for a 1.3 m diameter hole.

Galleries

Galleries are normally not used during reconnaissance and the main reasons for this are the following:

a) - the type, importance, reliability and volume of information they can provide exceed that generally required in that phase.

b) - the cost of these exploratory works is very high when compared to the amount which is commonly allowed to be spent at this phase in the site investigation.

c) - the time required to perform these works is too long.

It is important to note here that surveys are frequently carried out for more than one site in the particular case of dams, or along different alignments in the case of some tunnels and it is only after that the location has been decided, based on the results obtained from other methods, that galleries can be considered.

Regarding dam sites, the great benefit from galleries is revealed in the phase of preliminary site investigations, continuing during the phase of detailed studies; the benefits can be summarized as follows:

a) - the value of the information possibly obtained, regarding its type, quantity, accuracy and reliability, which are in accordance with that required by the corresponding phases of the design. During preliminary studies, the information gathered from using galleries is of utmost importance for geotechnical zoning and for predicting the behaviour of the rock mass.

b) - possibility of their utilization for mechanical characterization tests, performance of geophysical tests, grouting tests and monitoring the rock mass during site investigation and construction.

Sometimes the exploration galleries can still be used later as part of the undertaking; either for drainage purposes or for inspection or for monitoring. These hypotheses must be considered when planning for galleries. Furthermore, if the galleries have functions other than exploratory, it is only reasonable to debit the site investigations with part of their costs: this automatically leads to benefit/costs ratios which are more realistic and more favourable.

The excavation of galleries constitutes a current practice for

studying sites for concrete dams, mainly arch dams, except for those arch dams considered as low [1]. The vertical distance between galleries can be as little as 12-20 m or even less for the most important arch dams. In Table 2.XI the characteristics of galleries excavated for arch and gravity dam site investigations are presented, based on data from the studied projects.

In site investigations for tunnels, the utilisation of galleries (in these cases usually designated by pilot tunnels) along their entire length, is clearly very occasional and even rare. This does not mean that the benefit that they can bring is small. On the contrary, the excavation of such galleries along the alignment of the future tunnel allows a local study of the ground which will actually be crossed by the tunnel. They become a large scale real model which enables a prediction of the problems that will be encountered.

Cook (1976) refering to the benefit of pilot tunnels stated that "no matter how excellent the initial survey and probing, new information is bound to emerge once the excavation has started". The extense length normally involved in excavating a gallery along the entire length of a tunnel is a very expensive task that requires considerable time whereas even for the highest dams some hundreds of metres of gallery may be enough to investigate all the foundation. GARDNER (1968) recognizing the value of the pilot tunnels, stated that "only the largest projects can afford the time and money for a pilot tunnel or where it might have an auxialiary use for ventilation or drainage". On the other hand, OLIVEIRA (1975 a) noted that since a pilot tunnel cannot have a section smaller than roughly 2 m^2, its excavation is only acceptable for tunnels with sections of at least 10 to 12 m^2, therefore excluding the small tunnels [2].

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(1) According with the U.S. Bureau of Reclamation (1977), arch dams can be classified on a basis of their height as:

LOW DAMS - height up to 100 ft (30 m)

MEDIUM DAMS - height between 100 ft (30 m) and 300 ft (100 m)

HIGH DAMS - height greater than 300 ft (100 m)

(1) Regarding their cross sections tunnels can be classified as:

SMALL $< 10\ m^2$

MEDIUM $10 - 23\ m^2$

LARGE $> 25\ m^2$ (adapted from JACOBS 1973)

DAM SITE	TYPE A-Arch G-Gravity	MAX HEIGHT ABOVE FOUNDATIONS (m)	TOTAL CREST LENGTH (m)	NO. OF GALLERIES R-Right bank L-Left bank	TOTAL LENGTH (m)	VERTICAL DISTANCE (m)
ALTO LINDOSO	A	110	297	R-3 L-3	294	≃35
ALVIDE	A	25	100	R-- L-1	40	-
ATROZELA	A	24	95		NONE	
FAGILDE	A	27	63		NONE	
FOZ-CÔA	A	145	433	R-5 L-5	600	≃24
FRONHAS	A	62	190	R-4 L-4	209	≃13
FUNCHO	A	43	412	R-2 L-2	135	≃15
TORRÃO (*)	A	62	214	R-15 L-4	400	≃10 ≃20
JAMBA-IA--MINA	G	20	937		NONE	
JAMBA-IA-OMA	G	46	490		NONE	
RAIVA	G	34	200		NONE	
TOUVEDO	G	60	155		NONE	
VALEIRA	G	48	300	R-1 L-3	149	Excavated at same level
VALHELHAS	G	60	525		NONE	

(*) Refers to 1969 design

TABLE 2.XI - Number, vertical distance and total length of galleries excavated for the concrete dam projects studied (see Appendix A, vol. II).

Nevertheless, sometimes it can be worthwhile to excavate relatively short adits in the inlet and outlet zones for assessing the local conditions, which are normally their poorest at these locations. In the last case, they must have a length sufficient to enable statistically representative information to be gathered.

A pilot tunnel can be incorporated in the final tunnel thus reducing the volume to be excavated. However, one should be aware that it may cause stress relief and deterioration of ground conditions of the main tunnel rock mass. Nevertheless, if this is the case the observations and measurements of stress and displacements at the walls will be of utmost value for the design of the tunnel. So with any excavation that may be incorporated in the final design it should be conducted as it were part of the permanent works.

Pilot tunnels lose much of their value if for reasons other than ground conditions, e.g. hydraulic design, tunnel alignment and depth, undergo changes in a very advanced phase of the studies, and this happens, sometimes, even after all the site investigations are concluded.

Pilot tunnels can also be excavated in the construction phase, from the tunnel working face, maintaining a certain advance in relation to the tunnel progression. However, this is an exceptional practice because of its bad effect upon the ground around the tunnel excavation; it is also difficult to do this when full face machines are used.

As an alternative but not as a substitute to pilot tunnels, horizontal boreholes are frequently employed. They constitute a cheaper and quicker means, causing minimal rock disturbance and do not create safety problems. However, their disadvantages are the type and quality of the information they provide; limitations exist when their length is considered (e.g. MASTENYI, 1976).

2.2.5 - TRENCHES, SHAFTS AND GALLERIES: GENERAL DISCUSSION

The costs associated with gaining the benefits from trenches, shafts and galleries depend greatly upon the subsequent use that can be made of these structures. In this section, the costs of excavation are considered.

Table 2.XII illustrates the rate of excavation obtained for

exploratory trenches executed by manual means and employing crews of 4 to 6 local workmen, carried out for some of the dam projects studied (see Appendix A, vol. II).

DAM SITE	RATE (metres /month)
AZIBO	37
CAPINHA	75
FUNCHO	31
MEIMOA	45
MEIMOA (appurtnant structures)	59
SABUGAL	24

TABLE 2.XII - Performance of trench excavation (soil removal only) for some of the study dams.

When mechanical excavation is possible, rates are much higher and some tens of metres for a working day can be attained at a relative low cost. In the case of Azibo dam for example, a trench 74 meters in length was excavated by that process in 2 working days. According to the data in Table 2.XII this trench would take about 2 months if excavated by manual means; this is about 30 times more. Further, considering the salaries current at that time for local workmen its cost would have been 6 to 7 times that spent using the machine.

Increase in depth, especially in the case of small dimension manually excavated shafts, is a factor which reduces the rate of excavation due to the time required for mucking which becomes longer and longer as the excavation gets deeper. The presence of water is one of the most serious drawbacks which affects excavation, both in practical delay and economics.

The cost of a shaft can be calculated either on the volume excavated (cubic metre) or on the depth. In both cases the unit price varies generally according to pre-fixed depths (e.g. 0-2 m; 2-5 m; 5-10 m etc.) and when the excavation deals with rocks, different prices can be applied according to the state of weathering or

alteration.

As the encountered situations can vary greatly it is believed that no figures can be quoted here for costs and execution time for shaft excavation. Instead, and as an example, the cases of two shafts excavated at Odelouca dam site (see Appendix A, Vol. II) are referred to. Having a depth of 12 m and 14 m with a cross section of 3.2 m^2, their excavation by traditional means through sound rock of phyllites and metagreywackes, took about three months at a cost equivalent to 500 m of NX borehole drilling, sunk in the same site, at that time. Thus, 1 metre of shaft approximately costed the equivalent of 20 m of borehole drilling.

If the excavation is in soils, a shaft with the minimum cross sectional area for a man to work inside can be expected to reach a depth of 3 to 4 m by the end of 2 working days, employing two local workmen, as compared with 4 m per month for hard rock.

Similar to shafts, when contracting gallery excavation, prices can be based either on a volume basis or on a length basis for a fixed cross section. Usually, prices are quoted for determined length

DAM SITE	ROCK TYPE	TOTAL LENGTH OF GALLERIES (m)	EXCAVATION (m)	COSTS/m (Costs as m of NX boreholes)
FOZ - COA	Phillyte and grey-wacke	600	May 78 to Aug 79	7
ALVIDE	Limestone	40	Aug 72 to Aug 73	16
FUNCHO	Mainly shale and greywacke	105	Jan 76 to Nov 76	10
FRONHAS	Phillyte and grey-wacke	209	Oct 75 to Jul 76	5

TABLE 2.XIII Cost per linear metre of gallery as metres of NX borehole drilling.

intervals and for a given rock type; degrees of weathering or alteration can also be considered. Hence the final cost is a direct function of the length excavated and the ground conditions encountered.

In Table 2.XIII the average excavation costs are quoted per metre of length, dealing with galleries at four of the studied dam sites. The costs are refered to equivalent length of NX borehole drilling, which could be utilized in each case, in line with prices at the time of work.

2.3 - BOREHOLES

Exploration by boreholes is undoubtedly the most widely used method of investigation on site, with a large variety of techniques and equipment available. This does not mean that boreholes constitute a panacea for site investigations. Their results must be kept in perspective. They make it possible to sample very restricted volumes of the rock masses in comparison with the volumes usually involved in civil engineering works. On the other hand, the samples so obtained reflect conditions that are sometimes considerably different from those "in-situ". According to DEERE (1963) "rock cores obtained in exploratory drilling are, at best, small and inadequate samples of the "in-situ" rock masses".

In general, it can be said that boreholes are exploration methods that deal with an indirect study of the interior of the rock masses, usually from the surface.

The versatility in their orientation (vertical, inclined, horizontal) allow the investigation of the rock masses along well defined directions and the depths which can be reached are without limitations with respect to the common requirements of civil engineering works. For very great depths, as for tunnels with very high overburden, it is the economical aspect, that is the benefit/cost relationship, and not the practical feasibility that is the criteria for their use.

The main type of boreholes used in site investigations involve the rock core drilling method which permits continuous sampling and an indirect study of the traversed rock mass by studying the recovered cores. Other methods which do not permit core recovery, v.e. rotary, percussive and rotary - percussive drilling are much less frequently

employed for rock mass exploration, and will not be dealt with here. Auger borings are limited to soils and very soft or completely weathered rocks. Percussive boreholes are not economic beyond certain values of rock strength. Rotary drilling using roller cutters is a destructive process, the volume of information provided being small. This method is only worthwhile when the sole aims are to open holes for testing (v.e. permeability, mechanical, geophysical tests), monitoring, detecting cavities, grouting etc. HORNER and SHERREL (1976), refer to the detection of dissolution cavities and the lithology as an application of the air-flush rotary-percussion drilling quoting for this a cost of about $\frac{1}{6}$ of that involved in a similar diameter core drilled borehole.

During the reconnaissance stage, boreholes are generally not employed, because the particular characteristics of the methods used at this stage are their speed and cheapness. In turn, it can be said that preliminary site investigation phase is the borehole phase (cf. Table 1.I). Afterwards, the detailed site investigation phase will give occasion to utilise further boreholes in order to complete the studies.

In the particular case of dams, most times the dam type is selected only after the completion of the preliminary site investigation. On the other hand, alternative sites are still frequently considered during that phase. Thus, in these circumstances, borehole programmes must be designed in such a way that the necessary and sufficient information required to achieve a final decision is obtained. Subsequently, another phase of boreholes will be performed in order to complete the information needed for the design. It must be noted that very frequently boreholes of the first programme concern only the dam axis zone and so the subsequent programme will deal not only with the clarification of problems which arose in this zone but also with the other zones of the rock mass which will be involved in the associated structures. It is clear that the methodology used will depend to a great extent upon the complexity of site conditions and the importance of the dam.

In the case of tunnels engineering conditions (hydraulic aspects for instance) sometimes impose a determined depth and alignment regardless of geology and only one borehole programme is performed. In this case the engineering geologist has no chance of suggesting any modification. Such practice is not advisable as it departs from the general philosophy of site investigations which aims to identify the best combination of engineering design and construction for any

range of geological variables. This means gathering information in stages, by means of works always planned according to the results of the previous ones.

Economic conditions can also restrict the investigation to one programme of drilling.

When planning the site investigations, the engineering geologist is frequently faced with the misfortune of being, for economical reasons, restricted to a small number of boreholes. Under these circumstances, the additional expenditure related to the mobilization for a second programme of drilling can be usefully spent on either a greater number of a longer length of boreholes in the first and only programme of drilling. In other cases, the limited time available does not permit a plan of these works to be made in separate phases. These problems of cost/benefit associated with drilling, alone, clearly show the importance of a very detailed study of the rock cores and of the correct utilization of the "in-situ" tests.

Besides the performance of mechanical characterization and water absorption tests which can also be executed as the drilling progresses, the boreholes can, after completion, be used for monitoring purposes. This means that part of the information that can be gaterhed by monitoring must be accounted as a benefit of the boreholes, and this aspect must be taken into account during the planning when considering the possible benefits and the cost of this type of work. Piezometers, inclinometers, deflectometers etc., are some of the monitoring devices which can be used, being the first ones the most commonly employed.

Inclined and horizontal boreholes

In certain situations boreholes other than vertical are used as they may provide better information; in valley sides especially the same information can be obtained with a saving of borehole length.

Table 2.XIV presents some common situations where inclined boreholes can be used or even be advisable.

Inclined boreholes have, nevertheless, some disadvantages. One of these, is the greater length required to reach a given depth in the rock mass, in zones of flat or near flat topography (see

* Determination of layer and veins thickness and joint spacing, in case of very inclined or vertical attitude.

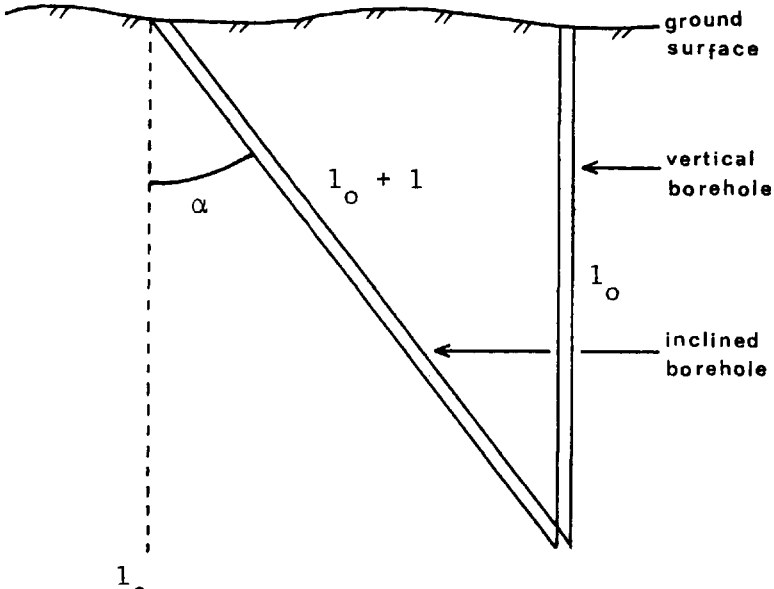
* Detection of vertical or subvertical features (veins, faults, crushed zones, etc.).

* Cases in which the length to be drilled to reach a specfiic zone of the rock mass is smaller than using vertical boreholes (i.e. sites of hard topography like river banks, for dam site investigations).

* Outline and detection of karst cavities as they tend to develop vertically.

* Cases where the general state of jointing is sought because a greater number of sets is traversed.

TABLE 2.XIV - Situations which justify or make advisable the use of inclined boreholes.



$$l_o + l = \frac{l_o}{\cos \alpha}$$

and

$$L\ (\%) = \frac{1 - \cos \alpha}{\cos \alpha} \times 100 = \frac{1}{\cos \alpha} - 1 \ .100$$

$$= (\sec \alpha - 1) \ . \ 100$$

FIG. 2.9 - Increase in length for an inclined borehole to reach the same depth of a vertical one. L (%) is the percentage increase of length.

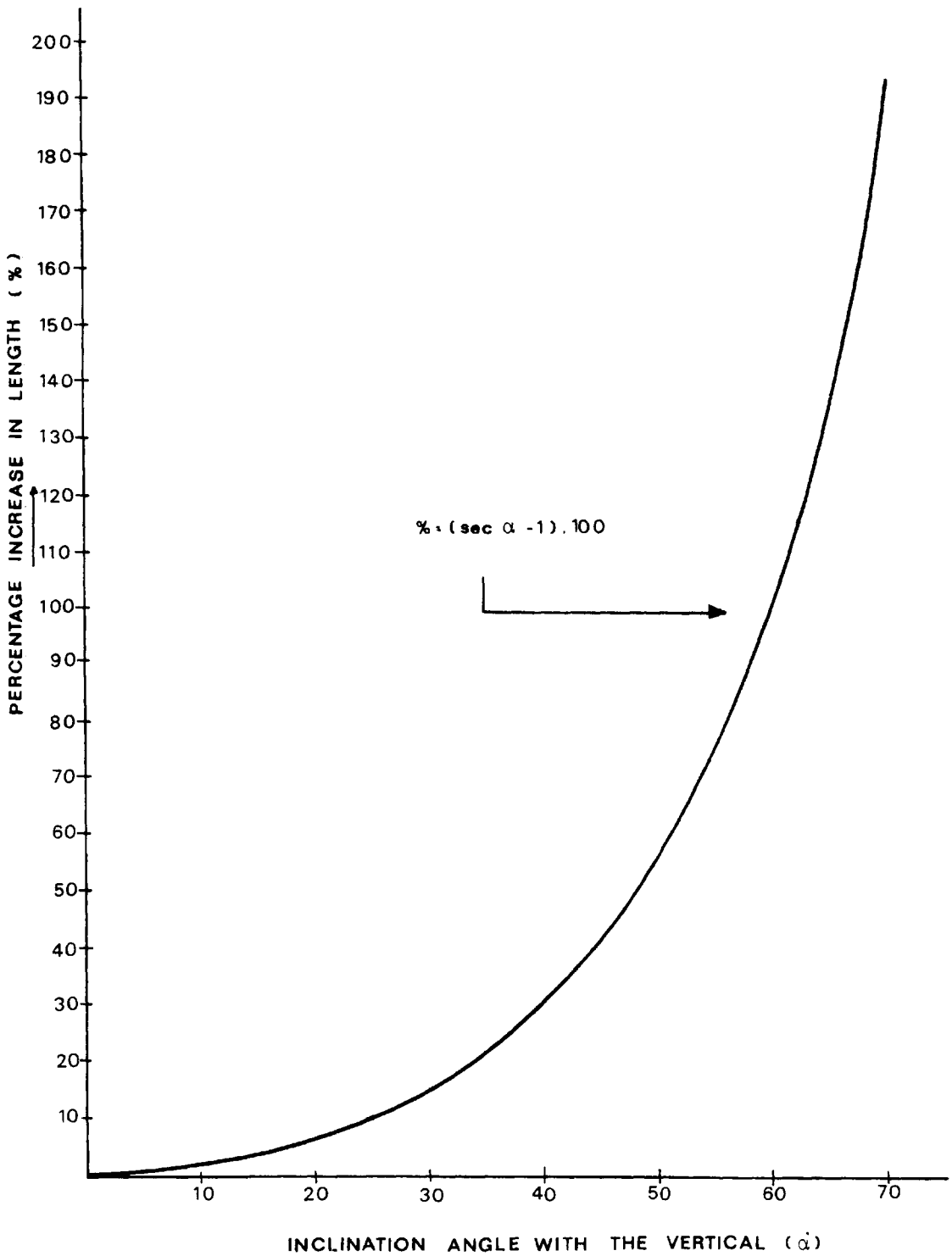


FIG. 2.10 - Percentage of length increase in an inclined borehole in relation to a vertical one, to reach the same depth.

Fig. 2.9 and 2.10), and increasing length means to increase cost. Also the deviation in borehole orientation can be remarkable when dealing with planar structures. This aspect is frequently considered in preparing contract specifications. COATES et al. (1976), for example, stated that for inclined boreholes performed for Kielder tunnel a tolerance of 2^{o} was allowed, increasing to 10^{o} provided the contractor bore the cost of any additional drilling required to reach the tunnel depth. If deviations exceeded 10^{o}, the contractor was required to take appropriate measures to reduce them at his cost.

With great inclinations, some monitoring, hole wall inspection, and geophysical borehole logging techniques can become impracticable.

Horizontal boreholes have more limited applications being used some times for tunnel site investigations, either from the portals or to probe ahead from the working face during construction. These boreholes can also suffer great deviations.

2.3.1 - SOURCES OF INFORMATION

By using boreholes, the possible information can come from different sources, as shown in Table 2.XV.

* Information obtained during drilling operations.
* Information obtained by studying the rock cores.
* Information obtained through inspection of hole walls by using appropriate devices.
* Information obtained by the execution of mechanical and water absorption tests inside the boreholes, down hole geophysical exploration and geophysical borehole logging.

TABLE 2.XV - Different sources of information from boreholes.

Information obtained during drilling operation

It is not common, for economical reasons, to maintain a resident geologist to follow all drilling operations although it would be desirable. Thus, the type of information obtained depends to a large extent upon the training, experience and skill of the foreman: much depends upon him as the success of a borehole (SERAFIM and SEABRA, 1962) depends upon controlling and adapting drilling conditions (e.g. rate of drilling progress, water pressure, trust into the bits) to the encountered terrain characteristics.

* Information regarding strength of the drilled rocks and presence of shear zones, based on the rate of drilling progress and on the type of bits used: penetration rates and bit wear

* Information about hydrogeological characteristics (presence of aquifers, variation of rock mass permability, position of the water table: casing position water returns, losses and need for grouting or use of mud.

* Detection of solution cavities and information about their eventual infilling: rod drops, caving zones and unstable borehole walls.

TABLE 2.XVI - Possibile information during drilling works.

In Table 2.XVI the types of information which can be obtained during drilling are stated.

Information obtained from rock cores

The types of information (potential benefits) which generally can be obtained by studying the rock cores are enumerated in Table 2.XVII.

* Lithological characterisation of the traversed (rock type, texture, fabric).
* State of weathering and their variations.
* Information about bedding, cleavage, schistosity, gneissosity, and foliation.
* Eventual detection of major defects (faults, crushed zones, solution cavities) and the study of their geometric and physical characteristics.
* Information dealing with jointing; their spacing, infilling, roughness.
* Evidences of water percolation.
* Assessment of the rock mass quality by using quality indices such as RQD or core recovery.
* Obtaining rock physical and mechanical characteristics by selecting representative samples and further testing.

TABLE 2.XVII - Potential benefits from rock core study.

Information from inspection of hole walls

Inspecting hole walls requires the utilization of appropriate techniques by employing special devices like calipers, photographic and television cameras, and periscopes (see e.g. BURWELL and NESBIT, 1954; BUTLER and HUGO, 1967; OLIVER, 1976; SCHALKWYK, 1976). The last author, for example, presented a good review of the techniques and methods available. Although such methods provide a means of observing the interior of the rock masses through boreholes, their employment is not frequent. This is because sophisticated and generally expensive equipment is required. In addition, they require a well trained and experienced geologist, involve long periods of execution and need good borehole conditions, particularly clear water and clean sides to a stable wall.

In Table 2.XVIII, the potential benefits from hole wall inspection are summarized. To obtain part of that information depends on the specifications of the devices employed as for example the possibility of inspection under water, assessing distances, depth of investigation

allowed, etc.

In addition to the improvement of knowledge they can bring in the case of rock core drilling, these methods can also be of great use when techniques without core recovery are used.

* Location of water table and zones of inflow.
* Orientation of geological formations.
* Identification and orientation of joints and major defects.
* Inspection of cavities.
* Inspection of hole wall roughness (of interest for the localization of further mechanical characterization tests).
* Inspection of zones where core recovery was not achieved.
* Measurement of joint openings, thickness of bedding etc.
* Inspection of joint infilling.

TABLE 2.XVIII - Potential benefits from hole wall inspection.

Besides the techniques already mentioned others have been attempted. BARR and HOCKING (1976), for example, developed a pneumatically inflatable impression packer for the structural logging of hole walls, to complement routine diamond core drilling. Wall features are impressed on a film around the packer which, when brought to the surface, will permit the study of orientation and opening of joints, voids etc.

Down hole geophysical exploration and geophysical borehole logging

This has already been refered to when dealing with geophysical exploration methods (Chapter 2.1) the utilization of the holes for measuring wave velocities and other parameters between points inside the hole and points at the surface or inside adjacent holes.

The measurement of longitudinal wave velocity can also be used

ACOUSTIC OR SONIC LOGS	* Changes in lithology and porosity of formations adjacent to the borehole * Assessment of the dynamic elastic properties * Correlation with mechanical properties and degree of fracturing and fissuring
RESISTIVITY LOGS	* Lithological correlations * To mark bed boundaries
SPONTANEOUS POTENTIAL LOGS	* Geological correlations * To mark bed boundaries * To detect permeable zones (not quantitatively)
NEUTRON LOGS	* Water filled porosity in non-saturated and non-shaly formations * Water saturation if porosity is known
GAMMA-RAY LOGS	* Useful to determine the lithology and to make lithological and stratigraphical correlations
GAMMA-GAMMA LOGS	* To assess density

TABLE 2. XIX Types of information from the main methods for geophysical logging of boreholes.

for borehole logging purposes as can electrical and radioactive methods. These methods described in several papers (PIRSON, 1963; SCHLUMBERGER, 1972 etc.) are not frequently employed in site investigations, mainly because they require the use of sophisticated equipment which often is difficult to use in the field. In many cases the information which would be possible to obtain by such methods becomes of little value once rock cores are available.

BALTOSER and LAWRENCE (1970), SHALKWYK (1976), KNILL, (1976) and ISRM (1976), are a few examples of the several authors who have pointed to the type of information to be obtained by using these methods. ISRM (1976), produced its suggested methods for geophysical logging of boreholes" seeking procedural standardization.

In Table 2.XIX, the main methods and corresponding types of information are summarized.

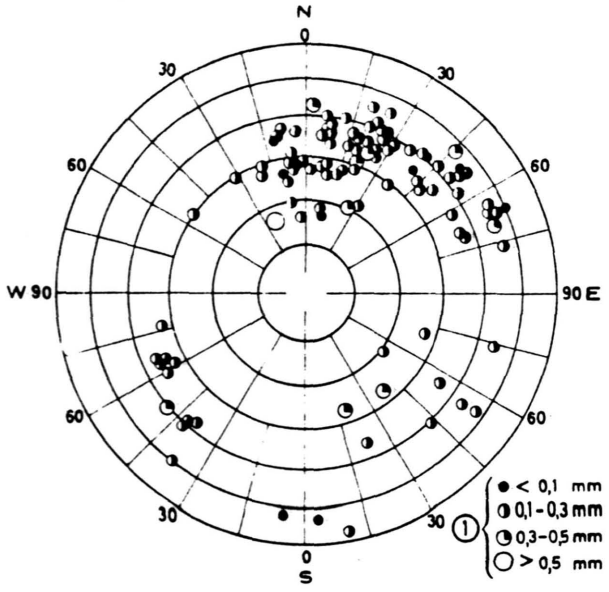
More than one method can be used simultaneously in order to obtain more reliable information, as for example the self potential and the resistivity logging, which are frequently used to draw attention to differences in lithology and permeability (v.e. SHALKWYK, 1976).

Information from integral sampling and oriented cores

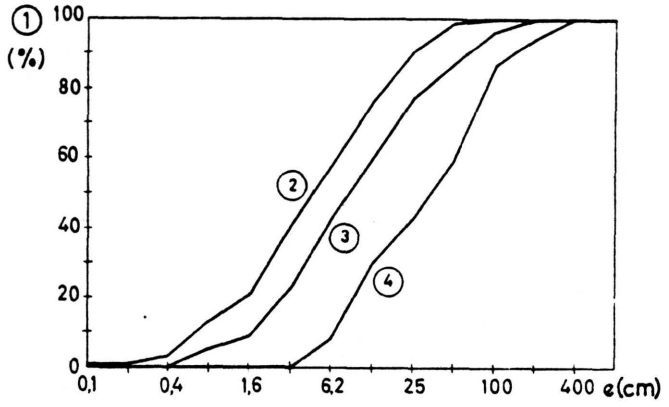
In poor quality rock masses, core recovery by rock core drilling is difficult and sometimes even impossible because the rock desintegrates. Circulating water used in drilling operation can wash out joint infilling making its characterization impossible. These aspects also severly limit an assessment of the mechanical characterization of the rock masses for which the presence of low resistance zones, especially joints and their infillings play a very important role.

The integral sampling method (ROCHA, 1971) enables a continuous and total recovery, having a suplementary advantage in that the samples are oriented with a consequent benefit for the study of joint geometry. The core diameter currently used (LNEC-LISBON) is about 75 mm but it can be greater. The cores can also be tested in the laboratory to study infilling and crushed zones properties, especially regarding their mechanical behaviour.

Fig. 2.11 shows applications of integral sampling for studying the attitude, opening and spacing of joints.



A– Joint attitudes and openings of a schistous formation. Aguieira dam site, Portugal
(1) Joint openings



B– Joint spacing distributions in a sedimentary formation, Cambambe dam site, Angola
(1) Cumulated percentages
(2) Subhorizontal family
(3) Subvertical family, perpendicular to the river
(4) Subvertical family, parallel to the river

FIG. 2.11 - Assessment of A) joint attitude and opening and B) joint spacing, using integral sampling method (ROCHA, 1974).

Costs and time necessary to carry out integral sampling are probably the main factors which make it less frequently used. According to ROCHA (1974), "once the technique has been mastered, the cost is about 1.5 to 2 times the cost of usual sampling and the time required may be twice as long". For OLIVEIRA (1981, personal communication) that cost reaches 3 to 5 times the cost of the usual sampling.

The logging of cores obtained by integral sampling is also more time consuming when related to that for usual rock cores, as the volume of information to be recorded is greater.

It is necessary here to differentiate between oriented boreholes and oriented cores. A borehole can easily be oriented by its azimuth and inclination to either the vertical or the horizontal where as in common rock core drilling it is generally not possible, after core extraction, to reproduce the position it had in the rock mass. This constitutes one of the most serious limitations of the traditional boreholes.

Only when, at least, one well defined, continuous and planar structure, whose attitude is known, is present in the rock mass (bedding, schistosity, foliation, gneisosity) it is possible to assess the attitude of the other structures including jointing, using the known structure as a reference.

In addition to the integral sampling technique which has the advantage of producing oriented cores, several other techniques either photographical or mechanical enable to establish orientation (see e.g. CARLSTON, 1951; BRIDGES and BEST, 1971; SEEGMILLER, 1976). Nevertheless, it seems that none of these methods can be considered at the same time to be practical, efficient and of reasonable cost. It should also be remembered, as supported by the last refered author, that to take advantage of rock core orientation methods "complete recovery of good quality is essential".

2.3.2 RECORDING AND REPORTING INFORMATION

The information gathered from boreholes must be recorded and reported in the form of logs, in order to enable a quick and general assessment of the encountered conditions. These logs should contain the rock core log and other important information gathered from the sources other than the rock core itself (see Table 2.XV). Several methods have been suggested for recording and reporting that information under the general designations of rock core logging methods (for example DEERE 1963; THE GEOLOGICAL SOCIETY OF LONDON, 1970; CORE LOGGING COMMITTEE OF SOUTH AFRICA, 1976; TEPORDEY, 1977).

The logs must be factual reports of the observed and measured features (including test results) as well as of the drilling conditions.

As far as the rock cores is concerned, they must be studied as soon as possible after their extraction to avoid possible changes in their characteristics and hence a decrease in benefit by gathering less reliable information through their study.

Logging must be committed into the hands of a trained engineering geologist (graduate with minimum of three years experience), although it can be sometimes a tedious and boring job. The involvement of young geologists with limited experience in the hope that this will save some money in the logging operations is almost certain to be unsatisfactory, unless very clear instructions are previously given; here the logging codes refered to above are of help.

It is necessary to consider that the gathered information may have to be looked at by different people and so the information must be presented in a clear way. Long and qualitative descriptions must be avoided and quantitative or semi-quantitative data must be quoted instead. Semantic and dubious sentences which can lead to different interpretations must also be avoided. In addition, any interpretation by the logger must be clearly separated from the characteristic factual descriptions (CORE LOGGING COMMITTEE OF SOUTH AFRICA, 1976).

A universally accepted logging method does not exist; nevertheless it seems to be advisable to log according to one of the existing methods which in most cases have been compiled after careful study.

2.3.3 - LOCATION,DEPTH AND SPACING

It is not possible to define rules regarding location, spacing depth or other borehole features like inclination, for borehole programmes. The study of each location must always be regarded as a particular case and the borehole characteristics must be adapted to the geological and topographic conditions, and to the type and importance of the construction.

BURWEL and ROBERTS (1958), quoted by WEAVER (1976), stated that "the extent and quality of such an exploration programme are naturally dependent on the purpose of the study, the importance of the project, the resources available and the completeness required of the investigation results".

Borehole specifications can even undergo modifications during the investigation, in relation to the initially established programme,

if that becomes available in the light of the results that are being obtained. Moreover, and this is a fundamental characteristic in any type of site investigation, such changes should be possible to provide the flexibility which a good investigation needs if the works are to be sensibly controlled by an engineering geologist. Sometimes, it is possible, for instance, to save borehole length or even to cancel some of the programmed ones. In other situations, it could be necessary to increase length in any particular borehole, and in this case it would be economical to make a decision before the borehole has been completed according to the previous specifications.

Although largely accepted as being of value, the possible benefit of the close assistance by an engineering geologist is still not widely appreciated. On several occasions, after a borehole investigation has begun, the results are analized only when a certain number of boreholes have been finished and commonly, after all of them have been completed. These practices can contribute to decrease the benefit/cost of such type of exploration works.

The U.S. CORPS OF ENGINEERS (1973) recommended that it is convenient to use the drilling information as it becomes available so as to modify the original hypothesis, to plan the remainder of the drilling investigation and to obtain additional information for final design.

Although no rules can be made, experience has suggested a certain number of empirical procedures which can be taken as simple guides in the planning of borehole programmes for tunnel and dam site investigations and these will be dealt with in the next two paragraphs.

DAMS

Most times, the valley is excavated along faults or other low strength alignments. Thus, it is a current practice to place, at least, two boreholes at the bottom of each bank, inclined towards the opposite bank, in such a way that they cross each other under the river bed.

The abutment zones of concrete arch dams, as they are the ones which will support the highest stresses, invariably imply also their study by boreholes.

SERAFIM and SEABRA (1962), for example, say that besides other

boreholes in the banks between the bottom of the valley and the abutments, exploration of rock mass upstream and downstream of the dam must not be neglected.

Regarding distance between boreholes along the dam axis, some suggestions are presented in Table 2.XX.

For SERAFIM and SEABRA (1962), the borehole spacing suggested by Lugeon seems to be exagerated and seldom justifiable.

In the author's opinion, for large dams either Terzaghi and

AUTHOR	SUGGESTION/RECOMMENDATION
LUGEON (1933)	Considers to be prudent, for rivers wider than 30 m, that the boreholes of, at least one of the transverse lines are spaced about 3 metres.
TERZAGHI AND PECK, (1948)	Refering to earth dams stated that the minimum number corresponding to a spacing of 30 meters is generally considered. For very large sites, it may be necessary to increase spacing to about 60 m.
HILF, (1973)	Stated that borehole spacing must not exceed 500 ft (150 m).
ISRM, (1975)	"In rock foundations the spacing of borings should be in accordance with the importance of the structure to be built but should never be larger than 25 meters.
MINISTERIO DAS OBRAS PUBLICAS-PORTUGAL. (DEC. LEI 48373/1968)	This Code of Practice for small earth dams (height $\ll$ 15 m), recommends that boreholes must be not less than 3 in number and spacing not greater than 50 meters.

TABLE 2.XX - Some suggestions/recommendations regarding borehole spacing for dam site studies.

Peck's or ISRM's criteria seem to be very reasonable as a working basis. These spacings may be reduced in each case, according to the results obtained.

Table 2.XXI shows some examples of suggested depths to be reached by borehole investigations, regarding dam sites as well. As can be seen, these are criteria based on dam height or on the permeability conditions, or a combination of the two aspects.

AUTHOR	SUGGESTION/RECOMMENDATION
LUGEON, (1933)	Up to depths where absorptions do not exceed 1 LUGEON (dams > 30 m high), or 3 LUGEONS (dams < 30 m high) even though it is preferable to extend the boreholes to reach absorptions of 1 LUGEON.
SERAFIM AND SEABRA (1962)	Up to a minimum depth of 0.8 times the dam height. If at that depth absorptions are greater than 1 LUGEON, the depths must extend until the value of 1 LUGEON is obtained.
HILF (1970)	At least up to a depth equal to the dam height.
OLIVEIRA, (1975b)	Besides the Lugeon cirterion, the author refers that boreholes sunk from near the river must reach a depth equal to the dam height while the ones sunk in the abutment zones must reach the river bed level. In addition, boreholes for assessing the depth of alluvia/colluvia must penetrate 5-10 m in the bedrock.
MINISTERIO DAS OBRAS PUBLICAS-PORTUGAL (D.L. 48373/1968)	This Code of Practice for small earth dams (height ≤ 15 m) recommends a depth at least equal to the maximum dam height.

TABLE 2.XXI - Some suggested borehole depths regarding dam site studies.

Tunnels

Borehole spacing for tunnels is, in several situations, greatly controlled by the thickness of overburden to be traversed for reaching the zone to be excavated, which is that of most interest. Thus, the benefit/cost relationship can play here a decisive role on the number of boreholes to be executed. It is very easy to understand that for the same kind of information gathered at tunnel level, this being the benefit, that relationship will become smaller as the overburden increases.

In Table 2.XXII, two suggested borehole spacings for tunnel investigations are indicated.

AUTHOR	SUGGESTION / RECOMMENDATION
DEERE, (1969)	"Tunnels should have, at least five boreholes, two at the portals and three along the tunnel". In addition, the following spacings are suggested: DEPTH OF EXCAVATION — SPACING 100 m — 30-150 m 100 - 250 m — 150 - 300 m >250 m — 300 - 750 m
OLIVEIRA, (1975b)	In principle, a spacing of around 50 m which may be increased to 500 m in the case of long tunnels in relatively simple geology.

TABLE 2.XXII - Suggested borehole spacings for tunnel investigation.

Regarding the first suggestion above, the quoted author suggested, nevertheless, that "in areas where the rock is considered to be of generally poor quality the spacing should be decreased. But, it should not be increased even if the rock mass appears to be relatively continuous and uniform". In addition, he stated that his proposed guide must be tempered by taking into consideration the results of the

preliminary geological studies.

The zones at the portals generally exhibit the poorest rock conditions due to the low overburden, and all the rock mass is involved in the opening. Experience has shown that it is advisable to place, at least, one borehole in each of these zones.

In the case of irregular topography with high overburden variations, some of the boreholes should be sunk in the lower points, even if this means disregarding variations in spacing that this may lead to. This can be justified in two ways. First, boreholes placed at higher points of the surface over the tunnel, will lead to greater borehole lengths to reach the zone to be investigated. Second, as said before for the portal zones, lower overburden mean generally poor rock mass quality.

In the case of very thick overburden and in order not to exceed the budget allowed for site investigations one can be faced with two alternatives - either to reduce the depth of the holes (the zone to be excavated will not be investigated at least by all the boreholes), or to increase the borehole spacing. This can be a difficult option. Refering to this problem, DEERE (1969) stated that "generally rock quality tends to improve with increasing depth. It thus appears permissible to space deep borings further apart than shallow borings".

The exploration by boreholes in very thick overburden can be extraordinary expensive and it may be necessary to reduce the number of boreholes to a minimum and adopt other methods such as pilot tunnels, or probing ahead from the construction tunnel face. Boreholes to deep tunnels are not necessarily a waste of money because in most cases the information from the cores of long boreholes give a very clear and reliable picture of the entire rock mass and is a good example of the conditions which will be found by the tunnel itself.

It is generally accepted that ideally, boreholes should reach the depth of 1.0 (WEAVER, 1976) to 1.5 times the tunnel diameter, beneath the tunnel floor.

2.3.4 - BOREHOLES: GENERAL DISCUSSION

Drilling costs vary, for the same length, with the selected diameter, for the rock type and the technical execution conditions (which can be regarded as the basic costs); in addition there are

other factors influencing the final cost of a borehole programme which must be born in mind when it is planned.

Drilling diameter can vary within large limits (see table 2.XXIII), the NX is generally considered as the useful minimum for geotechnical exploration purposes. Greater diameters increase the drilling costs although they are useful in very poor quality ground. Other factors being equal, the longer the core diameter, the better is the core recovery up to about 100 mm (WILSON, 1975). Boreholes with drilling diameters smaller than NX give generally poor recoveries and the cores can easily break or desintegrate. Another advantage of NX cores is the extraordinary accumulated experience concerning the tests performed on cores of that diameter.

Among the factors associated with drilling, the rate of penetration is of utmost importance in site investigations as it greatly influences core recovery. As a general rule, the higher the velocity the poorer the recovery. About this, CUMMING (1956) states that "in general it can be said that high speed benefits the contractor but is apt to adversely affect the main interest of the owner who wants a high percentage of recovery". The owner will face the question of hoping that the contractor will do his best to achieve the maximum possible recovery. HOEK and BRAY (1974) are of the opinion that "contracts for drilling investigations of rock masses should be negociable on either an hourly rate or in terms of payment for core recovery rather than for length of hole advanced". HORSLEV (1949), in WEAVER (1976), states that "by letting the contractor on a basis of lowest cost per metre, the driller is forced to accelerate his rate of penetration with consequent loss of significant or even vital information from the cores due to erosion or grinding or vibration".

BESWICK and CRAIG (1976) pointed out that in some areas of the world, rotary core drilling is measured on core recovery and often there are incentives for recovery of over 95%. In their opinion, it can be argued that specifications with incentives are susceptible to abuse, but core drilling, as with all site investigation field work needs adequate supervision and this system does at least attempt to achieve the objective.

The order of magnitude of average drilling penetration rates for the geological formations concerned, must be known by the engineering geologist when planning the borehole programme, thus enabling him to

STANDARD CORE DRILLING SIZES

BS 4019

Designation	Core dia.	hole* dia
ZX	165.1	199.3
UX	139.7	173.9
SX	112.7	145.2
PX	92.1	120.0
HX	76.2	98.9
NX	54.0	75.4
BX	41.3	59.5
AX	30.2	47.5
EX	20.6	37.4
XRT	17.5	29.5

* Average figures within normal tolerances

CRAELIUS METRIC STANDARD

Thinwalled bits (T,B)‡		Thickwalled bits (K_3)‡	
core dia	hole dia	core dia	hole dia
-	-	116	146
-	-	101	131
-	-	86	116
84	101	72	101
72	86	58	86
62	76	48	76
52	66	38	66
42	56	34	56
32	46	24	46
22	36	-	-
16	28	-	-

‡ T - thin walled double tube barrel
B - thin walled single tube barrel
K_3 - thick walled, face discharge, double tube barrel

TABLE 2.XXIII - Standard core drilling sizes (from WILSON, 1975).

compare the needed estimate time for its execution with the time allowed for site investigations. In cases when an average penetration rate cannot be anticipated, the rate of about 1 metre per hour can be used as a basis. In Table 2.XXIV, the average penetration rates for some rock types are presented.

In the Portuguese practice the costs are, as a rule, established on a basis of cost per drilling metre for a given diameter (double tube barrel is generally used), taking into consideration the rock type and its quality. Commonly, contractors present prices a little higher for fresh to slightly weathered rock, than for highly to completely weathered rock. This difference can be estimated as about 5%. Nevertheless, the author is aware of trends in which a single price was quoted for any ground condition; clients should be discouraged

Classification	Rock Type	Penetration Rate (m/hr)
Recent deposits	Boulders and gravel	0.8 0.8
Sedimentary rocks	Shale Siltstone Sandstone Tillite Chert and banded ironstone	2.0 1.8 1.5 1.0 0.3
Metamorphic rocks	Shattered rock Siliceous quartzites Quartzite Slate	0.5 0.5 1.0 1.0
Igneous rocks	Gabbro, dolerite, basalt Granite Vitreous granite	1.2 1.0 0.5

TABLE 2.XXIV - Average penetration rates using experienced crews and conventional diamond drilling equipment, as presented by WEAVER (1976) for South African conditions.

from accepting this approach because it does not work in their favour.

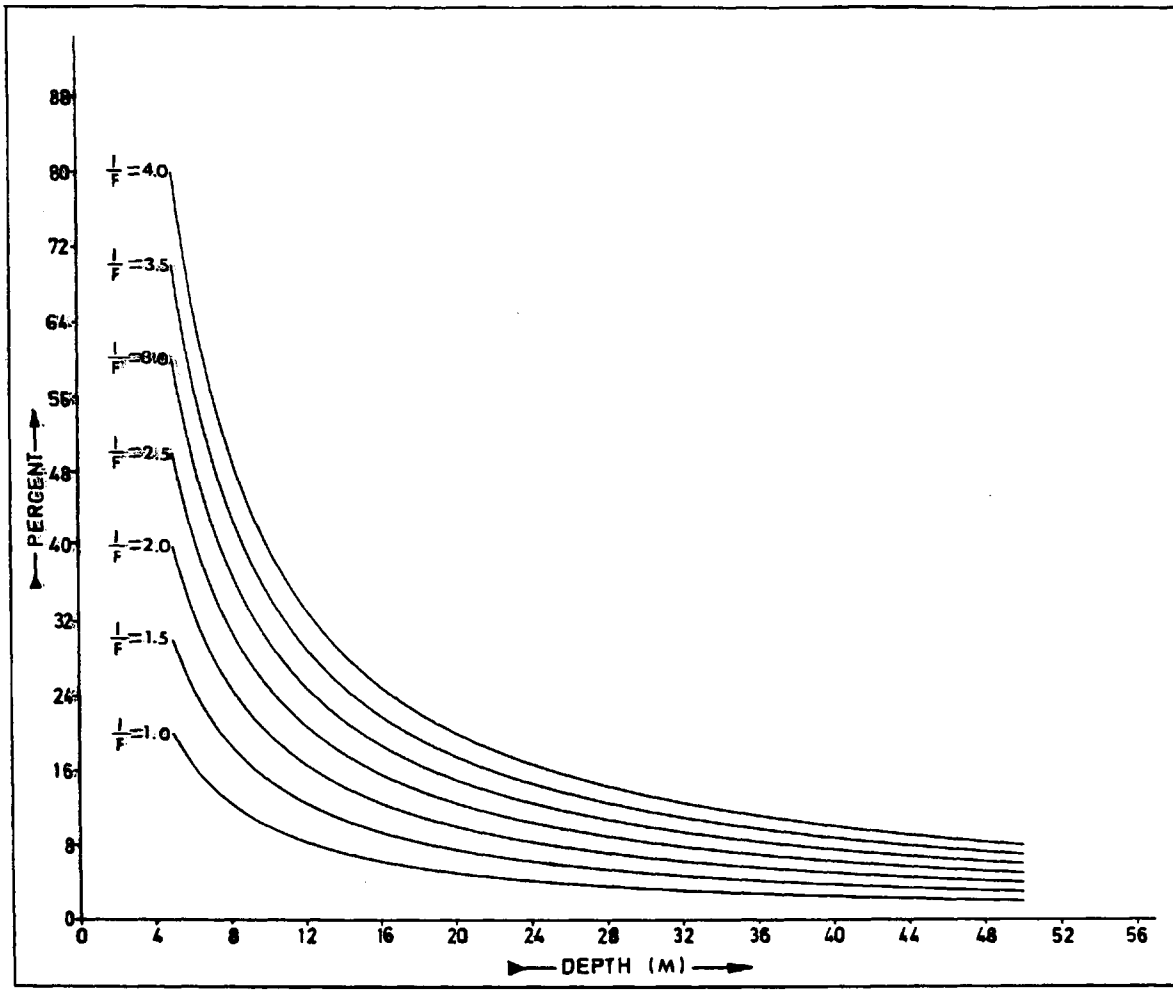
Site accessibility and topographic conditions, if difficult, will increase the programme costs due to either the greater time spent in placing equipment in the selected positions or to the need for making special access ways. If this access is used later by others, its cost should not be solely imputed either to the borehole programme or to the general site investigation costs.

When the time allowed for a given programme is restricted, the solution is to use more drilling rigs, and this can lead to some cost increase, at least that of the transport of the additional equipment and respective crews from base to the site.

For a given total borehole length at the end of a programme in a determined area, the number of boreholes and their relative positions influence, independently of any other factors, the cost of the programme. Generally, and this is the Portuguese practice, after the equipment has been taken to the site, the transport and mounting the equipment at each borehole position ("setting up"), is charged at an amount which can vary according to the distance between boreholes. For a given borehole, that cost will influence the average cost per drilled metre especially if it is shallower (see Fig. 2.12). As each change of borehole location implies dimantling, transport and mounting the equipment again, this means also an increase in the time consumed by the programme according to the number of boreholes and their relative distances.

As the borehole reaches greater depths, drilling cost increases mainly due to the time spent between the consecutive runs, to lift and to down the rods. In average terms and taking the cost of the first 50 m depth of borehole as a basis, cost per drilling metre associated with depth alone can be expected to increase by about 15-20% after 50 m and 30-40% after 100 m. Only in the programmes where the depth to be reached is established beforehand, can a sensible average price per metre of drilling be obtained from the contractor.

The tests carried out together with the drilling operations, commonly the water absorption tests, also increase the total duration of the programme and its costs. This aspect will be looked at later when dealing with those tests, and it should not be considered when fixing the drilling costs.



I - Cost of "Transport and setting up" of the equipment at each borehole position.

F - Cost per drilling metre.

M - Borehole depth (metres).

FIG. 2.12 - The curves show the cost increase (%) due to "Setting up" in relation to drilling costs, for boreholes up to 50 m ($\% = \frac{I}{F} \cdot \frac{1}{M} \cdot 100$).

The cost of "mobilization" constitutes a fixed amount for each programme that will be distributed amongst the number of boreholes performed. Thus, for programmes with a few shallow boreholes that cost represents a significant increase in relation to the final cost of the boreholes. For deep boreholes this influence is less significant becoming unimportant beyond a minimum number of them. This aspect is illustrated in Fig. 2.13. The ordinate axis represents the increase

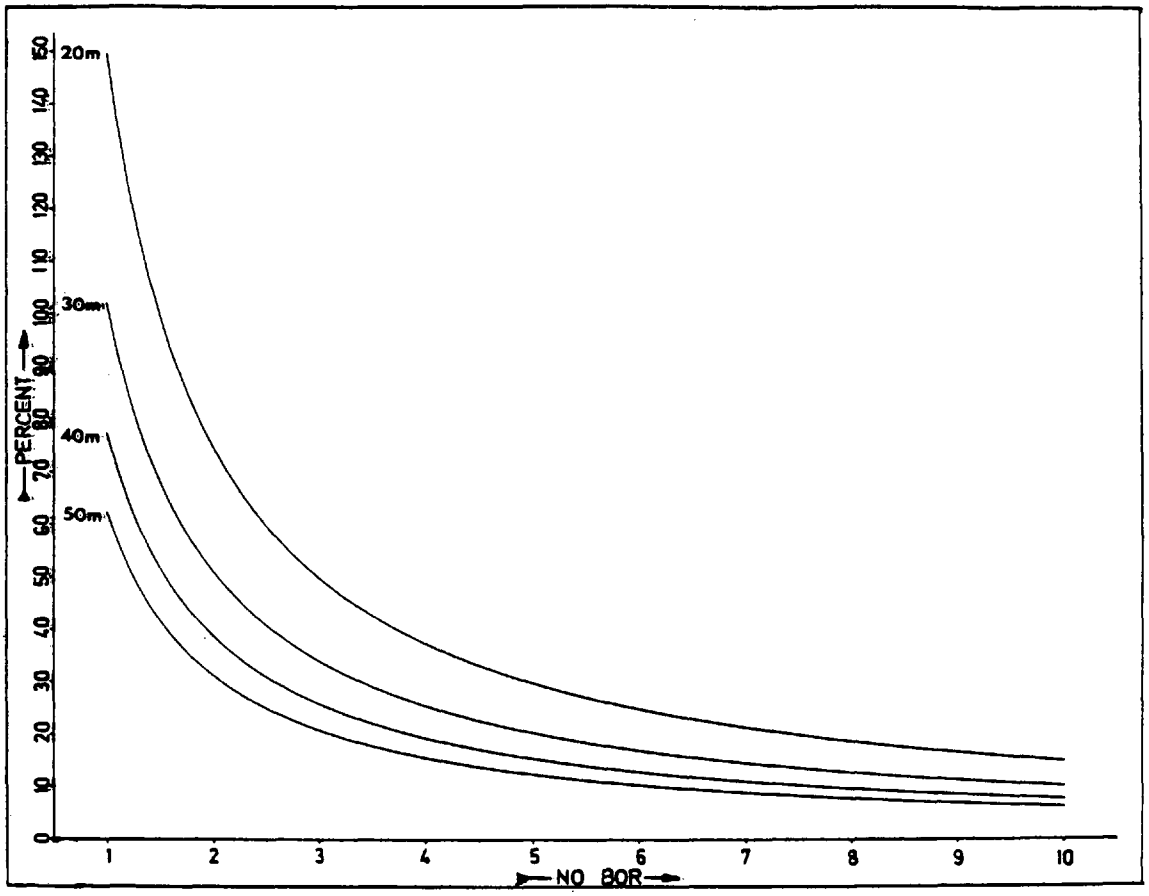


FIG. 2.13 - Cost increase (%) due to "Mobilization", in relation to the cost of "Setting up's" + drilling, for a given number of boreholes with average length between 20 m and 50 m.

due to "mobilization" costs after a given number of boreholes, as a percentage of the "setting up" plus drilling costs for the same boreholes.

In the calculation of the curves, a number of boreholes up to 10, with an average depth between 20 and 50 m were considered, and the calculations were based upon Portuguese 1980 current prices as follows, considering a site 200 km distant from the contractor's base:

"Mobilization"	90×10^3 ESCUDOS
"Setting up"	4.2×10^3 ESCUDOS
Cost per drilling metre	2.8×10^3 ESCUDOS

The price of the report was not considered as it has been

found to be unimportant for this assessment.

A long programme can, sometimes, allow the contractor to submit bids with slightly lower costs per item, but on the other hand such programmes usually lead to cost increases, mainly in countries with high inflation rates, due to the longer time involved. In this case, a formula for price adjustments should be discussed in advance.

One of the items invoiced by the contractor refers to the report concerning the performed works which includes logging, except when this is to be carried out by the consultant/owner. It must be said that even in the case where the rock core logging is done by the contractor, it always requires an inspection by the consultant/owner.

Logging costs depend upon the care and methods used to do this job. THE GEOLOGICAL SOCIETY OF LONDON (1970), quoted a cost of up to about 5% of the drilling costs to the client, and in some cases 10% or more can be reached. This estimate can constitute a starting point to negotiate from with the contractor, either for a separate price for logging, or for when the consultant/owner wants to take up the task on his own. THE GEOLOGICAL SOCIETY OF LONDON (1970, states that there could be a general improvement in both cost control and quality, if logging were treated and charged separately and not as a part of the drilling or reporting costs.

In budgeting for a given borehole programme it is always advisable to include an item for contingencies and extras. This amount which can be about 10-15% of the assessed total costs, is intended to cover costs from delays imputed to the owner and increases due to local conditions different from those anticipated, like for instance difficulties related to water supply.

CHAPTER III

TESTING METHODS

This chapter is divided into four major sections: 3.1 - mechanical characterization using common laboratory and field tests; 3.2 - index tests; 3.3 - the scale effect and 3.4 - water absorption tests.

3.1 - MECHANICAL CHARACTERIZATION TESTS

3.1.1 - LABORATORY TESTS

The type of information obtained by the most common rock mechanics laboratory tests, in terms of the parameters defining strength and deformability, is summarized in Table 3.I, together with the tests for assessing their values.

The tensile strength was deliberately omitted. The existing information about the behaviour of rock in tension is very scant and it can be said that, in general, the several techniques developed up to now (see e.g. LAMA and VUTUKURI, 1974) have not had a great success (HAWKES and MELLOR, 1970; ROCHA, 1973). Further, it is considered by many that the tensile strength of rock masses is of little relevance and is disregarded for design purposes.

Creep being a phenomenon of some practical importance in some cases like excavations for storage purposed in rock salt and other evaporites (HENDRON, 1975) is another omitted characteristic in Table 3.I. Its study is not frequent for the current situations of tunnels and dams.

Of the tests refered to in Table 3.I, the uniaxial tests to obtain E, σ_{uc} and ν are undoubtedly the most common. The triaxial test, although it can reproduce more truthfully than the uniaxial test,

the type of stresses to which the rock is submitted in the rock masses, is less frequently used for determining deformability characteristics.

MECHANICAL CHARACTERISTICS AND TESTS FOR THEIR ASSESSMENT			DERIVED PARAMETERS
DEFORMABILITY	UNCONFINED (UNIAX. TESTS)		E , ν
	CONFINED (TRIAX. TESTS)		E , ν
STRENGTH	UNIAXIAL COMPRESSIVE STRENGTH (UNIAXIAL TESTS)		σ_{ult}
	SHEAR STRENGTH	TRIAXIAL TESTS	c , ϕ_i
		DIRECT ROCK SHEAR TESTS	c , ϕ_{peak}
		JOINT SHEAR TESTS	c_{ap}, ϕ_{res}

E	- YOUNG MODULUS	(MPa)
ν	- POISSON RATIO	(Dimensionless)
σ_{ult}	- ULTIMATE STRENGTH	(MPa)
c	- COHESION	(MPa)
ϕ_i	- INTERNAL FRICTION ANGLE	(Degrees)
ϕ_{peak}	- PEAK FRICTION ANGLE	(Degrees)
c_{ap}	- APPARENT COHESION	(MPa)
ϕ_{res}	- RESIDUAL FRICTION ANGLE	(Degrees)

TABLE 3.I - Mechanical characteristics, tests and parameters, regarding the rock material.

The laboratory evaluation of parameters concerning the rock shear strength are often substituted by tests which specifically concern the shear strength along low resistance surfaces (joints, bedding, schistosity, cleavage and foliation planes). This has been accepted after it had been understood, and generally agreed, that shear in most rock masses takes place almost along these surfaces. For rock masses composed of soft rocks, the rock shear strength parameters can retain their importance because, in such cases, the rock mass shear strength as well as its deformability is less influenced by the presence of low resistance surfaces.

3.1.2 - NUMBER OF LABORATORY TESTS TO BE PERFORMED

The number of tests to be performed to obtain a reliable parameter value is, in principle, a statistical problem. For example the meaning of a strength value determined for a given type of rock depends upon the degree of heterogeneity exhibited by that rock throughout the rock mass to which it belongs.

According to the author's opinion two different approaches can be taken. When the strength variability is extensive, as evaluated by either visual inspection or other quick means, it is necessary to define the entire range of variation rather than an average value. In this case, a certain number os samples, representing the extremes of the assessed strengths, must be taken for laboratory testing. The other approach applies when the assessed variation is either small or where zones of similar expected values have been identified. In theory, this approach consists of testing enough samples taken at random from the rock mass (or zone) in order to obtain an average value and other statistical parameters as the standard deviation. When this method is chosen, the confidence interval for the determined average strength value decreases as the number of tests increases. The confidence interval for the average value, assuming a normal distribution and according to the theory of the small samples (e.g. WILSON, 1972) is, for a given confidence level:

$$X - t \frac{S}{\sqrt{n-1}} \quad \text{to} \quad X + t \frac{S}{\sqrt{n-1}}$$

Where

- $\overline{X}$ - sample mean
- t - student coefficient for n-1 degrees of freedom for the selected confidence level
- S - standard deviation
- n - sample size (number of samples)

By these expressions the influence of the number of tests on the confidence interval can be easily recognised; besides (t) decreases as (n) increases.

With regard the type of distribution, experience and investigation works have indicated that it can be considered as normal for the unconfined compressive strength and for the deformability modulus (RODRIGUES, 1974).

In practice, the number of tests performed is almost always fixed by practical reasons and some authors have proposed some order of magnitude for them.

For the uniaxial tests, the ISRM (1978) state that "at least 5 are prefered". ROBERTS, (1977) quoted the United States Bureau of Mines, whose proposed standard specifies at least 10 air-dried specimens. The LNEC-LISBON (1965) specified the utilization of 6 specimens - 3 dry and 3 in saturated conditions.

In the case of determination of rock shear strength either by direct shear tests or by using triaxial tests only one point of the MOHR's envelope, required to obtain the shear parameter values, is achieved by each test. The number of points and therefore the number of tests needed to draw the MOHR's envelope cannot, by any means, be less than 3. The most common practice is to test 4 or 5 specimens and the ISRM (1974, 1977a) suggested, at least, 5 tests.

3.1.3 - "IN-SITU" TESTS

As it is impossible to test representative samples of the rock masses by means of laboratory tests, the evaluation of their mechanical characteristics is carried out by performing "in-situ" tests, the most common being those presented in Table 3.II.

Triaxial tests "in-situ" similar to those carried out at Kurobe dam site in Japan (JOHN, 1961) are not practical as they are difficult to perform, very time-consuming and very expensive. According to ROCHA (1973) only very special conditions like those encountered at that dam site can justify that type of test. The "in-situ" uniaxial tests for stength assessment can be important for designing mine pillars although they are not utilized for civil engineering purposes. For this reason the above mentioned tests were not included in Table 3.II.

Regarding the "in-situ" shear tests they are used less as a direct shear test than for obtaining the shear parameter values along low resistance surfaces, this being due to the severe influence of

MECHANICAL CHARACTERISTICS AND TESTS FOR THEIR ASSESSMENT		DERIVED PARAMETERS
DEFORMABILITY	CONVENTIONAL JACK TESTS (PLATE LOAD TESTS)	E
	FLAT JACK TESTS	
	IN BOREHOLE TESTS (DILATOMETER TESTS)	E_{dil}
STRENGTH	DIRECT SHEAR STRENGTH	c, ϕ_{peak}
	JOINT SHEAR STRENGTH	c_{ap}, ϕ_{res}
"IN-SITU" STATE OF STRESS	STRAIN TENSOR GAUGE (IN BOREHOLE METHOD)	Magnitude and direction of the 3 principal stresses σx, σy, σz, in relation to a coordinate axis system Oxyz
	THIN FLAT JACK METHOD	

E	-	IN SITU DEFORMABILITY MODULUS	(MPa)
E_{dil}	-	DILATOMETER MODULUS	(MPa)
c	-	COHESION	(MPa)
ϕ_{peak}	-	PEAK FRICTION ANGLE	(Degrees)
c_{ap}	-	APPARENT COHESION	(MPa)
ϕ_{res}	-	RESIDUAL FRICTION ANGLE	(Degress)
σx, σy, σz	-	PRINCIPAL "IN-SITU" STRESSES	(MPa)

TABLE 3.II - Mechanical characteristics, tests and parameters regarding the rock masses.

such surfaces on the shear strength of the rock masses. From a certain block size onwards, the stresses which are required for achieving the shear failure are very high, limiting the direct test to use in rock masses composed of either soft or considerably fissured rocks. Nevertheless, it is exactly in these situations that it is more important to obtain the direct shear parameter values.

The most common shear areas involved with the "in-situ" shear tests are similar to those used in Portuguese practice (0.7 x 0.7 m^2), although greater areas can be utilized, as quoted by some authors. NIEBLE et al. (1974) for example, reported tests where an area of 6 x 6 m^2 was used. EVDOKIMOV and SAPEGIN (1970), reported shear tests carried out for a dam site in Russia dealing with a shear area of 8 x 12 m^2 which, in the author's knowledge, seems to be the largest ones performed in the world.

3.1.4 - NUMBER OF "IN-SITU" TESTS TO BE PERFORMED

Bearing in mind that "in-situ" tests for mechanical characterization are time-consuming and expensive it is generally a small number that is possible to perform, so not reaching, by many times, the number that would be statistically desirable. It is not possible, under these circumstances, to think in statistical terms and the values to be used in the design have much to do with the quality of the engineer's judgement which has to be based upon the small number of results, on his own experience and on the data from the other methods used in site investigations.

The chief problem which arises deals mainly with the variability throughout the rock mass of the mechanical behaviour to be assessed; i.e. with the selection of the test locations. To obtain favourable benefit/cost relationships in terms of reliable information it is necessary to directly relate the siting of tests carefully to the geotechnical zoning of the rock mass, which has taken regard of the features that can influence the "in-situ" mechanical behaviour; each zone can then be characterized by means of "in-situ" tests.

To determine rock mass deformability, that is the mechanical parameter to which more attention is generally given for dam site studies, the methodology suggested by the "LISBON SCHOOL" (e.g. ROCHA and DA SILVA, 1970; ROCHA, 1970; ROCHA, 1974; CHARRUA-GRAÇA 1979; and OLIVEIRA, 1980) is based on considering a three dimension zoning

of the rock mass based on available geological information and by using index tests which can be sensitive to deformability. After that, "in-situ" tests are carried out in each defined zone, the large flat jacks - LFJ - (ROCHA and DA SILVA, 1970) being the most commonly utilized method in Portugal. The results of these tests are correlated to the ones from the index tests.

The dilatometer test (ROCHA et al., 1970) which costs about one fifth of the LFJ test, has proved to be a good index for assessing the deformability, but other indices can also be used; for example, the ones derived from the employment of seismic methods (see Chapter 2.1).

The number of "in-situ" tests to be performed, as quoted below, refers to each previously established zone. Likewise, that concerning the shear tests along low resistance sufaces such as joints, applies for each well defined set.

In the author's opinion, for assessing the deformability modulus, not less than 4 tests should be performed, either when a certain rock mass isotropy has been verified or for each direction tested when evaluating the influences of the anisotropy.

The ISRM (1974), suggested that "a shear strength determination should preferably comprise at least five tests on the same test horizon with each specimen tested at different but constant normal stresses".

Shear strength tests regarding low resistance surfaces should be carried out also on five specimens, and the suggested conditions dealing with the normal stress should also be followed.

The determination of the "in-situ" state of stress either by using the strain tensor gauge - STT - or the small flat jack method - SFJ - (ROCHA, 1969; ROCHA et al., 1974) should comprise, at least 4 tests according to the Portuguese experience.

3.1.5 - MECHANICAL CHARACTERIZATION: GENERAL DISCUSSION

From the review of the laboratory and "in-situ" tests required to characterize rock materials and rock masses it can be appreciated that the cost and the benefit of such characterization is sensitive to the type, size and number of tests.

Data related to time, cost and personnel involved in laboratory and "in-situ" mechanical characterization tests are presented in Table 3.III and 3.IV respectively.

TEST	AVERAGE COSTS x 10^3 PORT. ESCUDOS	PERSONNEL INVOLVED*	EXECUTION TIME (min.)	TIME FOR SAMPLE PREPARA-TION (min)	REMARKS
UNIAXIAL TESTS (E, σ_{uc}, ν)	1.7	1C + 1D	20	60	Cost includes sample pre-paration from rock cores
TRIAXIAL TESTS (1 point of Mohr's envel.	4	1C + 1D	60	60	Cost includes sample preparation
DIRECT SHEAR TESTS (1 point of Mohr's envel.	1.5	1C + 1D	15	Variable	Cost includes sample pre-paration
JOINT SHEAR TESTS (1 point of Mohr's envel.)	1.5	1C + 1D	15	Variable	Cost includes sample pre-paration

* For explanation of symbols see page 51

TABLE 3.III - Time, cost and personnel involved in laboratory mechanical characterization tests.

Time and costs are indicated for unit testing, as average values. Time is quoted according to LNEC - LISBON experience and costs refer to those applied in 1980 by that institution.

The difficulty involved in performing the "in-situ" tests due to particular local conditions are not taken into account here

TESTS	TIME TO MOUNT AND DISMOUNT EQUIPMENT (1)	TIME FOR EXECUTION OF THE TEST (2)	TOTAL TIME (1)+(2)	PERSONNEL INVOLVED*	TIME FOR TEST PREPARATION	CALCULATIONS AND REPORT PREPARATION	AVERAGE COSTS(**) x 10^3 PORT. ESC.	REMARKS
PLATE LOAD TESTS	4 days	1 day	5 days	1A/B 1D/C 2E 2E	Variable 2-3 weeks	2 days	140	Cost does not include the necessary preparations (chamber, surfaces etc.)
LARGE FLAT JACK TESTS	4 to 5.5 days (A)	1 day	5 to 6.5 days (A)	1A/B 1D/C 3F	Variable (B)	1 day	90 (C)	(A) Includes drilling, slot opening, grouting and redrilling if necessary. (B) Refers to the preparation of the locality for the text. (C) Does not inlcude any necessary preparation such as enlargement of chambers, surface preparation to place the equipment etc.
DILATOMETER TESTS BDH - LNEC	0.5 to 1 day		0.5 to 1 day	1A/B 1D/C 2E	Nill to variable (D)	1 day	20 (E)	(D) Time increase if ground has to be treated (grouted) and test hole redrilled. (E) Cost applies for measurements in 4 directions at the same depth, and does not include any necessary preparation (borehole, grouting and redrilling if necessary, etc.)

ROCK SHEAR TESTS	4 days	1 day	5 days	1A/B 1D/C 1E 3F	≈2 weeks for chamber preparation plus 12 days for cutting 1 block	1.5 days	140 (F)	(F) Cost does not include opening of chambers when the tests are performed inside them, but includes the block cutting and the preparation of the reaction surface.
JOINT SHEAR TESTS	4 days	2 days	6 days	1A/B 1D/C 1E 3F	≈2 weeks for chamber preparation plus 12 days for cutting 1 block	1.5 day	150 (G)	(G) Cpst does not include the preparation of chambers, but includes the block cutting and the preparation of the reaction surface
"IN-SITU" STATE OF STRESS (SMALL FLAT JACK TESTS - SFJ)	3 days (H)		3 days (H)	1A/B 1D/C 1E 1F	Variable (I)	1 day (for 4 tests) (K)	15 (J)	(H) Includes drilling, slot opening, cementation and reperfuration if necessary. (I) Refers to the preparation of the local for the test. (J) Cost does not include any necessary preparation as enlargement of chambers, surface regularization for placing the equipment etc. (K) At least, 4 tests are advisable.

"IN--SITU" STATE OF STRESS (Strain tensor gauge--STT)	5 days (L)	5 days (L)	1A/B 1D/C plus drilling crew	Variable (M)	1 day	40	(L) Includes all operations except drilling (M) Depends almost entirely upon the drilling time needed to reach the depth for the test

* For symbols explanation see page 51

** Cost does not include travel expenses for both personnel and equipment.

TABLE 3.IV - Time, cost and personnel involved in "in-situ" mechanical characterization tests

although it is obvious that these aspects play an important role not only in the costs of the tests but also in the time involved in their execution.

The data presented in Table 3.IV assume average difficulty conditions as regards the time quoted, in accordance with the experience from a large number of tests carried out by LNEC - LISBON (see CUNHA, 1975) and the costs quoted do not include the travel expenses for both personnel and equipment.

It is sometimes very difficult to make the owners understand the benefits of the "in-situ" tests for mechanical characterization. Those who oppose these tests usually argue that they are extraordinary time-consuming and that their cost is prohibitive. It is very important to stress that, as a rule, the benefits gained by performigg these tests are always in the owner's favour. The results may avoid an over-conservative design which means a saving of money; they may lead to a more safe design as the mechanical characteristics determined by the tests can be revealed less favourable than the ones which would be predicted without them.

It is worthwhile to quote a BIENIAWSKI and VAN HEERDEN's (1975) example, although it refers to mining aspects. These authors reported the benefit gained by a coal mine performing large scale tests. These tests enabled them to conclude that the pillars which were being utilized were about 50% stronger than previously predicted and so the mine could increase percentage of extraction by 9.3% using the same safety factor.

3.2 - THE INDEX TESTS

The index tests referred to here, also designated as "quick" tests, are simple tests giving results that can be correlated, in some way, with the values of the mechanical parameters obtained by using more conventional testing methods as described in 3.1. The difference between index tests and the common tests for mechanical characterization is that mechanical characterization tests provide parameters for design whereas index tests provide a simple means for identifying and grouping similar materials and indicating their likely range of properties.

The main conditions which any test should fulfill in order to be considered as an index test are those mentioned in table 3.V.

* To be quicker and less expensive than the traditional tests
* To be of easy execution in the field or in the laboratory
* To be reproducible
* To yield values which can be related to the values of the parameters obtained by using the traditional tests, through simple formula

TABLE 3.V - Conditions to be obeyed by the index tests .

The number of conventional tests performed is often low, frequently due to their cost. In other cases it is necessary to make decisions within a time limit that does not permit the conventional tests to be performed, especially "in-situ" tests. In such cases, the index tests can be of help.

In other situations the degree of accuracy required of the results is small and the execution of traditional tests (at least in a great number) does not bring any additional benefit when compared with the results by means of the index tests; here only the cost of testing is increased, so bringing a less favourable benefit/cost relationship to the investigation.

The most common index tests for rocks will be reviewed, these being the point load test, the Schmidt hammer test and the ultrasonic wave velocity test. A brief reference will also finally be made to some others.

3.2.1 - THE POINT LOAD TEST

Originating from the works of PROTODIAKONOV (1960, 1961) this index test has been used in practice, with success, since the results presented by d'ANDREA (1965) and particularly since those of FRANKLIN (1970, 1970a), FRANKLIN et al., (1971) and BROCH and FRANKLIN (1972); in the last paper, the test is exhaustively described and discussed.

One of the advantages of the test is that either irregularly -shaped samples or cored rock samples can be tested.

The point load test utilises the tensile strength of the tested material, but very close correlations (correlation coefficients of 0.9 or even greater) between the results obtained and the results of the unconfined compressive strength tests have been achieved, the more usual being that given by the relationship:

$$\sigma_{uc} = 24\ I_{s(50)}$$

where:

σ_{uc} - unconfined compressive strength (MPa)

$I_{s(50)}$ - strength index value (MPa) obtained from diametral tests on rock core pieces with diameter of 50 mm (this is, approximately the diameter of the NX cores - 54 mm).

The Geological Society of London (1977) suggested a size correlation chart for point load strength to unconfined compressive strength: the conversion presented in Fig. 3.1.

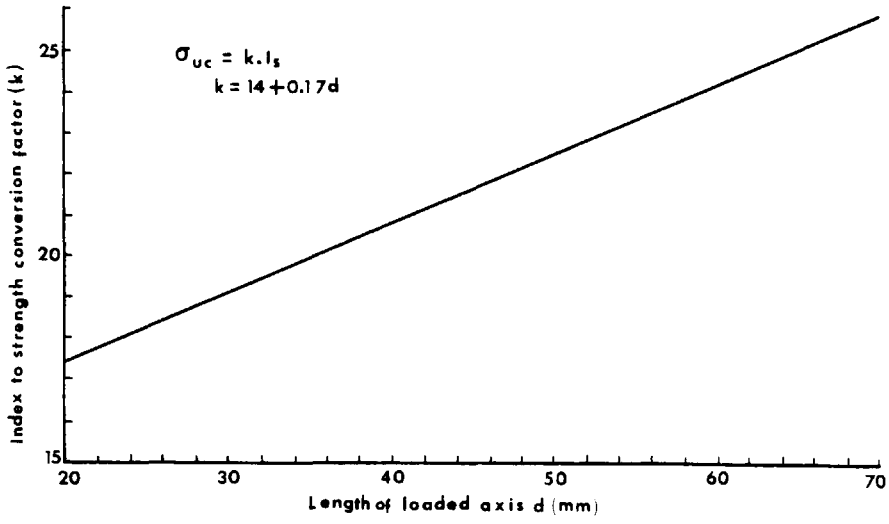


Fig. 3.1 - Size correlation chart for point load strength to unconfined compressive strength conversion (Geological Society of London, 1977).

As an example, the correlation obtained for the rock material of the Castelo do Bode tunnel, one of the studied projects (see Appendix A, vol. II), is presented in Fig. 3.2. In this case the correlation coefficient obtained was r = 0.87 (LNEC, 1977).

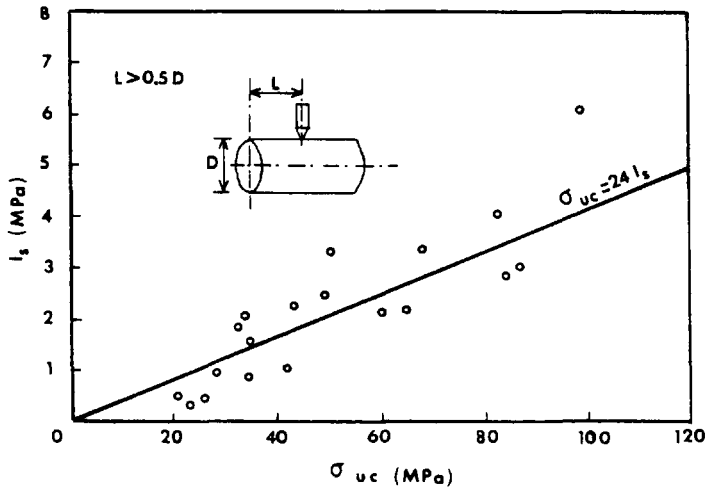


Fig. 3.2 - Correlation between σ_{uc} and I_s obtained for the rock material at Castelo do Bode tunnel.

According to BIENIAWSKI (1975), the only limit of applicability for the point load test would be in the domain of the soft rocks ($\sigma_{uc} < 25$ MPa or $I_s < 1$ MPa). Nevertheless, the author verified that even for soft rocks, namely chalk, the point load test still gives very acceptable correlations with σ_{uc} (CARVALHO, 1980).

Even when applied alone (although not recommended) this test gives a generally reasonable assessment of the unconfined compressive strength, using the index to strength conversion of 24.

BROCH and FRANKLIN (1972), after a comparative assessment, stated that the point load test results show less scatter than the ones obtained by the uniaxial tests; this was contested by BIENIAWSKI, (1975). A large number of results available from either uniaxial compressive tests or point load tests carried out on the chalk for the Chinnor tunnel, suggest that Bieniawski's opinion is justified (CARVALHO, 1980).

Other authors have found correlations between the I_s values

and other parameters, for example, with the percussive drilling rate (BERGH - CHRISTENSEN and SELMER OLSEN, 1970), however, the merit of the test resides, without any doubt, in the fact that it can be considered the best index test to assess the unconfined compressive strength.

FRANKLIN (1970) in relating the value of I_s and the fracture spacing (or layer thickness), suggested a rock quality classification, which is presented in Fig. 3.3 with some modifications to take account of the Basic Geotechnical Description of Rock Masses (BGD) suggested by ISRM (1980).

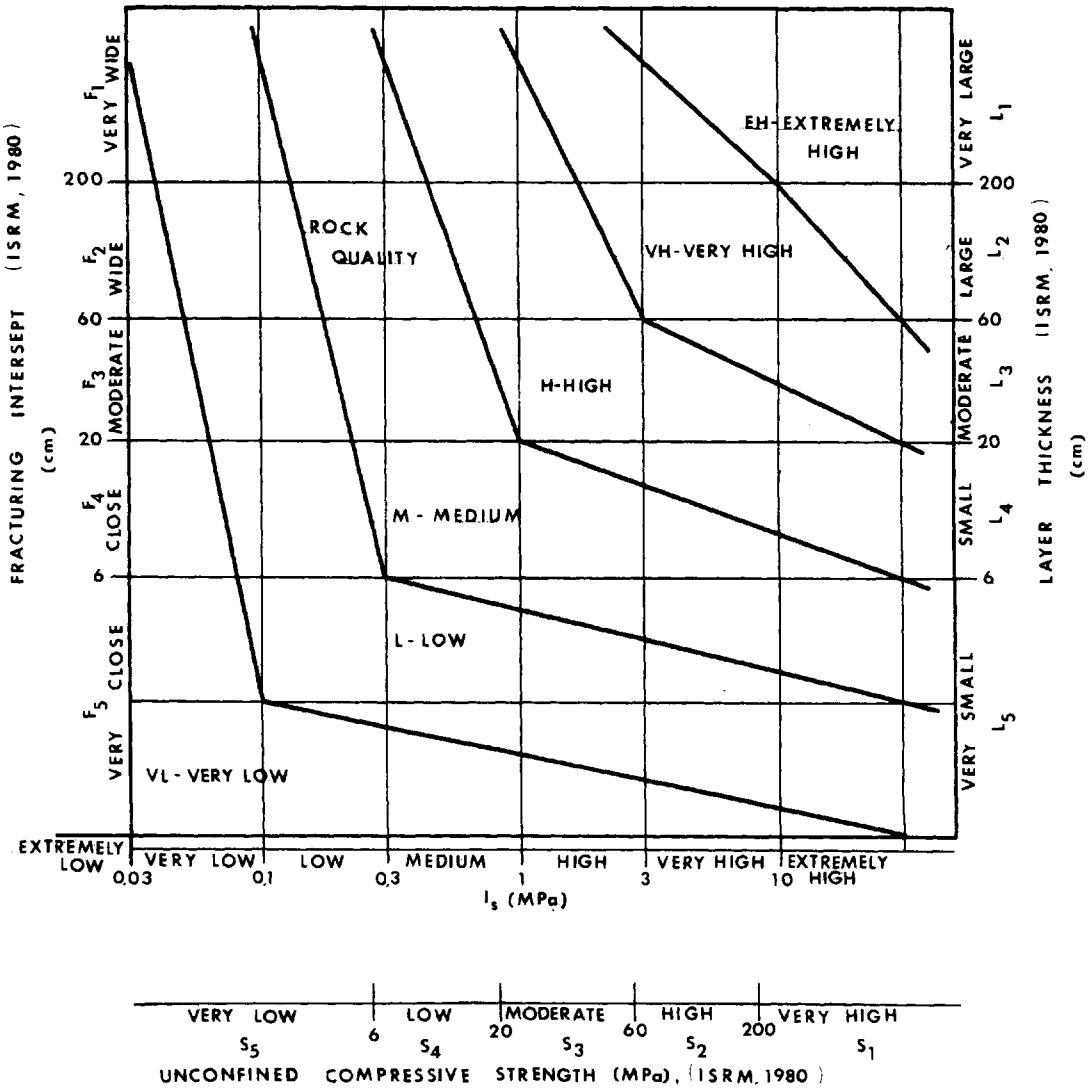


Fig. 3.3 - Rock quality classification based on the point load strength index and on fracture spacing (or layer thickness). Modified from FRANKLIN et al. (1971). The index to strength factor used was 24.

The same author also presented an alternative diagram to evaluate the excavatability of rock masses, this being presented, also modified, in Fig. 3.4.

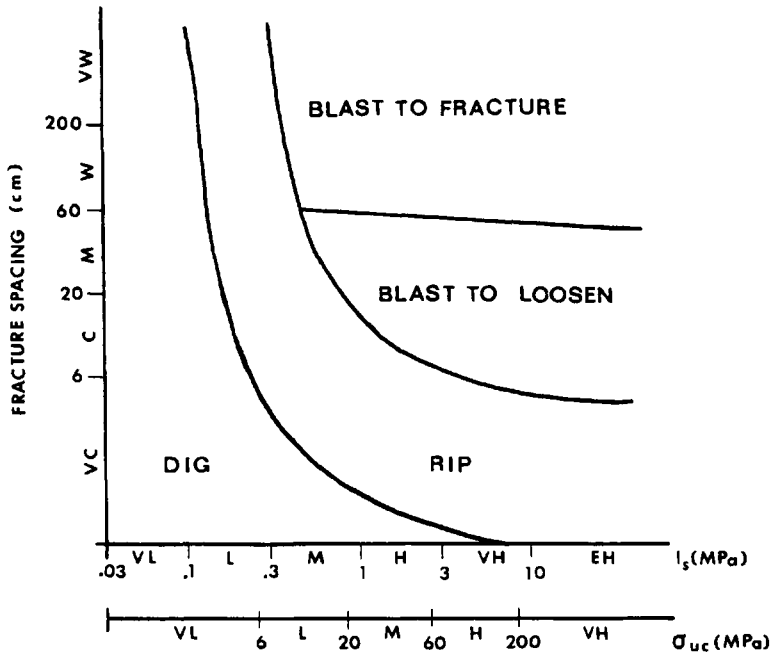


Fig. 3.4 - Assessment of rock mass excavatability. Modified from FANKLIN (1971). Unconfined compressive strength and fracture spacing according to ISRM (1980)

Although the zones of this diagram should be established on the basis of experience, rather than in a rigorous manner its use can be greatly increased if the fracture spacing of a rock mass can be also assessed using quick techniques such as stereoscopic photographs (CARVALHO, 1976).

The point load test is one of the tests recommended by ISRM (1972) who suggested a methodology for its execution.

3.2.2 - THE SCHMIDT HAMMER TEST

The Schmidt hammer (e.g. SCHMIDT 1951; HUCKA, 1965; DEER, 1965; DEER and MILLER, 1966), having been developed initially to evaluate the strength of concrete, was extended to rocks. Basically the hardness of the rock is determined and the result presented as a "rebound number" which can be correlated with the unconfined compressive strength when this is greater than a certain value. This index test, is also recommended by the ISRM (1977) for the assessment of the "in-situ"

strength of rock, and is specially recommended in the determination of the strength of joint walls.

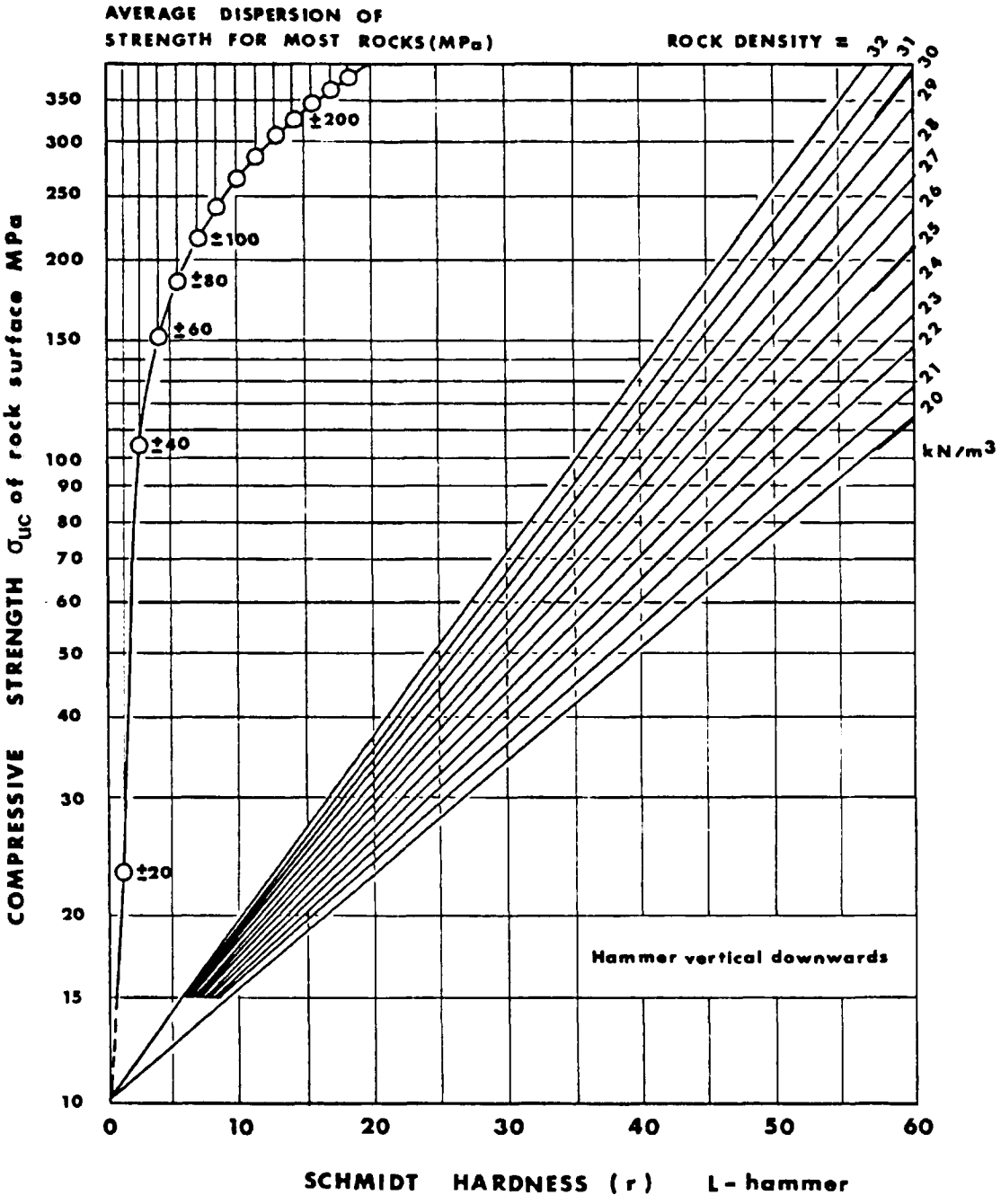


FIG. 3.5 - Correlation chart for Schmidt (L) hammer, relating rock density, compressive strength and rebound number, after MILLER (1975), as presented by the ISRM (1977). Dispersion limits defined for a 75% confidence level.

The Schmidt hammer test is less reliable than the point load test. As stated by the Geological Society of London (1977) there is only 75% probability that the laboratory determined uniaxial compressive strength will lie within 50% of the strength determined from the correlation chart prepared by MILLER (1965), presented in Fig. 3.5.

Both Schmidt hammer and point load tests are also recommended by the Geological Society of London (1977) "as having been proved to provide quantitative or semi-quantitative data rapidly and at low cost". The Geological Society also stated that, in the case of the Schmidt hammer, the method is only applicable to rocks in the strong to very strong range, that is, when $\sigma_{uc} > 50$ MPa.

3.2.3 - THE ULTRASONIC WAVE VELOCITIES

Another frequently used strength index test is the laboratory measurement of the ultrasonic longitudinal wave velocities. The results of this test can also be compared with the longitudinal wave velocities measured "in-situ", to provide a geological quality index for rock masses. This aspect was discussed in Chapter 2.1.

The utilization of high frequency pulses fulfills the condition that the corresponding wavelengths are several times smaller than the length of the samples commonly used. These samples can be either small rock blocks with two parallel faces or, more usually, rock core pieces.

The velocity of the longitudinal ultrasonic waves V_p is itself a good guide to the state of weathering of the rock material. This fact was refered by RODRIGUES (1975).

The ultrasonic shear wave velocity V_s, can also be taken as an index, using adequate equipment and when used in conjunction with the V_p values enables the dynamic Poisson's ratio to be assessed.

Regarding the relationship between V_p and the mechanical characteristics of rocks, good correlations have been achieved with the strength and with the deformability modulus obtained from uniaxial compressive tests. Fig. 3.6 presents a correlation between V_p and σ_{uc} for the rock material from the Alto Lindoso tunnel (see Appendix A, Vol. II). Notice the close correlation achieved for $V_p < 3500$ m/sec. or $\sigma_{uc} < 80$ MPa.

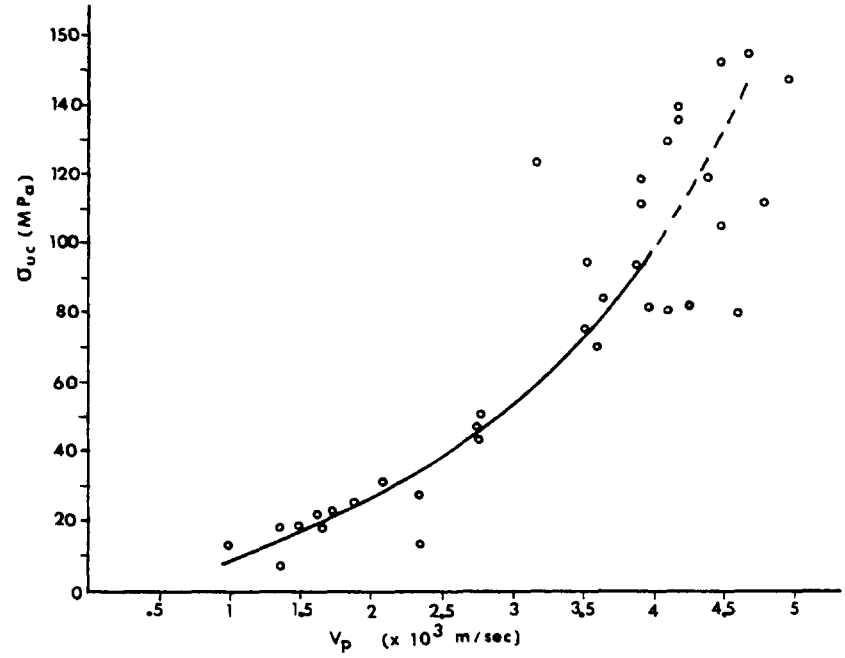


Fig. 3.6 - Correlation between ultrasonic longitudinal wave velocity and σ_{uc} for the rock material from the Alto Lindoso tunnel (LNEC, 1977a)

A correlation between V_p and the deformability modulus (E) is presented in Fig. 3.7; the rock tested was from Castelo do Bode tunnel (see Appendix A, Vol. II).

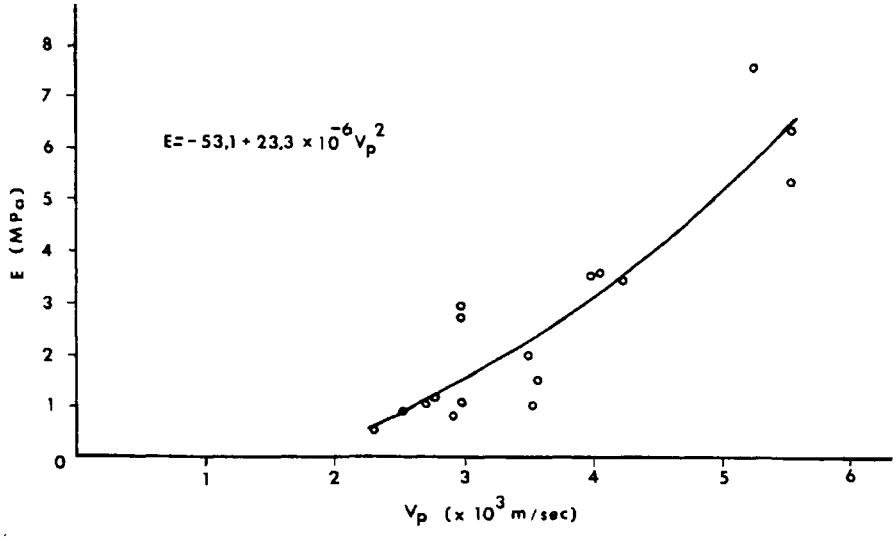


Fig. 3.7 - Correlation between V_p and E for the rock from Castelo do Bode tunnel (LNEC, 1977)

It is also interesting to note the reasonable correlation between the ultrasonic longitudinal wave velocity V_p and the point load strength values I_s for the same rock, as presented in Fig. 3.8.

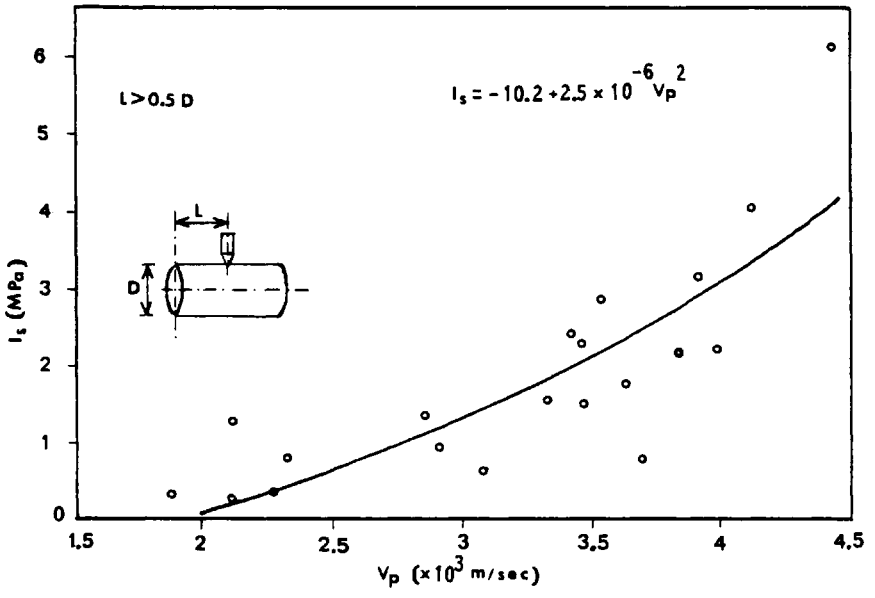


Fig. 3.8 - Correlation between V_p and I_s for the Castelo do Bode rock material.

Although V_p is an index that is generally not as reliable as the point load strength (I_s) its utilization can be of considerable importance in some cases, mainly because it is a non destructive test.

3.2.4 - OTHER INDEX TESTS

The use of other tests as index tests has been studied and sometimes practiced, although generally in a more restricted way.

SMORODINOV et al. (1970) for example, found good correlations between porosity and unconfined compressive strength for some particular rock types - see Fig. 3.9 and Table 3.VI.

Similarly, FRIEDMAN (1976), quoting KOWALSKI (1966), HOSHINO et al. (1972), DUNN et al. (1973) and HOSHINO (1974) stated that unconfined compressive strength of sandstones, siltstones and a few limestones decreases with increasing porosity, this trend following the general form:

$$\sigma_{uc} = a\, n^b$$

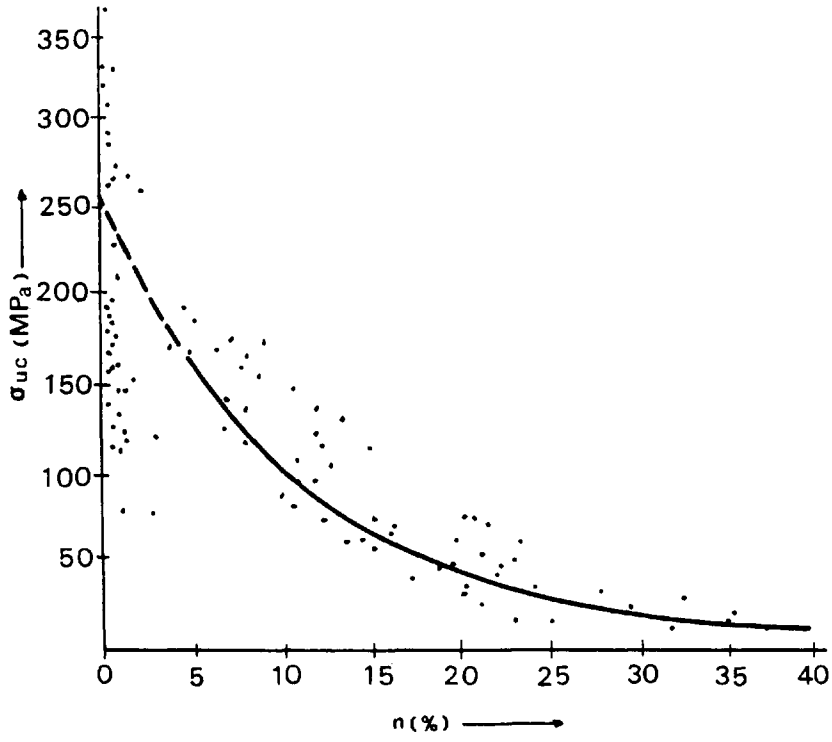


Fig. 3.9 - Relationship between porosity and unconfined compressive strength for some carbonate rocks (SMORODINOV, et al., 1970; modified).

ROCK TYPE	CORRELATION σ_{uc} (MPa) =	CORRELATION RATIO
MARBLES LIMESTONES AND DOLOMITES	$259\ e^{-0.091n}$	0.81 ± 0.019
LIMESTONES AND DOLOMITES	$322\ e^{-0.093n}$	0.93 ± 0.014
QUARTZITES AND QUARTZITE-SAND-STONE	$350\ e^{-0.108n}$	0.85 ± 0.046

TABLE 3.VI - Correlations between porosity (n) and σ_{uc} for some groups of samples tested by SMORODINOV (1970)

where σ_{uc} is the unconfined compressive strength, n is the porosity, a and b are constants which depend upon the lithology, degree of weathering, grain size etc.

The measurement of porosity is never an easy task and prevents this index from being economic, hence widely used. In certain circumstances porosity related indices can be of very great use as was proved at Alto Rabagão dam site where the rock water absorption test was used as defined by the relationship:

$$i(\%) = \frac{\text{weight of absorbed water by the rock dried at } 105^{\circ}C}{\text{weight of the dried rock}}$$

In the granite foundations of the Alto Rabagão dam (ROCHA, 1966), it was used to assist in deciding the depth of excavation in the more deformable zones of the foundations, to reach the values of deformability modulus adopted in the design. Fig. 3.10 shows the achieved correlation between i(%) and the deformability modulus of the rock mass obtained by performing 63 plate load tests at the surface.

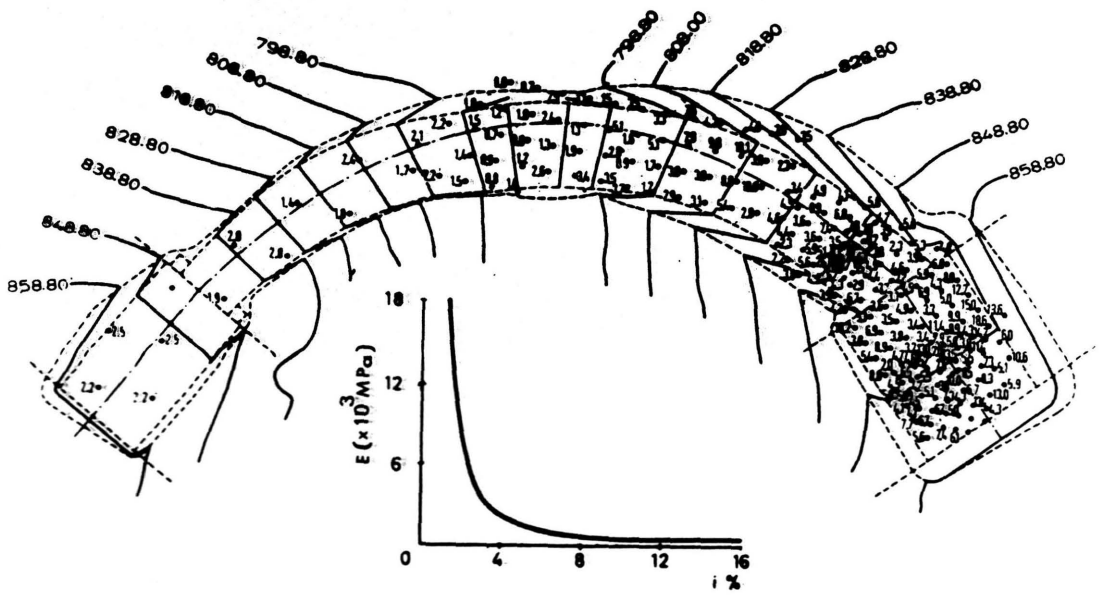


Fig. 3.10 - Correlation between i (%) and E_{field} at Alto Rabagão dam site (ROCHA, 1966)

In Fig. 3.11 the correlations between i (%) and the shear strength parameters also obtained at Alto Rabagao dam site by means of 44 rock shear tests are presented.

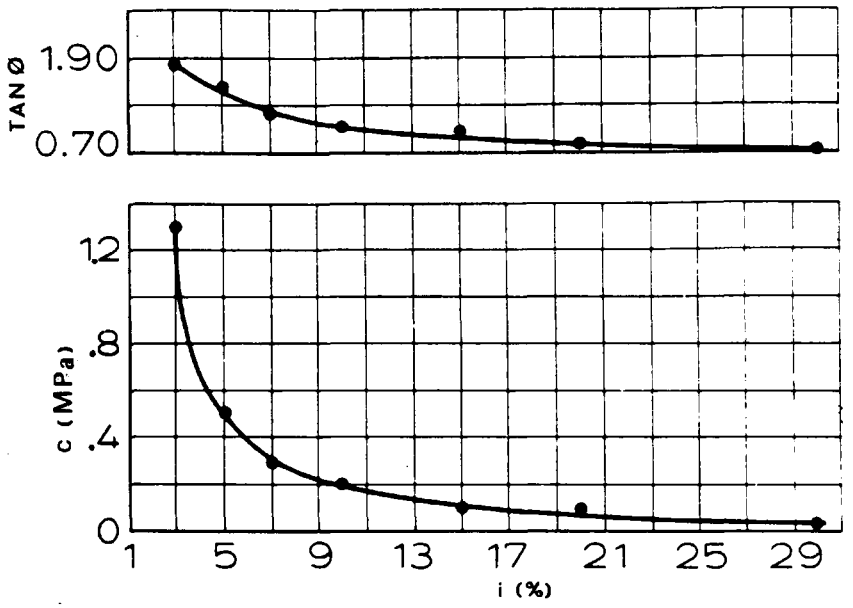


FIG. 3.11 - Correlation between i (%) and E_{field} at Alto Rabagão dam site (ROCHA, 1966).

3.2.5 - INDEX TESTS: GENERAL DISCUSSION

The main virtues of index testing are their ease of execution, the short time required and their low cost (including personnel costs) and this results in benefit/cost relationships that are very favourable for indicating the mechanical characteristics of the ground from which the samples were taken.

In Table 3.VII a comparison is made between the chief index tests which give measure of the unconfined compressive strength and the traditional uniaxial compressive strength test. Costs include sample preparation and are in accordance with the prices for single tests applied by LNEC-LISBON in 1980. Time refers only to that involved in testing. All the tests are assumed to be performed in a laboratory on previously selected samples, and the number of tests suggested is considered the minimum that should be carried out: typical values are shown.

The performance of both the point load test and the Schmidt hammer test in the field implies the careful selection of samples that can be considered representative. This should result in the employment of more trained personnel than that quoted in Table 3.VII, e.g. at least one junior graduate (1B).

The characteristics of the point load test propitiate its utilization as a routine test for rock core logging as it is

TEST	AVER. UNIT COST (PORT. ESCUDOS)	PERSONNEL INVOLVED*	TEST TIME	NO. OF TESTS SUGGESTED	REMARKS
UNIAXIAL COMPRESSIVE STRENGTH	1 700	1C + 1D	20	5 (ISRM, 1978)	Cost includes sample preparation from rock core. Time refers only to test performance.
POINT LOAD	250	1D	5	10 (BROCH and FRANKLIN, 1972)	Data refers to diametral tests performed on rock core pieces.
SCHMIDT HAMMER	200	1D	1	10 (THE GEOL. SOC., 1977) (ISRM, 1978)	
ULTRASONIC WAVE VELOCITY	380	1B, or 1C or 1D	10	5	Measurement of V_p and V_s.

* For symbols explanation see page 51

TABLE 3.VII - Comparison of cost, personnel involved and execution time, between the uniaxial compressive strength test and some index tests.

recommended for example, by BIENIAWSKI (1975). This practice must be encouraged. Because of its greater reliability the point load test should be used in preference to either the Schmidt hammer test or the ultrasonic wave velocity test, for assessing strength.

It is not suggested that the index tests are a substitute for the traditional tests, but that it is desirable to extend their utilization. As stated by FRANKLIN and CHANDRA (1972) "the index tests represent a compromise between simplicity (and low cost) on the one hand the precision on the other". As a rule, the methodology to be followed should be to take advantage of these tests during the early phases of site investigations, together with a few traditional tests in order to obtain an approximate correlation. It is during the geotechnical zoning of the rock masses that the greatest benefits can be obtained by performing index tests to define zones which will be tested later by the conventional methods. This can lead to a reliable assessment of the correlations that exist, which may allow the later use of these tests alone, even during construction.

3.3 - THE SCALE EFFECT

3.3.1 - GENERAL

One of the most difficult problems which has to be faced in the design of civil engineering works, especially in the case of concrete dams and tunnels, is the application to the rock masses of values for both resistance and deformability as indicated by the results of the tests performed.

It is broadly accepted that the dimensions of the test specimen influence the results: an influence that is generally marked by an increase in deformability and by a decrease in strength, as the dimensions of the loaded volume are increased. This phenomenon is usually referred to as the scale effect.

It is a well known fact that samples which are tested in the laboratory are not representative of the rock masses from which they came because of their small dimensions. They do not represent the rock masses with regard to their macro-structure, heterogeneity and anisotropy.

The size of laboratory samples is limited in practical terms by the ease with which they can be obtained and handled, and by the capacity of the available loading equipment. The latter can become an

important constraint, for instance, even in the case of the most popular test - the uniaxial compressive strength test. PRICE and KNILL (1974) found that "the load required to fail a ZX (165 mm diam.) specimen of strong granite would be of the order of 4,500 KN (450 tf) while most load testing machines used for rock testing have a maximum load capacity of about 1,000 KN (100 tf). So that to achieve failure the sample has to be cored to a smaller size".

For subsurface sampling, except the particular cases when shafts or galleries are excavated, the samples available are those obtained by means of boreholes that, for economical and practical reasons, do not normally exceed the NX diameter ($\simeq$ 76 mm) with a corresponding sample diameter of 54 mm.

3.3.2 - THE SCALE EFFECT ON THE STRENGTH OF THE ROCK MATERIAL

A - Influence of specimen size

BRAYBROOKE (1966) in reviewing the dependence of strength on specimen size, stated that little reliable quantitative data was available. Five to six years later, BROWN (1971) pointed out that "attempts to determine whether or not a strength-size effect exists for rock material have produced a body of conflicting and generally inconclusive data". Later, this author (HOEK and BROWN, 1980) proposed a relationship between uniaxial compressive strength and specimen diameter for specimens between 10 and 200 mm diameter given by

$$\sigma_{uc} = {}_{uc\ 50}\ (50/d)^{0.18}$$

where: $\sigma_{uc\ 50}$ is the uniaxial compressive strength of a specimen of 50 mm diameter and

d is the diameter of the specimen in mm.

MOGI (1962), submited prismatic specimens of marble with heights of 20, 8, 6 and 4 cm to uniaxial compressive tests, the lateral dimensions being half of the respective heights. The results - Fig. 3.12 - revealed a scale effect according to the equation:

$$S = So\ L^{-0.092}$$

where

So is the compressive strength of the smallest specimen tested and

S is the compressive strength of a given sample size, as percentage of So.

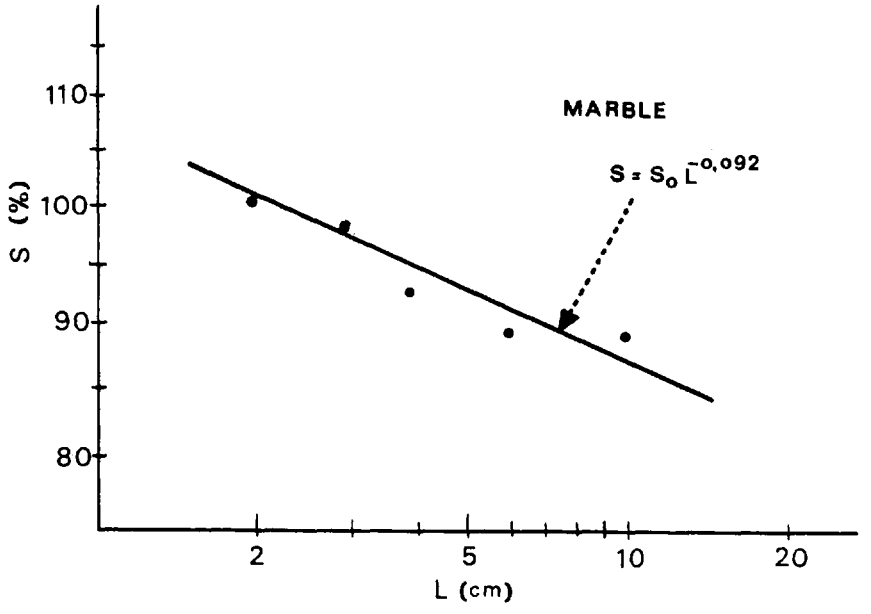


Fig. 3.12 - Relation between the compressive strength and the dimension of specimen in Yamaguchi marble. L: lateral dimension of prism (MOGI, 1962).

In the same paper, MOGI also referred to a very close correlation obtained by the U.S. Bureau of Reclamation (1949) for cylindrical specimens of concrete with aspect ratio of 2 and diameter between 5 and 90 cm, expressed by the equation $S = S_o L^{-0.107}$ (see Fig. 3.13, curve A). The results from the concrete specimens also showed, in addition, a certain independence between the Young modulus and specimen size (Fig. 3.13, curve B).

LANDBORG (1967), testing cylinders of "homogeneous" granite with diameters of 1.9 cm to 5.8 cm with an aspect ratio equal to 1, stated that the results show the existence of scale effect (with average values of 219 MPa for the 1.9 cm diameter samples and 175 MPa for those of 5.8 cm in diameter). The author also stated that the results were in accordance with the Weibull's theory (WEIBULL, 1939).

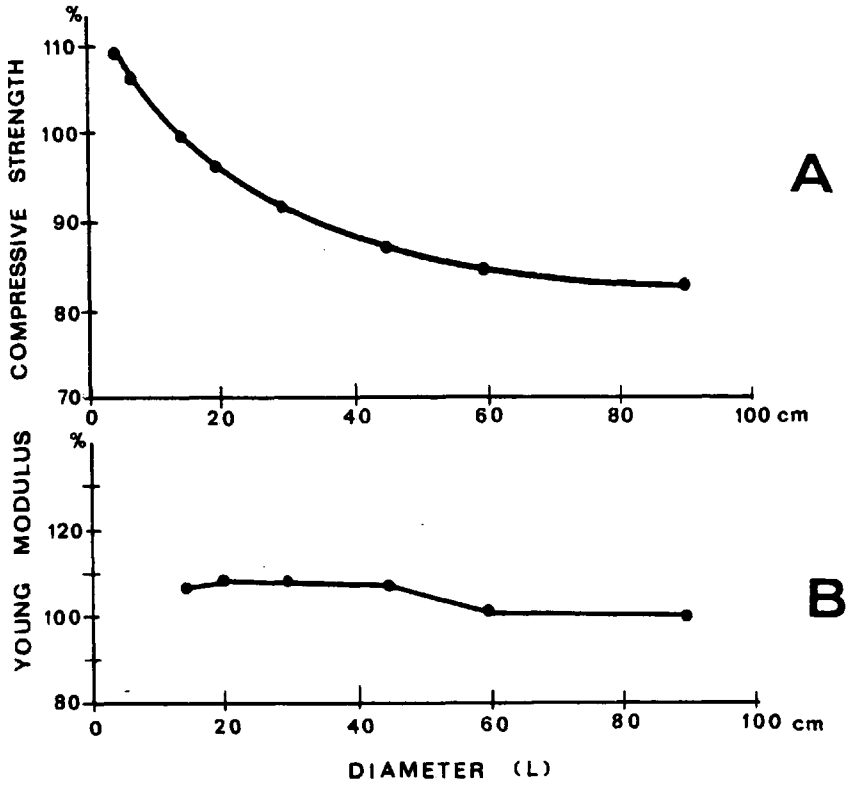


Fig. 3.13 - Relation between the compressive strength and the dimensions in concrete specimens (A). (B) shows the relation between Young's modulus and the specimen size. (After U.S. Bureau of Reclamation, in MOGI, 1962).

Cubic samples of coal covering an extensive range of dimensions were tested by BIENIAWSKI (1968). The cubes were arranged in three groups according to their dimensions, as follows:

Small size specimens	1.9 - 76. cm
Medium size specimens	15.2 - 45.7 cm
Large size specimens	61 cm - 2 m

The usual laboratory equipment was used for testing the small samples and other specially devised equipment was utilized in the case of the samples of larger dimensions. The results led the author to state that "there appears to be no more change in strength once the specimen size of 1.5 m is reached, which means that the results obtained in samples of that size could be applicable to the design of full-size coal pillars." Quoting PROTODIAKONOV and KOJFMAN (1964) and WEIBULL (1939) he also made it clear that their experimental results do not confirm either Protodiakonov's or Weibull's theories. According to Bieniawski, the strength-size relationship for coal (and perhaps for other rocks) cannot be described by one mathematical expression, as those presented by

either Protodiakonov or Weibull. Three different equations are needed for that purpose, in order to describe: i) the initial constant strength relationship (below a given small dimension the strength is independent of the sample size); ii) the subsequent strength reduction relationship with the strength decreasing as the dimensions increase and iii) the final constant strength relationship (after a given dimension the strength do not vary with sample size). It was also noticed that the scatter of the results gradually reduced as the size of test specimen increased.

A range of specimen sizes of unjointed quartz diorite and granodiorite were tested under static uniaxial laoding by PRATT et al. (1972) by means of laboratory and "in-situ" tests. The "in-situ" test specimens consisted of right equilateral triangular prisms varying approximately from 30.5 cm to 2.74 m in length. The laboratory test ones were triangular prisms from approximately 11.4 cm to 30.5 cm in length and cylinders with lengths 8.1 cm to 10.8 cm. The length-to-edge, or length-to-diameter ratios of specimens were 1.5 or more. The results from these tests, the maximum stress showing an assymptotic behaviour for samples greater than 90 cm (3 feet) in length, were found to be in agreement with the results of BIENIAWSKI (1968). The results (Fig. 3.14) show that there is a large decrease in strength with specimen size and the authors stated that "the results indicate that strength of laboratory samples is probably not representative of the strength of large volumes of "in-situ" unjointed rock, at least for the rock types tested.

JAHNS (1966) had also performed four "in-situ" tests on cubic specimens of calcareous iron ore with variable lengths up to 1 m. In addition 13 laboratory tests were carried out and the results showed a strength-size dependence. For example a 0.1 m cube exhibited a strength of 117 MPa against one of 49 MPa for a 1 m cube. The results were quoted by BIENIAWSKI and VAN HEERDEN (1975) and compared with those of PRATT et al. (1972) and BIENIAWSKI (1968) - fig. 3.14; they also referred to RICHTER (1968) who reported four tests of iron ore, sandstone and shale with dimensions varying from 1.25 m to 1.5 m (height), 1.25 m to 1.4 m (width) and 1.5 m to 2.15 (length), performed together with laboratory tests on cylinders 48 mm diameter. The strengths measured varied from 20 MPa (laboratory) to 360 MPa ("in-situ") for the iron ore specimens, and from 7.5 MPa to 30 MPa for the sandstone.

It was found, in addition, that there was no significant scale effect in the deformability modulus.

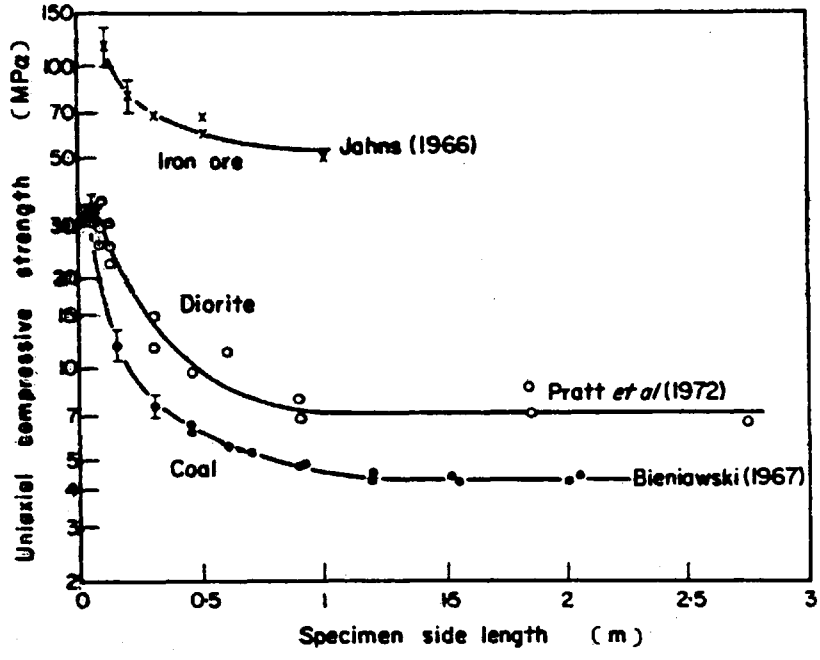


FIG. 3.14 - Strength - size dependence (BIENIAWSKI and VAN HEERDEN, 1975).

B - Influence of fractures

HABIB and BERNAIX (1966) studied the scale effect, particularly that due to fractures, in uniaxial compressive tests performed on cylindrical specimens with diameters varying from 10 mm to 60 mm and aspect ratio of 2. A summary of the results is presented in Table 3.VIII. These results led to the conclusion that the existence of a scale effect due to fissures becomes more pronounced as the number of fissures increases.

ROCK	LIMESTONE I	GNEISS II	LIMESTONE III	MALPASSET GNEISS right bank	MALPASSET GNEISS left bank
$\frac{\sigma_{uc}\ \varnothing 10mm}{\sigma_{uc}\ \varnothing 60mm}$	1	1.25	1.4	1.9	2.9

TABLE 3.VIII - Scale effect due to fissuration degree (HABIB and BERNAIX, 1966). The degree of fracturing increases in direction of the arrow. Limestone I is not fractured.

C - Influence of Stress gradient

The effects of both size and stress gradients on strength were studied by HODGSON and COOK (1970). Cylindrical samples of shale and quartzite with diameters varying approximately between 0.6 cm and 15 cm and with an aspect ratio of 3, were tested. Special precautions were taken to subject the specimens to a uniform stress and thus remove stress gradients. The authors concluded that the results obtained (Fig. 3.15) "indicate that the strength of a brittle rock is virtually independent of size under conditions of uniform stress". This contrasted with a severe scale effect that was obtained from the same materials tested under conditions specifically chosen to produce stress gradients.

This aspect of stress gradient on the strength of the rocks has been little studied despite their great importance as stated by JAEGER and COOK (1976).

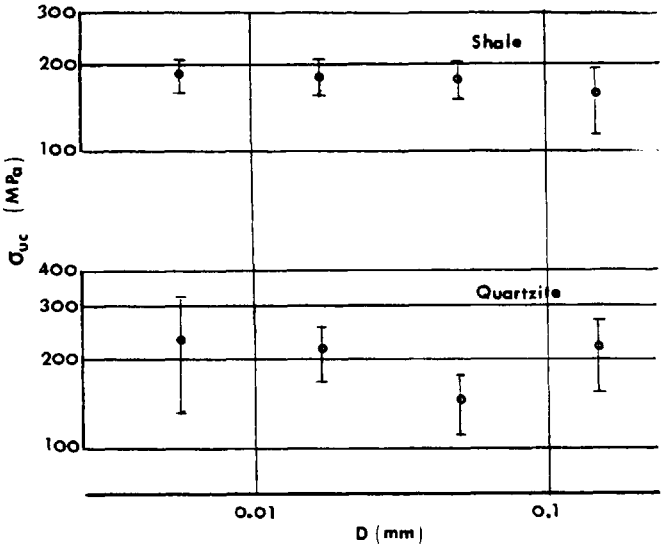


Fig. 3.15 - Variation of uniaxial compressive strength with specimen diameter (HODGSON and COOK, 1970).

D - Influence of the Strain Energy

The role in the scale effect of the strain energy stored within the failing specimen, in uniaxial compressive tests, has attracted the attention of several authors like GLUCKLICH and COHEN (1968), EINSTEIN et al. (1970), BROWN (1971) etc., but is still little studied for the

case of rock materials. More recently, ABOU-SAYED et al. (1976) reported the results of laboratory compressive tests carried out on quartz diorite from the same location (Cedar City) as the samples already tested by PRATT et al. (1972). Cylinders with diameters between 2.5 cm and 14.7 cm and aspect ratio of, at least, 2.5 were tested. The stiffness of the laboratory machine was modified (according to the authors) in an attempt to remove some of the uncertainties which arise when the effects of stored strain energy on the compressive strength tests of rocks are neglected. It was concluded that the results indicated a slight decrease in strength with an increase in size of the test samples (Fig. 3.16). It was also stated, by the authors, that "one is inclined to conclude that different size samples of the same material fail at

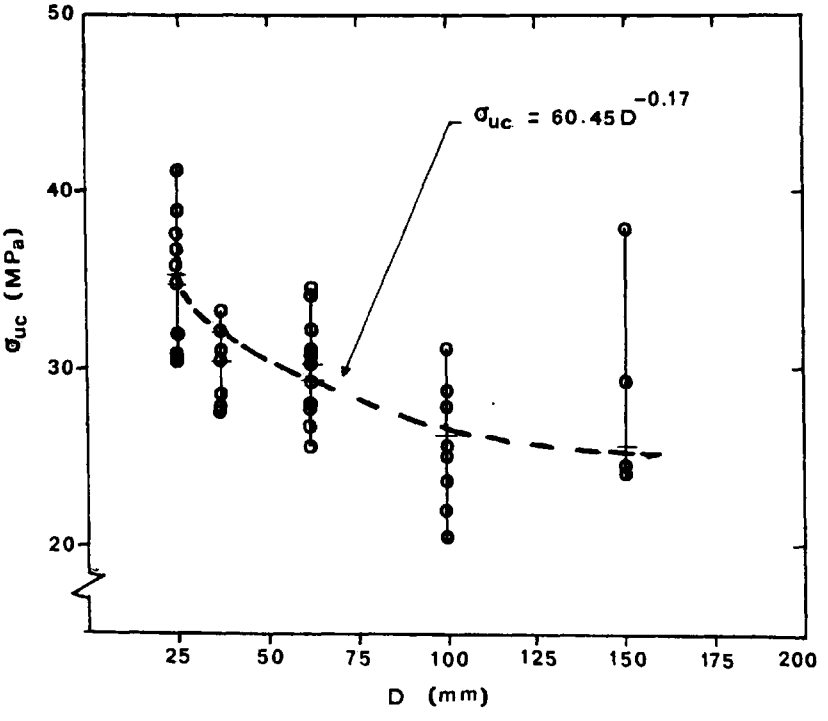


Fig. 3.16 - Variation of measured uniaxial compressive strength with specimen diameter (ABOU-SAYED et al., 1976).

different loads not because the material properties change with size, but rather because the geometry of the specimen and its constituents change. Such changes are due to inclusion of more joints, large cracks or other material nonhomogeneities".

E - The surface effects

HOUPERT (1970) in studying scale effect by testing cylindrical samples of granite with diameters between 2 cm and 10 cm and aspect ratio of 8 found that there was an increase in strength as the specimens diameters increased (Fig. 3.17). This inverse scale effect was explained by that author, quoting PROTODIAKONOV and KOJFMAN (1964), as being a secondary scale effect or a suface effect, related to the modification of the properties of the external surface of the specimens producing during their preparation. In certain rocks, especially the less fissured ones, this effect is predominant producing a scale effect that is the inverse of that due to the increase of the samples volume.

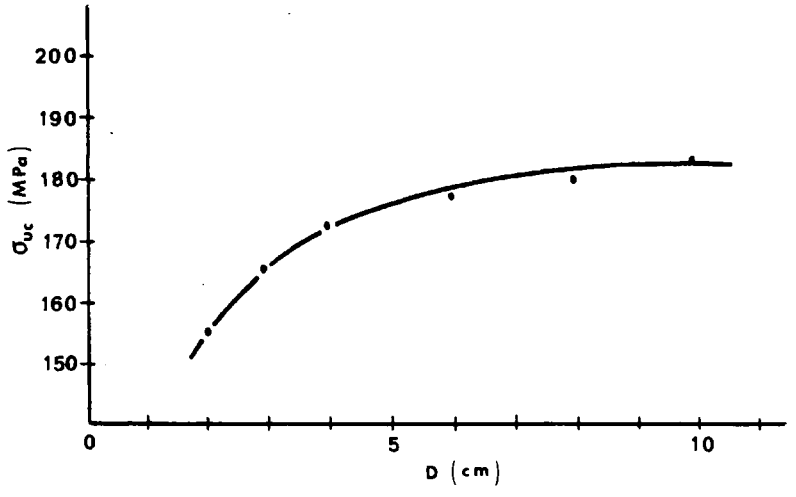


Fig. 3.17 - Inverse scale effect on cylindrical samples - see text (HOUPERT, 1970).

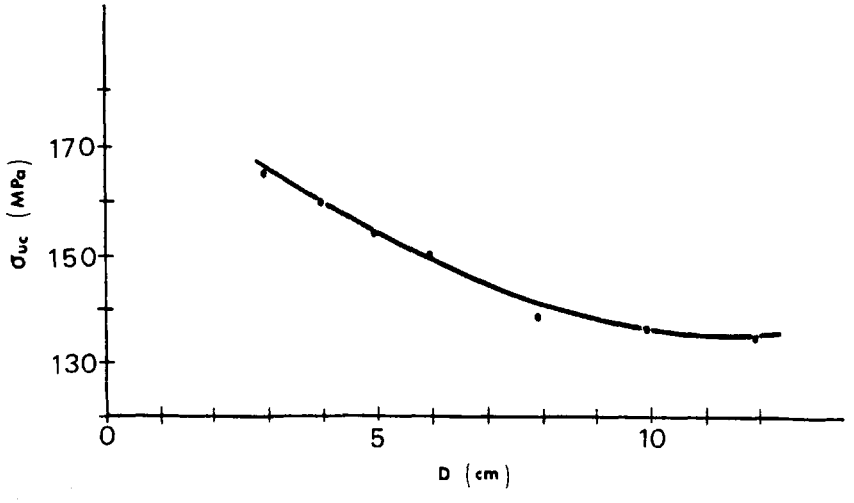


Fig. 3.18 - Scale effect on tube shaped specimens - see text . (HOUPERT, 1970).

To eliminate the surface effect, Houpert (1970) tested tubular specimens of the same material, with internal diameters from 3 cm to 12 cm, but having a constant volume/lateral surface relationship, the thickness of the specimens and the aspect ratio being equal to 0.8 cm and 5 respectively. These samples showed, in uniaxial compression, a decrease in strength as the diameter increased - Fig. 3.18.

F - The Influence of grain size

OROWAN (1949), as quoted by ROBERTS (1961), studied the influence of the grain size in the scale effect and stated that the brittle strength of a polycrystalline aggregate is inversely proportional to the square root of the mean grain size. To prevent the scale effect in this case, some empirical rules have been suggested. The ASTM (1942), in OBERT et al. (1946), suggested that for very coarse grain size materials like some granites, the diameter of the specimen should not be less than 6.4 cm (2.5 inches). JOHN (1962), recommended practical sample diameters of 7-100 times the grain size of the rock material, while ROBERTS (1977) stated that the ratio (test-specimen/ grain size diameter) should be at least 10:1, and quoted the ratio of 20:1 as recommended by other authors.

3.3.3 - THE SCALE EFFECT ON THE DEFORMABILITY OF THE ROCK MATERIAL

With regard to the deformability of the rocks subjected to uniaxial compression tests, it seems that a scale effect either does not exist or it is of little importance. MOGI (1962) did not achieve correlation between the specimen dimensions and the Young's modulus for the tested samples of marble. In the same way, as quoted by BIENIAWSKI et al. (1975), no significant scale effect was found in the tests performed by RICHTER (1968). Finally, PRATT et al. (1972) based upon the results of their experiments, found that "deformation modulus appears to decrease only slightly, if at all, with increase of specimen size".

In conclusion it can be said that the scale effect on the rock material is much more important in its effect upon strength than in its effect upon deformability. Thus, when these mechanical characteristics are to be determined for use in design, greatest care should be paid to investigating the scale effect on the strength. This will mean that in general more money may have to be spent in obtaining reliable values

for strength than for deformability of the rock materials.

3.3.4 - THE SCALE EFFECT ON THE ROCK MASSES

While microfractures, heterogeneity and grain size are factors of significant influence in the mechanical behaviour of the rock material, especially in its strength, their influence becomes insignificant for the case of rock masses. In this case, other elements dominate. These are the macro-discontinuities as well as the anisotropy and heterogeneity which exist at the rock mass scale.

Nowadays, there is no doubt that the role of the joints is of extreme relevance in affecting both strength and deformability of the rock masses. It can be said that the joints are what essentially control the mechanical behaviour of rock masses at depths conventionally reached by civil engineering works and as such, the rock masses are generally weaker and more deformable than the rocks from which they are formed.

Due to the influence of the discontinuities, the rock mass strength can vary to as low as 1/300 of that of the rock material (MULLER, 1963).

In Table 3.IX some comparative results dealing with the laboratory (Er) and "in-situ" (Em) deformability modulus for several sites studied at LNEC-LISBON, are presented.

PRATT et al. (1974) reported the "in-situ" deformability modulus of multiple-jointed quartz diorite which was seven times lower than the intact rock modulus.

BURLAND and LORD (1970) compared deformability modulus of chalk obtained in laboratory tests with that "in-situ" by performing a full scale test using a tank with a diameter of 18.3 m. The E_m/E_r relationships for the different chalk grades tested varied between 1:1.3 and 1:10 (Table 3.X).

The decrease of the Em/Er relationship with a decrease in the joint spacing was also verified.

PRICE and KNILL (1974) presented some examples of the variation of both deformability and strength with the volume (or area) tested, with deformability increasing and strength decreasing with increased size of test; their examples considered results from laboratory compression

tests, borehole tests, plate bearing tests and prototype results.

Rock Type	Site	Modulus of deformability 10^3 MPa		$\frac{E_m}{E_r}$
		Labor. E_r	In-situ E_m	
Granite	Alvarenga (P)	52	49	1/1.1
Granite	Alto Rabagão (P)	2.6	0.9	1/2.9
Granite	Alto Lindoso (P)	32	6	1/5.3
Granite	Vilarinho (P)	43	1.5	1/29
Gneiss	Cabora - Bassa (P)	80	65	1/1.2
Schist	Cedillo* (S)	90	40	1/2.2
Schist	Cedillo** (S)	65	12	1/5.4
Schist	Alcantara* (S)	140	5	1/28
Conglomerate	Avlaki (G)	60	6	1/10
Sandstone	Cambambe (P)	65	8.6	1/7.6
Silstone	Avlaki (G)	15	1.5	1/10
Claystone	Karun (I)	11.5	7	1/1.6
Marl	Karun (I)	47	43	1/1.1
Limestone	Karun (I)	70	60	1/1.2
Limestone	Karun (I)	50	7.5	1/67
Quartzite	Alvito (P)	43	0.4	1/108
Quartzite	Alvito (P)	33	7	1/4.7

*Parallel to schistosity

**Perpendicular to schistosity

P — Portugal S — Spain G — Greece I — Iran

TABLE 3.IX - Comparison between laboratory and "in-situ" deformability modulus (ROCHA, 1974, modified).

Grade	Brief description	Em/Er
V	Structureless remoulded chalk containing lumps of intact chalk.	$\frac{1}{10}$
IV	Rubbly partly-weathered chalk with bedding and jointing. Joints 10-60 mm apart, open up to 20 mm and often infilled with soft remoulded chalk and fragments.	$\frac{1}{10} - \frac{1}{5}$
III	Rubbly to blocky unweathered chalk. Joints 60-200 mm apart, open up to 3 mm and sometimes infilled with fragments.	$\frac{1}{5} - \frac{1}{2.5}$
II	Blocky medium-hard chalk. Joints more than 200 mm apart and closed.	$\frac{1}{1.7} - \frac{1}{1.3}$
I	As for Grade II but hard and brittle.	

TABLE 3.X - Relationship between deformability of chalk, obtained "in-situ" - Em, and in laboratory - Er (BURLAND and LORD, 1970).

Care must be taken when considering what constitutes a rock mass: basically it is composed of rock material cut by sets of discontinuities which devide the rock in blocks. According to JOHN (1962) "unit rock block refers to the smallest homogeneous rock unit produced by a system of geological separations intersecting a rock mass". The average volume of unit rock blocks is given by:

$$V_{UB} = \frac{1}{ka} \cdot \frac{1}{kb} \cdot \frac{1}{kc} = (da) \cdot (db) \cdot (dc)$$

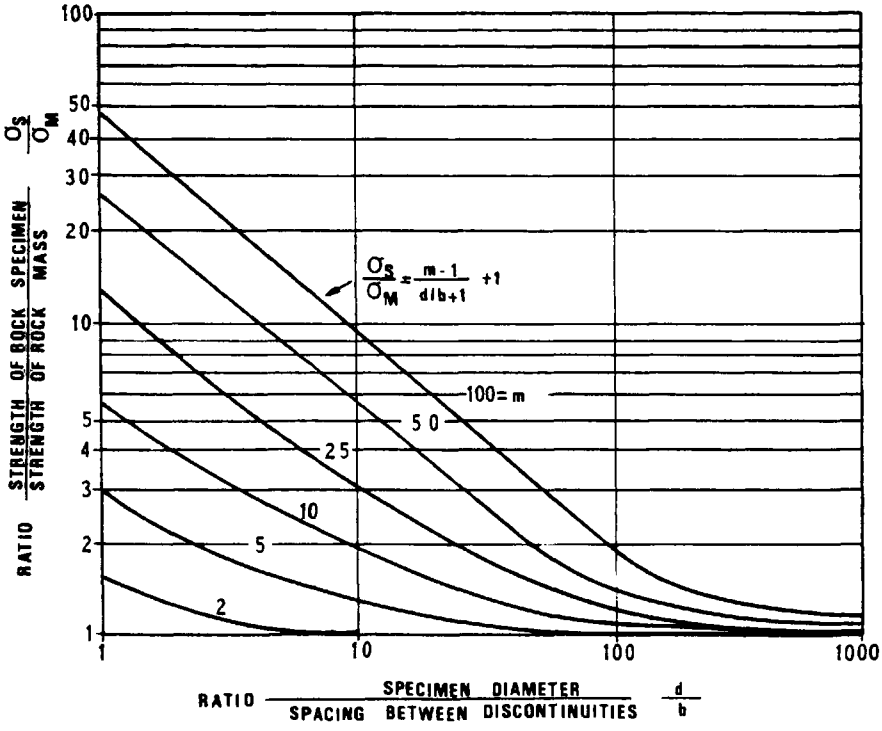
where ka, kb, kc are the frequency, and da, db, dc, are the spacing of each of the sets of separations.

As stated by BRIOILI (1974), "the mechanical properties of a rock mass are based more on the mechanical properties of planar structures (geological discontinuities) than on the properties of the rock considered as rock material. Therefore, rock mechanics is essentially mechanics of the structures and respectively mechanics of a discontinuous". This is the reason for more and more attention being given either to the study of the physical and geometric characteristics of the discontinuities or to the study of their mechanical behaviour at levels of stress that permit their presence to affect the mechanical behaviour of the rock mass.

Laboratory testing on samples which could represent the rock masses as a physical body in such a way that their discontinuites, heterogeneities and anisotropy could be taken into account, is not possible. This leads to the necessity of performing "in-situ" tests involving greater volumes. Even so, it must be said that most times the volumes are much smaller than would be desirable, due to either practical or economic reasons. In addition, the number of tests performed is frequently small because the money and time involved. In such cases, the mechanical characterization of the rock mass is deficient.

In order to solve the problem caused by heterogeneity, on a scale of the rock masses, the most appropriate methodology consists in dividing the rock masses into zones or units which can be considered as homogeneous (geotechnical zoning). This is one of the most important tasks required of the engineering geologist. Later, the mechanical behaviour of each of these zones is assessed through the available methods, mainly by means of "in-situ" testing.

For the tests to be considered as representative with regard the state of jointing, some desirable dimensions (volume of ground to be tested) have been suggested. MULLER (1963) suggests that "the number of fractures in a test sample volume should be no less than 200 or the sample volume should contain 100 or 200 unit rock blocks". PENDER et al. (1963) states that if fracture spacing is 5 cm, tests on 90 x 30 cm cylinders probably would be of value for laboratory strength tests. This would be equivalent to testing a maximum of 17 fractures belonging to one set or approximately 500 unit rock blocks.. Nevertheless, JOHN (1962) quoted the practical test dimension, commonly used in civil engineering, as varying from 8 to 16 times the unit rock block. According to Protodiakonov's relationship (see Fig. 3.19), and based on the values of (m) suggested by HOEK (1966), for rock masses constituted by hard rocks subjected to compression ($2 < m < 5$), the diameter of the specimen should be 4 to 19 times the joint spacing to obtain a strength value which fail within 20% of that of the rock mass. Considering an aspect ratio of 2, the approximate minimum number of blocks which should be tested in this case would be 100 ($m = 2$).



d - diameter of the specimen b - joint spacing

m - parameter which depends upon the nature of the material and also upon the state of stress which the specimen is subjected.

FIG. 3.19 - Relationship between specimen size and strength according to PROTODIAKONOV (in HOEK, 1966).

Experimental results comparing the variation in compressive strength of rock masses with size of rock mass tested are very scarce and the few available results are practically limited to studies dealing with coal. This happens because coal, as a soft rock, causes less practical problems to test performance and is extensively mined into pillars. However, it is a recognised fact that generally, the failure of rock masses involved in large civil engineering works is in shear. This failure can involve the rock material itself, but tends to occur preferentially along low resistence surface (discontinuities) and so the most important strength tests to be carried out "in-situ" are the shear strength tests along those mentioned surfaces.

Some examples of values for both rock shear strength and joint shear strength obtained by means of laboratory and "in-situ" tests involving areas of 10 x 10 cm^2 and 70 x 70 cm^2 respectively are presented in Table 3.XI, according to ROCHA (1973).

ROCK SHEAR STRENGTH

ROCK TYPE	LABORATORY		"IN-SITU"	
	c (MPa)	$\phi(^o)$	c (MPa)	ϕ (^{o})
Shale*	4.4	66	1.2	53
Granite	4.4	71	1.0	58
Gneiss	3.2	37	4.0	56
Granite	3.7	61	2.4	56

* Shear normal to the schistosity σ and τ parallel to the schistosity

JOINT SHEAR STRENGTH

ROCK TYPE	LABORATORY		"IN-SITU"	
	c (MPa)	$\phi(^o)$	c (MPa)	ϕ (^{o})
Clay shale*	0.22	38	0.31	36
Granite	0.17	36	0.30	34
Gneiss	0.34	38	0.35	33

* Parallel to the schistosity.

TABLE 3.XI - Rock shear strength and joint shear strength obtained by means of laboratory and "in-situ" tests.

According to the above mentioned author, laboratory rock shear strength tests will lead, in general, to greater values of the shear parameters than the "in-situ" tests because the specimens which can be tested in the laboratory are not representative, as a rule, either of the poor quality zones in an heterogeneous rock mass or of its jointing. The shear strength decreases as the total area involved by the tests increase and this is where the back analyses of failed slopes is so valuable: it indicates the variation in strength that can occur with size (see for example KRAHN and MORGENSTERN, 1976).

The similar results between the laboratory and "in-situ" joint shear tests permitted ROCHA (1973) to conclude that it is possible to determine the joint shear parameters in the laboratory if irregularities like undulation and roughness which it would be possible to represent only by means of larger specimens, are absent (see Fig. 3.20). Nevertheless, such laboratory assessment requires that a great number of samples be tested (10 as a minimum).

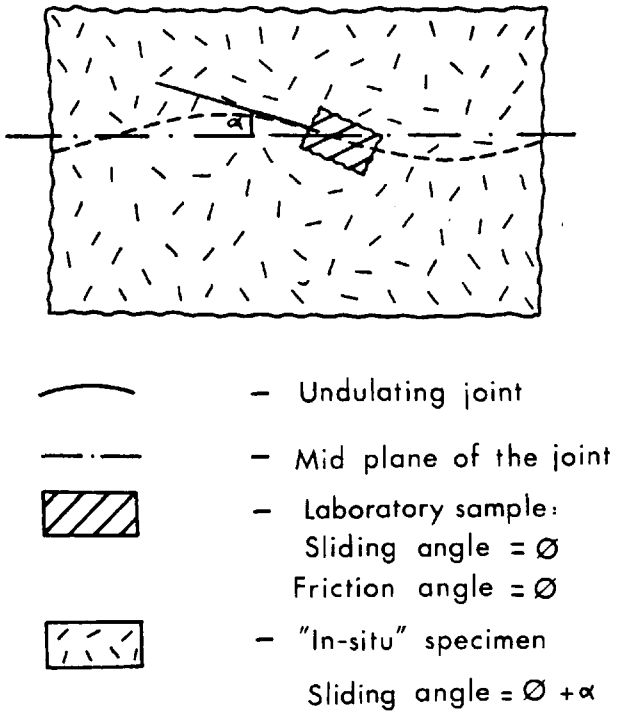


FIG. 3.20 - Comparison between laboratory and "in-situ" joint shear specimens regarding the undulation.

Undulations introduce a scale effect which consists, in general, of an increase in strength as the shear area increases, because the overall resistance to shear that is mobilized is greater. Thus, the "in-situ" tests tend to give less conservative results than the ones performed in the laboratory. If (Fig. 3.20) ϕ is the friction angle of a smooth joint, the existence of undulation making an angle with its mid-plane will increase the sliding angle to $\phi + \alpha$. When the values of the normal stress are great enough, shear will take place along the mid-plane of the joint (Fig. 3.20) and so the frictional strength assessed by the tests tend to that of the rock material itself.

LOCHER and RIEDER (1970) are of the opinion that as a whole the laboratory testing on small samples seems to give fairly reliable, although slightly conservative results. They reported results of both laboratory and "in situ" shear tests on generally smooth bedding planes of a limestone, concerning shear areas of 0.02 m^2 and 3 m^2 respectively. The field tests showed a friction angle of 5^o higher than the laboratory ones.

The fact that these authors have obtained more conservative values for friction angle in the laboratory as opposed to the results presented by Rocha, can be attributed to the presence of some irregularities in the shear surfaces which, due to the great difference in the shear areas involved were present only in the large dimension tests.

Roughness and undulation are, nevertheless, aspects which are still seldom taken into account, when they cannot be duly represented in the test dimensions used, as in the case of undulations with large wavelength. Their geometrical characterization remains difficult in spite of methods developed for this purpose (e.g. RENGERS, 1970; FECKER and RENGERS, 1971).

BARTON (1973), based on experimental observations of the behaviour of rough artificial joints, studied an empirical relationship between joint shear strength (τ_{peak}) and roughness degree (joint roughness coefficient - JRC), and later BARTON and CHAUBEY (1977) published several typical rougnness profiles (Fig. 3.21) together with the corresponding JRC values in order to make it possible the assessment of ϕ_{peak} by the formula:

$$\phi_{peak} = JCR \log_{10} \frac{JCS}{\sigma_n} + \phi_r$$

where:

JCS - joint wall compressive strength

σ_n - effective normal stress

ϕ_r - residual friction angle

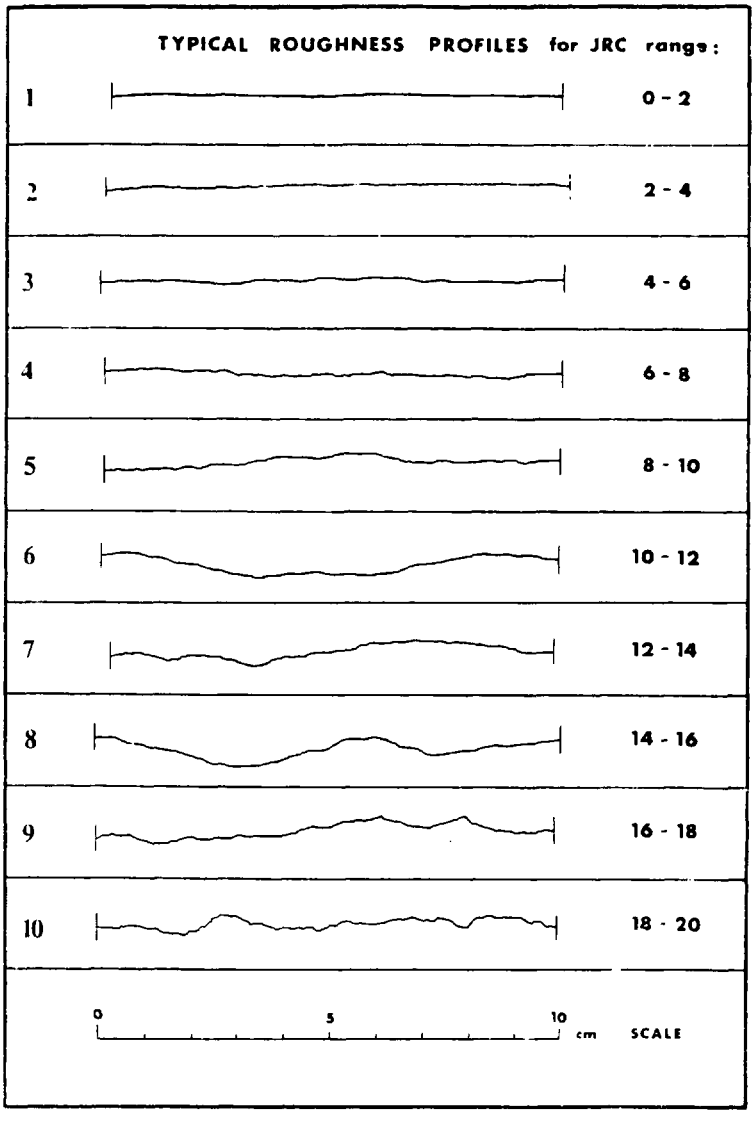


Fig. 3.21 - Roughness profiles showing the typical range of JRC values associated with each one (BARTON and CHAUBEY, 1977).

BARTON AND CHOUBEY (1977) and BARTON and BANDIS (1980) paid attention to the scale effect, stating that JRC is only a constant for a fixed joint length and generally longer profiles have lower JRC. This is equivalent to stating that the scale effect consists in a

decrease of strength as the dimensions of the test increases. They further stated that in jointed rock masses smaller blocks formed by closer spaced joints have greater freedom to follow and "feel" the smaller scale and steeper asperities of the joints and hence respond as if they have higher JRC values (Fig. 3.22). Values of JRC ranging between 9.1 and 16.1 for joint spacings between 60 mm and 360 mm were obtained by BANDIS (1979).

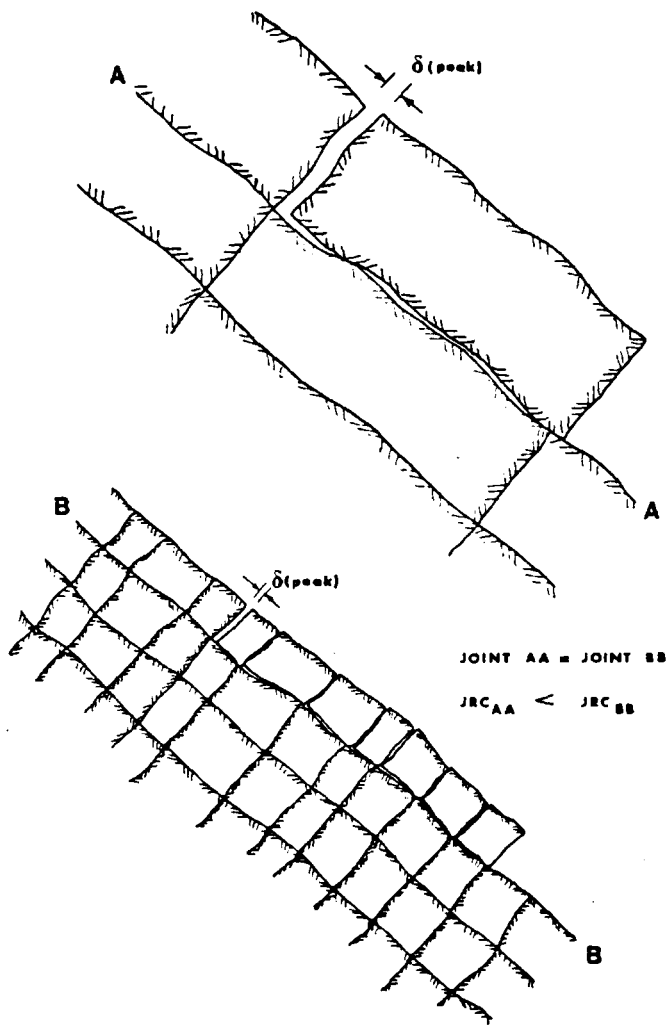


Fig. 3.22 - Scale effect determined by block size
(From BARTON and CHAUBEY, 1977)

Scale effect resulting from area can also be observed with single joints provided they are not smooth and loaded to the point where the opposite surfaces of the joint mate. PRATT et al. (1974) tested a quartz diorite shearing samples that had different total areas and found that shear strength decreased by 40% over a range of surface areas from 142 cm^2 (laboratory) to 5131 cm^2 ("in-situ").

The above authors stated that this decrease is due to the lower contact area of the specimens with large surface areas.

Special reference should be made to rock masses made of low strength rocks. These low strengths can be due to the rock composition (e.g.chalk) or to an advanced state of weathering as can happen to many rocksclassified as hard when they are fresh[1].

With regard to deformability, it is in these rock masses that its determination needs to be made with great accuracy. In the case of concrete arch dams for example, where the consideration of the deformability of the foundations is of utmost importance, its influence in the state of stress of the structure (ROCHA, 1964, 1975) is high for rock masses with a deformability modulus (E_m) < 3 000 MPa (about $\frac{1}{8}$ of that of the concrete - E_c), becoming very high when E_m < 1500 MPa ($E_m < \frac{1}{16} E_c$) - see table 3.XII.

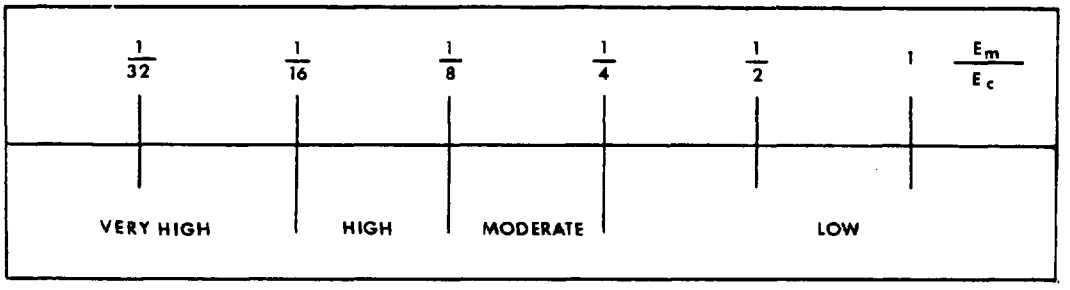


TABLE 3.XII - Influence of the deformability of the foundations in the state of stress of arch dams (from ROCHA, 1974).

As rock masses of soft rocks have, in general, deformability modulus lower than 4 000 MPa, it can be concluded that in such cases the characterization of their deformability and its variation along the foundation is required with increasing accuracy as deformability increases. In these cases the assessment of the scale effect on deformability would be very important especially if the influence of the discontinuities was assumed identical to that in rock masses of

(1) According to ISRM's classification (1980), low strength rocks exhibit uniaxial compressive strength values lower than 20 MPa, while those with uniaxial compressive strength lower than 6 MPa are considered as of very low strength.

high strength rock. As stated by many (e.g. ROCHA, 1975) the importance of the scale effect due to jointing decreases, as a rule, as the strength of the constituent rocks decreases, in such a way that for rock masses composed essentially of low strength rocks one can, most times, consider the rock mass deformability as being appropriately indicated by that of the rock material. As a consequence, the deformability characterisation of this kind of rock mass can, with a certain frequency, be assessed by means of laboratory tests if heterogeneity does not call for samples with excessive dimensions and if sampling operations do not greatly disturb the specimens. The results of PRIEST et al. (1976) are not in line with the above. Based on several laboratory and "in-situ" tests by using the Menard pressure meter carried out for the Chinnor tunnel, they found that the results suggested that the chalk mass could be expected to behave as if having a modulus as low as 4% of that obtained from laboratory tests on intact samples. Working in the same site, PRIEST (1976) stated that back calculations from ground deformation measurements using finit element analysis indicated a modulus for the chalk rock mass that was 5% to 7% of that of the intact modulus, that is, the modulus measured on an intact laboratory specimen of chalk. The chalk rock mass studied was grade III - IV (see Table 3.X), the joints showing no significant opening or infilling.

With regard to the shear strength of rock masses, the influence of discontinuities decreases as the strength of the rock material decreases. The shear occurs more and more within the rock material itself and so, the shear strength of the rock mass tends to the one of the rock material. In general, therefore, the scale effect on shear strength caused by the presence of discontinuities diminish in importance as the strength of the rock material decreases.

3.3.5 - THE SCALE EFFECT: GENERAL DISCUSSION

With the above, the author tries to focus attention on some aspects related to the complex problem of scale effect which concerns mechanical characterization.

The effect of the scale upon the strength of rock material is generally recognized as a complex and not yet satisfactorily solved

problem in spite of various theories and considerable experimental work, examples of which have been presented here.

It has been shown that this scale effect can be due to or influenced by different factors (size of specimen, stress gradient, degree of fracturing, grain size, etc.), which are difficult to take in account together.

One thing is certain: the studies already undertaken lead to the conclusion that the scale effect on the strength of rock material is not simply a function of laboratory practice and specimen preparation, but is a material response that must be remembered when assessing the values of the strength to use in practice, i.e., "in-situ".

This effect is even beyond the domain of "in-situ" tests because of the dimensions of the rock mass involved, however at present the best approach is based on the site investigaton results, as there is no possibility of testing samples with such dimensions like the ones involved by the prototypes, e.g. the dams: it is here that the careful back analysis of failures is of value.

Other important aspect consists of whether or not the load conditions imposed by the construction are appropriately reproduced during the tests.

In addition, another question remains which can also be considered as a scale effect in terms of time, in the assessment of deformability. This deals with the loading time involved in the tests which is, as a rule, extraordinarly small when compared with the life of the construction. Although the time can be relevant in certain cases, it cannot be reporduced in full and sometimes it is even not considered.

The values of mechanical characteristics to be used for design purposes have to be extrapolated from both the tests that are possible to perform and the knowledge of the engineering geological features that, at the construction scale, can influence the mechanical behaviour of the rock masses. The success of this is mainly dependent upon the knowledge of the stresses to be imposed by the construction, upon appropriate geotechnical zoning, upon the number, dimensions and representativeness of the samples, upon the reliability of the performed tests and upon the skill and training of the people responsible for the site investigations. Finally, the great importance

of case histories should be appreciated. The behaviour of prototypes reveals the effect of scale on both the material properties of strength and deformation and their transience with time.

Obviously, scale effect can be considered when planning a site investigation and the costs related to its consideration can be evaluated if required, however, the benefits that come from assessing scale effect are almost unquantifiable.

3.4 - WATER ABSORPTION TESTS (LUGEON TESTS)

3.4.1 - GENERAL

The hydrogeological characteristics of rock masses, especially permeability, can be very important to either the construction phase or the operation period of an undertaking. It is enough to remember the dangerous problems which can arise due to an unexpected percolation either through the foundation of dams or from the reservoirs, e.g. uplift, degradation of the mechanical characteristics, leakage, etc. In tunnels, several cases of waste of money and time due to unanticipated problems caused by water flow during construction are known.

During site investigations, the permeability of rock masses is generally assessed, in an indirect way, by performing the well known Lugeon or packer test. The information obtainable will be discussed here together with the costs and time involved in the execution of these tests.

The primary permeability, i.e. the permeability of the rock material, is generally very low, much inferior to that of the rock masses, and is only determined by laboratory tests. In Table 3.XIII permeability coefficients for some rock types are presented, according to SERAFIM (1968).

Percolation through the rock masses is designated secondary permeability. The water absorption tests[1] inside boreholes is used

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(1) The designation of the results from Lugeon tests as the direct equivalent of permeability tests is incorrect; to emphasise their difference it is Portuguese practice to consider the Lugeon tests as water absorption tests.

Rock material	k (cm/sec) (lab. determination)
Sandstone (Cretaceous flysch)	10^{-8} to 10^{-10}
Siltstone (Cretaceous flysch)	10^{-8} to 10^{-9}
Granite	5×10^{-11} to 2×10^{-10}
Slate	7×10^{-11} to $1{\cdot}6 \times 10^{-10}$
Breccia	$4{\cdot}6 \times 10^{-10}$
Calcite	7×10^{-10} to $9{\cdot}3 \times 10^{-8}$
Limestone	7×10^{-10} to $1{\cdot}2 \times 10^{-7}$
Dolomite	$4{\cdot}6 \times 10^{-9}$ to $1{\cdot}2 \times 10^{-8}$
Sandstone	$1{\cdot}6 \times 10^{-7}$ to $1{\cdot}2 \times 10^{-5}$
Hard mudstone	6×10^{-7} to 2×10^{-6}
Black schists (fissured)	10^{-4} to 3×10^{-4}
Fine-grained sandstone	2×10^{-7}
Oolitic rock	$1{\cdot}3 \times 10^{-6}$
Bradfort sandstone	$2{\cdot}2 \times 10^{-5}$ to 6×10^{-7}
Glenrose sandstone	$1{\cdot}5 \times 10^{-3}$ to $1{\cdot}3 \times 10^{-4}$
Altered granite	$0{\cdot}6$ to $1{\cdot}5 \times 10^{-5}$

TABLE 3.XIII Coefficients of permeability of various rock materials

for its indirect assessment: values are recorded as Lugeon units (Lugeon, 1933), one Lugeon being approximately equal to $K = 1 \times 10^{-5}$ cm. sec^{-1}. Secondary permeability takes place through existing discontinuities. Its characteristics depend upon geometrical (spacing continuity, opening) and physical (infilling, roughness) properties of those discontinuities.

The Lugeon tests, devised by the Swiss geologist Maurice Lugeon early in this century, are often referred to in English as packer tests. Having been initially developed for assessing the percolation through dam foundations and for designing their treatment (LUGEON, 1933, the tests were later extended to situations where the assessment of rock mass permeability is important.

This is a type of test to be performed with care and whose results must be interpreted with caution. ROSELLO et al. (1974), for example, state with respect to this, that the tests are not always well performed and results are seldom correctly interpreted. Both, methodologies and techniques are discussed in several works (e.g. LUGEON, 1933; GUERRA et al., 1967; MAINI, 1971; OLIVEIRA, 1972; ROSELLO et al., 1974; LANCASTER JONES, 1975; DIXON and CLARKE, 1975 HOULSBY, 1976; PEARSON and MONEY, 1977).

The water absorption unit is the "Lugeon" which corresponds to an absorption of 1 litre per linear metre of borehole, per minute, using a water injection pressure of $1 MN/m^2$ in excess of existing pressure head in the ground. The test is usually performed during 10 minutes to

allow estabilization.

For assessing the equivalent coefficient of permeability from results of the absorption tests, a formula like the one proposed by the U.S. Bureau of Reclamation (DIXON and CLARKE, 1975) can be used; thus

$$K = \frac{q}{C_s RH}$$

where:

q - steady flow rate under an effective applied head H (corrected from friction losses)

R - radius of the drill hole

C_s - constant depending upon the length and diameter of the test section.

The equivalent permeability coefficient (k) refers to the response from an homogeneous permeable media under the same H used in the Lugeon test, and the above equation is only valid for laminar flow, i.e. when the flow regimen falls within the limits of applicability of the Darcy's low.

3.4.2 - INFORMATION FROM THE WATER ABSORPTION TESTS

The main types of information which can be obtained when Lugeon tests are performed are those presented in Table 3.XIV.

* Assessment of the equivalent permeability of the rock mass
* Indications about certain physical characteristics of the joints
* An evaluation of the hydrological regime through the rock mass
* Assessment of the need to grout, the efficiency of grouting and to establish the depth to be reached by grouting treatment

TABLE 3.XIV - Main types of information obtainable by performing Lugeon tests.

Beyond the assessment of the "permeability" of rock masses, Lugeon tests can be used to obtain other kinds of important information by considering the form of the water pressure (P) - flow (Q) curves, when several pressure levels are used in a programme where they are increased and then decreased. These curves reflect the flow type, the physical characteristics of the fissures and other phenomena which occur during the tests. Their interpretation can make a useful contribution to the understanding of a rock mass and its response to an hydraulic gradient, and such comparisons should always be attempted.

Although the P-Q curves can exhibit great variability, as shown for example by DIXON and CLARKE (1975), their main types can be reduced to five and these are presented in Fig. 3.23. Of these, type A (laminar) means that Darcy's law was obeyed during the tests, there being a linear relationship between the head applied and the flow. This means, in general, that flow took place through thin unfilled fissures or through fissures filled by granular material. An increase of fissure infilling can cause the descending relationship between Q and P to differ from the ascending relationship.

When the fissures are wide, the turbulent flow (type B) can take place and the P-Q relationship is influenced by the fissure wall roughness. In a typical turbulent type the Chezy's law is obeyed, the flow (Q) being a function of the square root of the head applied (P).

The void filling type (type C) is identified by the considerable difference between the Q values at the beginning and end of the test. In the decreasing stage of the test the flow is lower than in the increasing stage, for a same head used. In opposition to this, the type D (wash-out) shows greater flow in the decreasing stage than in the increasing stage of the test, for a same head. The occurence of this type of curve indicates that fissures have been washed out during the test.

Finally type E can occur when the pressure used are great enough to cause the existing ground structure to change.

The frequency of each type of curve was studied by HOULSBY (1976) regarding four dam sites with a total of 811 tests. This author recommended which of the values of the absorption coefficient, for each particular curve type, should be used - see table 3.XV. At the present there is no internationally recommended method of either interpreting the results or deciding which results should be used.

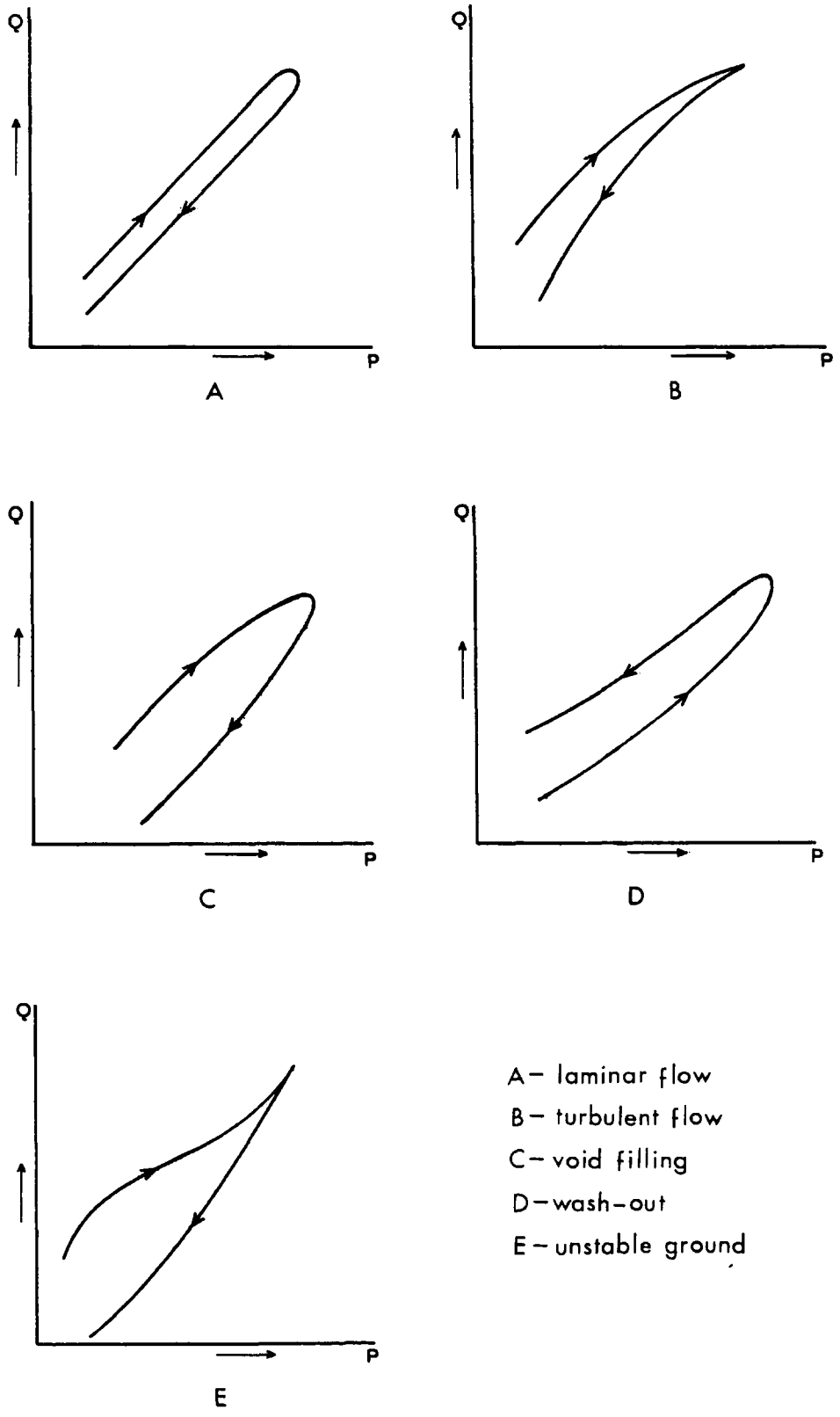


FIG. 3.23 - Main typical pressure (P) - flow (Q) curves for Lugeon tests

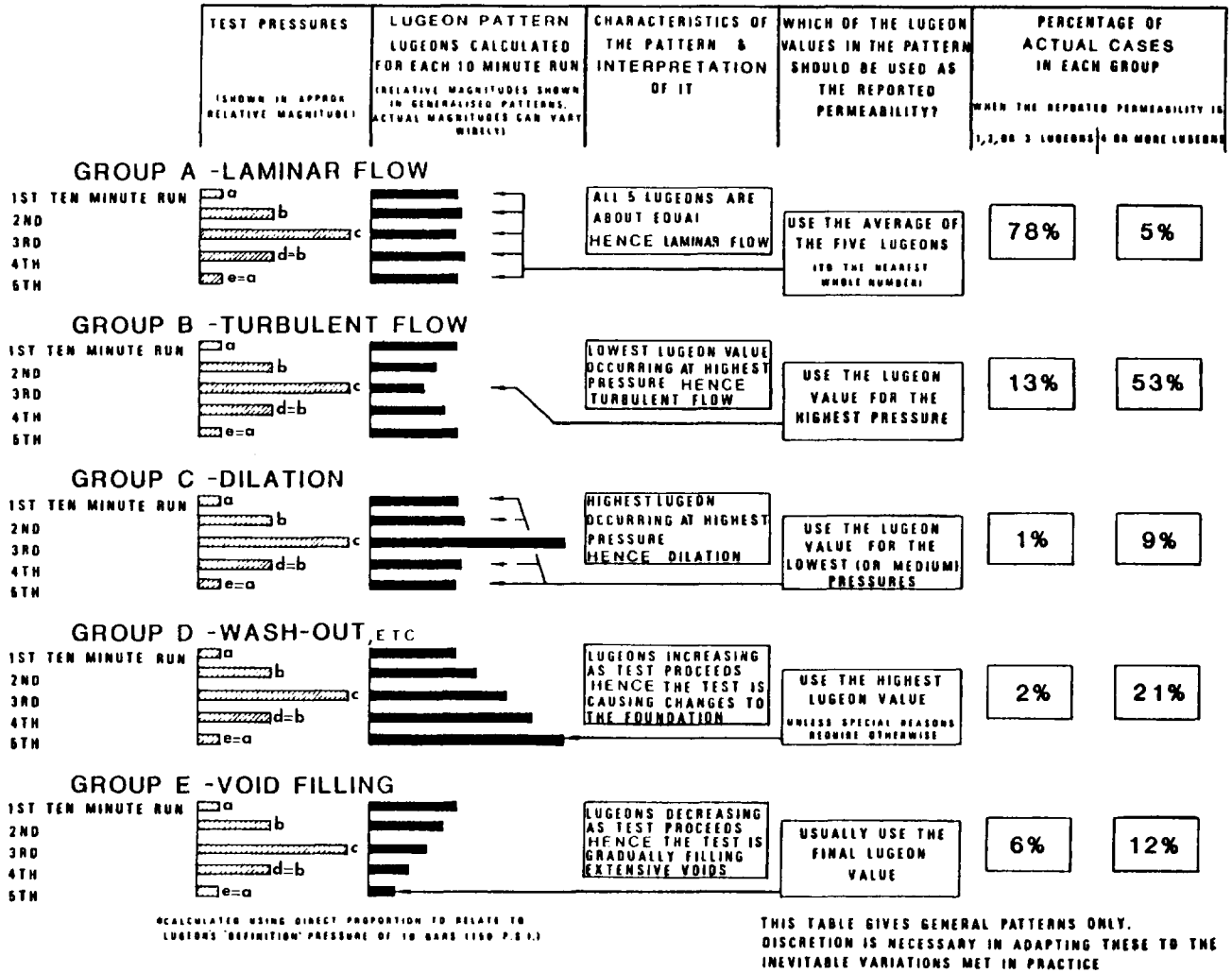


TABLE 3. XV - Lugeon patterns for various occurences during testing: their interpretation and percentage of occurences (HOULSBY, 1976) - see text .

In interpreting the Lugeon test results it is fundamental to take into account the state of jointing obtained by a careful examination of the rock cores, especially when the results are to be used to design ground treatment (e.g. MAINI, 1971), and to bear in mind that tests give average values of the rock mass "permeability" for the section tested (SERAFIM, 1972). The relationship between Lugeon values and the water returns measured during drilling can be helpful.

SABARLY (1968) call the attention to the fact that in addition to being proportional to the injection pressure, the inflow rate is also proportional to the 3rd power of fissure width. ATTEWELL and FARMER (1976), quoting SABARLY and also VAUGHAN (1963), present the following relationship between permeability coefficient, the number of fissures and their width, for an ideal case of fissures normal to the borehole - Fig. 3.24 and Table 3.XVI.

$$\delta^3 = \frac{12\,\eta\,k_r}{N\,\gamma_w}$$

where

δ - fissure width
η - coefficient of viscosity of water
k_r - radial permeability coefficeint
γ_w - density of the water
N - fissure spacing

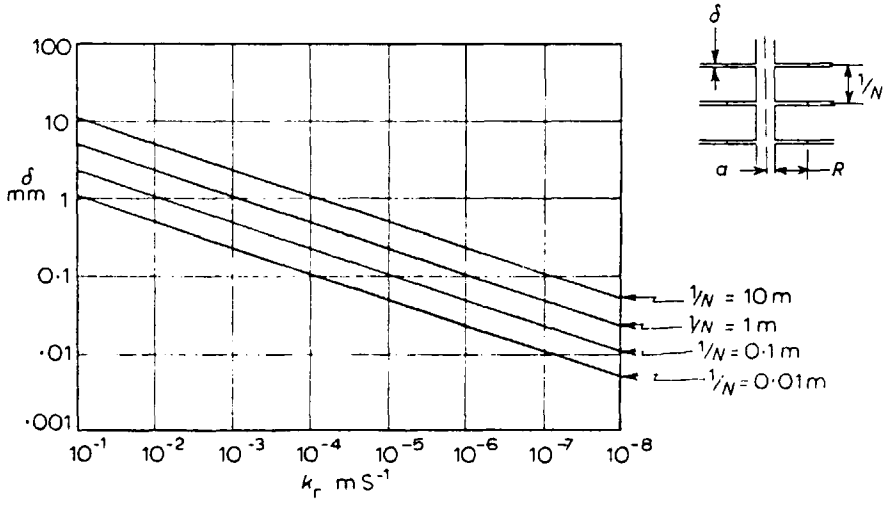


Figure 3.24 - Fissure width - spacing - permeability relationship (in ATTEWELL and FARMER, 1976).

Table 3.XVI, which was derived from the above equation, shows that the permeability coefficient corresponding to one Lugeon can be due to a single fissure 0.25 mm wide per metre, or to ten fissures 0.12 mm wide per metre, or still to one hundred fissures 0.05 mm wide per metre.

k_r m s^{-1}	δ mm			
	1/N = 10 m	1/N = 1m	1/N = 0.1 m	1/N = 10 mm
10^{-1}	11.5	5.3	2.5	1.15
10^{-2}	5.3	2.5	1.15	0.53
10^{-3}	2.5	1.15	0.53	0.25
10^{-4}	1.15	0.53	0.25	0.115
10^{-5}	0.53	0.25	0.115	0.053
10^{-6}	0.25	0.115	0.053	0.025
10^{-7}	0.115	0.053	0.025	0.011
10^{-8}	0.053	0.025	0.011	0.005
	Massive igneous rocks →			Laminated shales

TABLE 3.XVI - Estimates of fissure width from spacing at various k_r values. (In ATTEWELL and FARMER, 1976).

A similar relationship was derived by SNOW (1968) and adjusted by a factor of 1.25 to allow random orientation of the borehole through a cubic system of fractures, taking the form:

$$\delta^3 = \frac{15\ n\ k_r}{N\ \gamma_w}$$

The above equations are merely theorectical and difficult to apply in practice but sometimes it is possible to achieve reasonable empirical correlations between the joint frequency assessed by rock core inspection and the results of water absorption tests provided the joints exhibit similar physical characteristics especially regarding their width and infilling. Fig. 3.25 represents one of the above refered correlations, dealing with one of the studied projects - the Castelo do Bode tunnel (see Appendix A, vol. II).

Where in-situ stresses cause borehole walls to arch, fissures intercepted by the borehole will close and the permeability of the borehole wall becomes much lower than that of the rock mass.

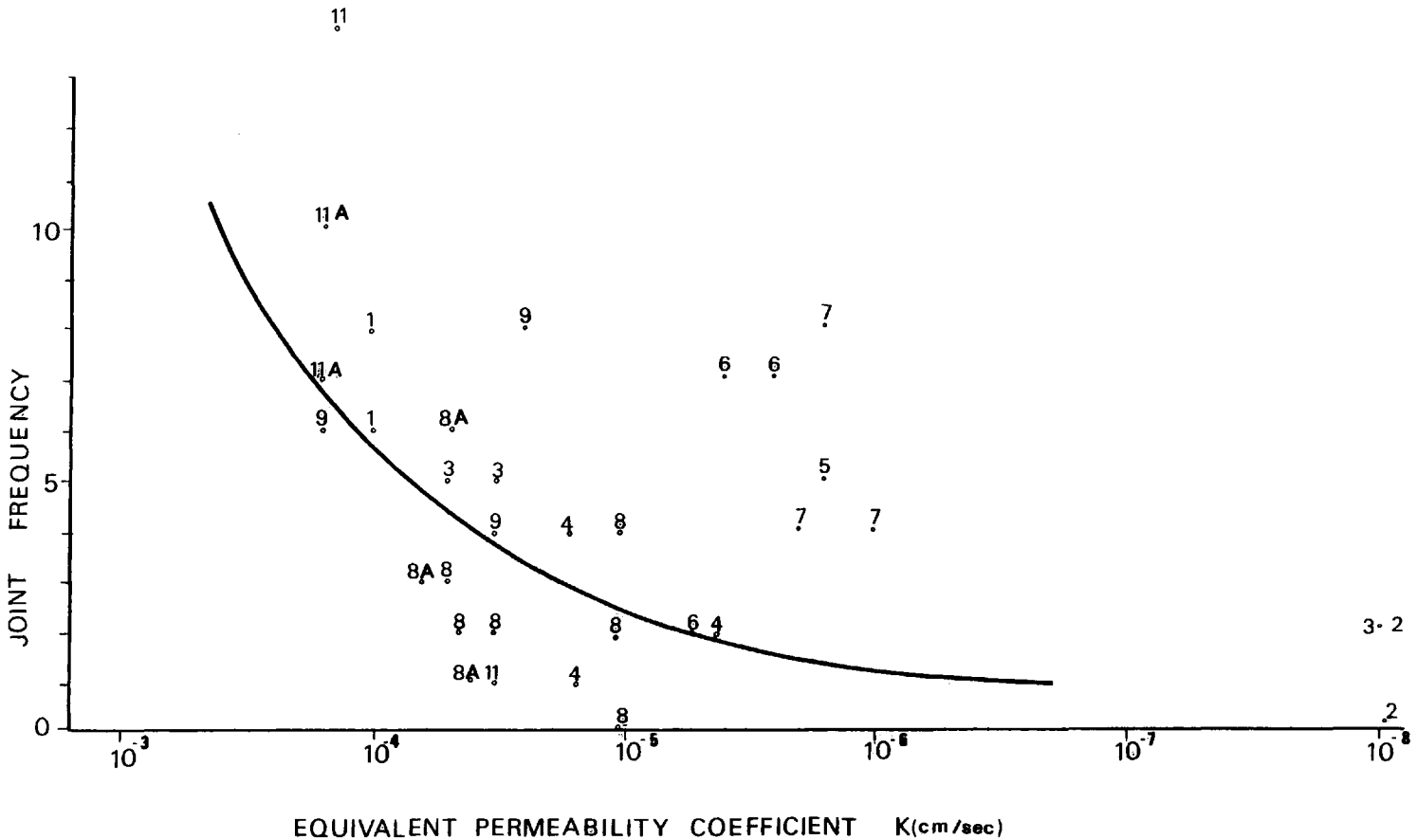


FIG. 3.25 - Correlation between equivalent permeability coefficient and joint frequency at Castelo do Bode tunnel, taken from the geotechnical report for the scheme. Figures refer to borehole designation. NOTE: the solid line has not been constructed using statistics.

Lugeon tests are used to check both the need for and efficiency of ground treatment, not only for dam foundations but also for the rock masses dealing with tunneling and other underground openings, and to assess the conditions to be observed in the treatment. Regarding dam foundations the specification of depth to be reached by the grouting depends on the water absorption coefficients measured and on the hydraulic head to be dissipated or retained. For water absorption coefficient values lower than 1 Lugeon, it is considered that rock mass behaves as impervious and the practice shows that for values lower than that injection of current grouts are not feasible.

3.4.3 - WATER ABSORPTION TESTS: GENERAL DISCUSSION

It is a common practice in site investigations involving rock masses, to perform water absorption tests together with the exploration borehole programmes. It is not usual to drill boreholes only for the purpose of performing these tests; this only makes sense, in general, during the later phases of site investigations, to assess grouting needs.

When establishing borehole programmes and their specifications, the water absorption tests are normally chosen as a compromise between the information needed and the information they can provide. The already mentioned fact that these tests can supply very useful information other than that of rock mass "permeability" (Sec. 3.4.2), should be taken into account, and test specification must be established accordingly.

Regarding dam foundations it is usual, and this is normally the Portuguese practice, to perform Lugeon tests systematically down the boreholes, starting 3 to 5 m beneath ground level. Alternatively, it is also usual to establish a minimum depth to be reached by the tests. After that, the tests will stop when absorptions lower than 1 Lugeon has been obtained for two or three continuous test sections, it being assumed that beneath that depth the rock mass behaves as impervious unless there is reason to believe that this is not the case.

For tunnels or other underground openings the philosophy is not necessarily the same. As it is well known, in those cases, the boreholes sometimes have to be excessively long to reach the depths concerned by the openings. On the other hand, the results of water absorption tests performed either in the excavation zones or in their

proximity often provide information thought satisfactory in relation to that required for design. Performing Lugeon tests along the entire borehole length will lead thus to a lower and sometimes very much lower benefit/cost relationship.

The reason stated above, can result in Lugeon tests being performed along a certain borehole section between points above and below the future excavation. For tunnels, OLIVEIRA (1975a) for example, refers 1.5 to 2.5 the tunnel diameter above and below the tunnel axis. This criterion is, by the same reasons, also used in the assessment of rock mass deformability by dilatometer tests, in integral sampling, etc.

One problem can arise from that practice and it is related to changes in the design when the tests had been finished: the excavation can thus be located in a zone which exhibits different characteristics from those previously studied.

Lugeon tests are executed, generally, in sections of boreholes 3 to 5 m long. These sections can either be between the bottom of the borehole and one packer located above (single packer method), or between two packers (double packer method). The costs and benefits of the test vary with the test method used. When the test section is limited by two packers, the tests can be performed at once after the drilling has been finished, but the problem of leakage through the lower packer can occur. This leakage may not be detected and so lead to serious errors. The execution of the test between a packer and the bottom of the hole, although it can solve the above mentioned problem, implies the performance of the tests as drilling progresses and this involves stoping the drilling to perform each test. In addition, the tests cannot be repeated once the drilling has resumed. The time involved by their performance also increases greatly, in relation to the other method, when an overlap between the successive tests is required. PEARSON and MONEY (1977) refering to this aspect state that "packer" (Lugeon) tests are expensive, particularly when single packer is used, because they interrupt the programme of drilling ..... and result in the drill rig standing idle for at least an hour in the shortest of multi-stage tests".

Although a high degree of precision should not be required for the results in current practice, it is doubtful that this justifies the tests being performed by the drilling crew, as usually happens, instead of by better and specially trained staff, with their inherent increase

in cost. In the first case, the results and some test conditions are written down and both calculations and result interpretation are made later in the office, normally after the equipment has been dismantled. Thus any specification change which could prove useful during the test execution is impossible and any repetition of the tests is costly and not practical, even impossible without new borehole drillings if the tests were carried out using the single packer method.

Under such practice, what can be saved in costs is frequently lost in benefits and most technique improvements do not meet this aims. As stated by PEARSON and MONEY (1977, op. cit.) calculation of results while the test is in progress is the simplest and most cost-effective of improvements in test technique because a test that is discovered to be faulty when it cannot be repeated represents a total waste of time and money.

In the Portuguese practice, as well as in many other countries, Lugeon tests are almost always carried out using a single packer and each test takes about one hour, or up to one and a half hours in special cases: results are usually analysed "off-site". According to the 1980 prices, each test costs the equivalent of drilling one metre of NX borehole, or slightly cheaper.

CHAPTER IV

THE BENEFITS OF SITE INVESTIGATIONS

4.1 - GENERAL

The benefits of site investigation can be regarded in two ways - geotechnical and financial: - they are normally connected in real life. Advances in the understanding of both geological and geotechnical site conditions (= geotechnical benefits) result in improved designs for the conditions encountered and this affects both safety and cost (= financial benefits).

Geotechnical benefits are difficult to assess whereas financial benefits can, in some cases, be quantified because site investigations can be regarded as a capital investment.

It is understandable that people, mainly the owners of projects, who are the investors expect some kind of returns for the money which they invest. For the Funcho - Benaciate tunnel, one of the projects studied (see Appendix A, vol. II), a design was produced in 1977 based on limited information available regarding the geological and geotechnical conditions. Only desk studies and reconnaissance had been undertaken. Later, in 1980, the project was fully redesigned for which a comprehensive new phase of site investigations, using boreholes, geophysical exploration, "in-situ" and laboratory tests, was carried out. This enabled an appropriate geotechnical zoning and characterization of the rock mass to be made and led to a substancial reduction in the lining thickness of the tunnel in relation to the 1977 design, with a significative reduction in the estimated construction costs.

In the case illustrated above, the financial benefits of the second phase of site investigations could be assessed by comparing the cost of the works performed with the money saved due to the reduction in lining thickness. Unfortunately, examples like the one referred to above are unusual and data which allow benefits to be assessed using

that appproach seem to be very scarce. Indeed, assessing the benefits, using this or other quantitative methods, in such a way that a comparison can be made between either the benefits of each type of work performed for a given project or the benefits of the works performed for different projects, does not appear to be feasible. For instance it is not possible to assess individually the benefit of each of the works performed during the second phase of the studies in the tunnel example quoted above.

During site investigations the knowledge of the geological and geotechnical conditions which are relevant for design are obtained step by step by means of the works performed. Each type of work brings, in principle, a given contribution to the "total knowledge" of the conditions on site, where "total knowledge" is taken as that gained at completion of the site investigations. But even this is an approximation because what happens in practice is that the results of each type of work are not interpreted in isolation.

The benefit of each type of work carried out during the studies depends not only upon the type, amount and reliability of the information it provides towards the knowledge of the geological and geotechnical conditions which are relevant for the design but also upon the other work that is performed during the investigation. That benefit constitutes in first instance the return of the capital invested in performing such works. This leads to the conclusion that for a given amount of money invested in site investigations the return which can be obtained depends very much upon the skill and training of the people involved in selecting the investigation methods and locating the works in the field, and interpreting the results. When the results are to be statistically interpreted, a minimum is required for the information to be regarded as reliable. Establishing the amount of works to undertake is still a point where the qualifications of the people responsible for the studies play an important role.

Beyond a given amount of works the additional information which it is possible to obtain by performing further works begins to decrease, also decreasing the return for the respective money invested. This applies either for works of a given type or for the whole site investigations.

The decision as to whether or not more works must be done is

one which should be made by the people involved in the site investigations, in close cooperation with those involved in the design. Nevertheless it must be stated that on several occasions this decision is, unfortunately, made by the owners directly or indirectly by providing insufficient money for adequate investigation. This practice may eventually be responsible for failures which are usually felt most by the owners, who pay for the project and particular note should be made of this fact.

4.1.1 - APPROACH USED

The assessment of both the benefit of each type or work performed and the benefit of the site investigations as a whole for each of the 26 projects studied has been made in a subjective way. The benefit of each type of investigation work is assumed to be the amount of effective knowledge it provides with respect to the geological and geotechnical conditions that are relevant for each design, expressed as a percentage of the complete knowledge which would be desirable at completion of site investigations. Complete knowledge is taken to mean that designs require no further information about the ground. Total knowledge, as defined earlier, is the total information provided by a site investigation regardless of whether it answers all the design questions.

The benefit of a site investigation as a whole is in this thesis expressed as the percentage (total knowledge ÷ complete knowledge).

This is an appropriate moment to point out again that the site investigations dealt with throughout this research are those performed just before construction.

Careful and time consuming research in 26 Portuguese projects for dams and tunnels on rock masses was undertaken in order to gather information about the type and amount of works performed during site investigations for each project. This required a study of the design reports and the reports from the contractors involved. Additional information was obtained either from the consultants or from the owners, or even from individual people involved in the site investigations. A method for collating the data was devised in order to present the information and this forms part of Vol. II.

After the information had been gathered, several meetings were arranged with the individuals involved in the 26 site investigations. They were asked to provide, for each project:

i) marks for the benefit obtained from each type of work performed, in terms of a percentage of the total benefit assumed here as 100% of the site investigations, where total benefit = total knowledge (which is less than complete knowledge).

ii) a mark for the total benefit obtained at completion of the site investigations as a percentage of that for complete knowledge, in their opinion.

Answers were given individually by each assessor.

The meetings referred to above involved a total of 10 assessors, most of them with considerable experience in site investigations. Information about the age and profession status of these assessors is presented in Table 4.I.

ASSESSOR NO.	AGE	PROFESSION
1	< 30	Civil Engineer
2	30-40	Engineering Geologist
3	40-50	Civil Engineer
4	50-60	Civil Engineer
5	30-40	Engineering Geologist
6	> 60	Geologist
7	< 30	Civil Engineer
8	40-50	Engineering Geologist
9	30-40	Engineering Geologist
10	50-60	Civil Engineering

TABLE 4.I - Age and profession status of the assessors.

The number of assessors assigning marks for each project varied between 2 and 4 except in two cases where only one person had been involved in the site investigations.

The marks obtained as described in (i) do not reflect the benefit of each type of work in terms of complete knowledge of the geological and geotechnical conditions relevant to design, as defined in the beginning of this section. Thus, those marks had to be corrected taking into account the total knowledge obtained at completion of site investigations, as explained in (ii). Let us consider a programme of boreholes. If the answer to question (i) was 25% and the answer to question (ii) was 80%, then the benefit of that programme in terms of complete knowledge (geological and geotechnical knowledge gained of the site) will be 25% of 80 .ie. 20%.

The marks obtained for each project as explained in (i) and (ii) are presented in appendix B, vol. II. Within the brackets are the marks corrected as explained above, which have been the ones used in the calculations throughout this work.

4.1.2 - RESULTS

The marks attributed by the assessors to the benefit derived from each type of work for each site investigation, have been separately grouped for concrete arch dams, concrete gravity dams, earth dams and tunnels and are presented in tables 4.II to 4.V. Further, the average of the marks has been assessed and is given in Tables 4.VI (Dams) and 4.VII (tunnels).

A problem arose when preparing these tables: in the case of Fronhas dam, information was obtained from 4 galleries (80 m) and 14 boreholes (280 m) performed in 1938, after which the investigations were stopped until they restarted in 1955. A single mark was given by the assessors for the benefit of both types of works and the author decided to split its value equally between galleries and boreholes.

TYPE OF WORK \ ASSESSOR	ALTO LINDOSO				ALVIDE		ATROZELA		FAGILDE		FOZ-CÔA				FRONHAS*				FUNCHO		TORRÃO			
	3	6	8	10	8	9	8	9		8	3	6	8	10	3	6	8	10	5	8	3	6	8	10
Reconnaissance & gathering existing information	90.0	31.5	8.5	12.8	9.5	4.8	19.0	14.3		20.8	4.8	13.5	4.8	14.3	4.8	21.3	4.5	7.5	4.5	6.7	4.5	13.5	8.0	13.5
Geophysics					4.8	9.5					14.3	18.0	19.0	14.3	8.5	8.5	9.0	7.5			13.5	13.5	16.0	9.0
Trenches	4.5	13.5	4.3	21.3	1.9	1.9	2.9	4.8		4.0	14.3	22.5	14.3	14.3	8.5	8.5	9.0	11.3	10.8	6.7	18.0	22.5	4.0	13.5
Shafts																								
Galleries	22	18.0	21.3	25.5	16.2	17.1					14.3	9.0	14.3	14.3	23.4	14.9	20.3	16.9	27.0	31.4	18.0	13.5	16.0	16.5
Boreholes	40.5	13.5	38.3	17.0	55.1	47.5	73.2	76.0		56.0	23.8	13.5	23.8	23.8	23.4	14.9	29.3	20.7	18.0	23.8	13.5	9.0	20.0	9.0
"In-situ" tests					4.8	9.5					14.3	9.0	14.3	9.5	12.8	12.8	13.5	7.5	18.0	19.0	13.5	13.5	12.0	9.0
Laboratory tests	13.5	13.5	12.8	8.5	2.9	4.8					9.5	4.5	4.8	4.8	4.3	4.3	4.5	3.8	11.7	7.6	9.0	4.5	4.0	4.5
Total knowledge at completion of S.I.	90	90	85	85	95	95	95	95		80	95	90	95	95	85	85	90	75	90	95	90	90	80	75

* Marks related to 4 galleries and 14 boreholes performed 37 years before have been equally split between galleries and boreholes as the marks have not been obtained separately.

TABLE 4.II - Individual marks given to the benefit of each type of work - CONCRETE ARCH DAMS

	JAMBA-IA -MINA		JAMBA-IA -OMA		TOUVEDO				RAIVA			VALEIRA			VALHELHAS			
ASSESSOR / TYPE OF WORK	8	9	5	8	3	4	6	10	6	7	8	6	7	8	3	6	8	10
Reconnaissance & gathering existing inf.	12.0	12.8	8.0	8.5	20.0	24.0	42.5	15.0	34.0	5.0	15.0	4.8	4.0	7.0	12.8	34.0	12.8	17.0
Geophysics	16.0	21.3	16.0	17.0	16.0	16.0	21.3	22.5	12.8	7.5	11.3				17.0	0.0	17.0	17.0
Trenches									8.5	10.0	7.5	9.5	8.0	7.0	8.5	8.5	4.3	12.8
Shafts																		
Galleries												14.3	16.0	17.5				
Boreholes	52.0	51.0	56.0	59.5	44.0	40.0	21.3	37.5	29.8	27.5	41.3	66.5	52.0	38.5	38.3	29.8	42.5	34.0
"In-situ" tests																		
Laboratory tests															8.5	12.8	8.5	4.3
Total knowledge at completion of S.I.	80	85	80	85	80	80	85	75	85	50	75	95	80	70	85	85	85	85

TABLE 4.III - Individual marks given to the benefit of each type of work - CONCRETE GRAVITY DAMS

	AZIBO			CAPINHA		MEIMOA		MORGAVEL		ODELOUCA		SABUGAL	
ASSESSOR / TYPE OF WORK	5	8	9		8	8	9	2	8	5	8	8	9
Reconnaissance & gathering existing information	75.0	16.0	12.8		8.5	7.5	11.3	31.4	24.7	4.5	9.0	8.5	9.0
Geophysics										13.5	13.5	21.3	14.4
Trenches	22.5	20.0	25.5		34.0	18.8	13.5	5.7	6.7	9.0	9.0	8.5	9.0
Shafts										6.3	4.5		
Galleries										25.2	22.5		
Boreholes	45.0	44.0	46.8		42.5	48.8	50.3	58.0	63.7	31.5	31.5	46.8	57.6
"In-situ" tests													
Laboratory tests													
Total knowledge at completion of S.I.	75	80	85		85	75	75	95	95	90	90	85	90

TABLE 4.IV - Individual marks given to the benefit of each type of work - EARTH DAMS

ASSESSOR / TYPE OF WORK	ALTO LINDOSO				CASTELO DO BODE			FUNCHO BENACIATE		SABUGAL-MEIMOA			SADO-MORGAVEL		STA CLARA -MTE DA ROCHA	
	1	6	8	10	5	8	9	5	8	5	8	9	2	8	2	8
Reconnaissance & gathering existing inf.	17.0	31.5	16.0	12.0	4.8	9.0	4.5	9.0	8.5	10.0	10.0	15.0	17.5	13.5	12.0	19.0
Geophysics					9.5	13.5	13.5	9.0	8.5				14.0	18.0	28.0	25.7
Trenches	8.5	22.5	4.0	12.0												
Shafts																
Galleries																
Boreholes	25.5	13.5	40.0	24.0	42.8	42.3	40.5	40.5	42.5	40.0	40.0	35.0	24.5	40.5	24.0	33.3
"In-situ" tests	17.0	13.5	12.0	6.0	19.0	9.0	9.0	22.5	17.0				10.5	13.5	12.0	12.4
Laboratory tests	17.0	9.0	8.0	6.0	19.0	16.2	22.5	9.0	8.5				3.5	4.5	4.0	4.8
Total knowledge at completion of S.I.	85	90	80	60	95	90	90	90	85	50	50	50	70	90	80	95

TABLE 4.V - Individual marks given to the benefit of each type of work - TUNNELS

TYPE OF WORK	CONCRETE ARCH DAMS								CONCRETE GRAVITY DAMS						EARTH DAMS					
	ALTO LINDOSO	ALVIDE	ATROZELA	FAGILDE	FOZ-CÔA	FRONHAS	TUNCHO	TORRÃO	JAMBA IA MINA	JAMBA IA OMA	TOUVEDO	RAIVA	VALEIRA	VALHELHAS	AZIBO	CAPINHA	MEIMOA	MORGAVEL	ODELOUCA	SABUGAL
Reconnaissance & gathering existing information	15.4	7.1	16.6	20.0	9.3	9.4	5.6	9.8	12.4	8.2	25.3	18.0	5.2	19.1	12.1	8.5	9.4	28.0	6.7	8.7
Geophysics		7.1			16.4	8.3		13.0	18.6	16.5	18.9	10.5		12.7					13.5	17.8
Trenches	10.9	1.9	3.8	4.0	16.3	9.3	8.7	14.5				8.6	8.1	8.5	22.6	34.0	16.1	6.2	9.0	8.7
Shafts																			5.4	
Galleries	21.8	16.6			12.9	18.8	29.2	16.0					15.9						23.8	
Boreholes	27.3	51.3	74.6	56.0	21.2	22.0	20 9	12 8	51.5	57.7	35.7	32.8	52.3	36.1	45.2	42.5	49.5	60.8	31.5	52.2
"In-situ" tests		7.1			11.7	11.6	18.5	12.0												
Laboratory tests	12.0	3.8			5.9	4.2	9.6	5.5						8.5						
Total knowledge at completion of S.I.	87.5	95.0	95.0	80.0	93.7	83.7	92.5	83.7	82.5	82.5	80.0	70.0	81.6	85.0	80.0	85.0	75.0	95.0	90.0	87.5

TABLE 4.VI - Averages of the marks given to the benefit of each type of work - DAMS

TYPE OF WORK	ALTO LINDOSO	CASTELO DO BODE	FUNCHO-BENACIATE	SABUGAL--MEIMOA	SADO --MORGAVEL	STA CLARA-MTE DA ROCHA
Reconnaissance & gathering existing inf	19.1	6.1	8.7	11.6	15.5	15.5
Geophysics		12.1	8.7		16.0	26.8
Trenches	11.7					
Shafts						
Galleries						
Boreholes	25.7	41.8	41.5	38.3	32.5	28.6
"In-situ" tests	12·1	12·3	19.7		12 .0	12 .2
Laboratory tests	10.0	19.2	8.7		4.0	4.4
Total knowledge at completion of S.I.	78.7	91.6	87.5	50.0	80.0	87.5

TABLE 4.VII - Averages of the marks given to the benefit of each type of work - TUNNELS

4.2 - DIFFERENCES IN ATTITUDE AMONG THE ASSESSORS

It seems to be interesting and important to compare the attitude or feeling of the assessors in relation to the benefit derived from each type of work performed. Although in some cases the marks they attributed are similar, in other cases they were remarkably different in their judgement. It can also be verified that certain assessors are relatively consistent in giving greater or smaller marks than the other persons involved for particular types of works. This shows a special and personal feeling which is certainly related not only to the information given by each type of work, but also with the assessor's idea about the design needs in terms of that information.

A careful analysis of Tables 4.II to 4.V would be enough to point out the above statements but it seems to be worthwhile to illustrate the problem with some actual figures. For this 4 concrete arch dams were selected with regard to the fact that the same group of 4 assessors (3, 6, 8, 10: Table 4.I) assigned marks; further, the concrete arch dams provide a better comparison as they involve a greater variety in the type of works performed.

The degree of agreement (or disagreement) of the marks given by the 4 assessors is well represented by the statistical parameter "coefficient of variation "(V), expressed as:

$$V\ (\%) = \frac{S}{\overline{X}} \cdot 100$$

where S is the standard deviation and $\overline{X}$ is the mean of the marks.

The mean, the standard deviation and the coefficient of variation of the marks given for the selected dams (Alto Lindoso, Foz-Côa, Fronhas and Torrão) are presented in Table 4.VIII. The last column in this table contains the mean of the parameter V = $(\overline{V})$.

The first conclusion derived for this group of dams is that the 4 assessors only reached a very close agreement in two situations: 1) the benefits from the geophysical exploration and 2) the benefit from the laboratory tests, both dealing with Fronhas dam . This represents only 2 cases out of 26, that is approximately 8% of the situations.

Based upon the empirical scale below:

GOOD AGREEMENT	$V \leq 25\%$
FAIR AGREEMENT	$25\% < V \leq 50\%$
DISAGREEMENT	$V > 50\%$

the following conclusion can be drawn:

1 - A good agreement between the marks given by the 4 assessors as to the benefit of each type of work was reached in 15 cases out of 26, i.e. in about 58% of the cases.

2 - A fair agreement was achieved in 5 of the 26 cases, i.e. in about 19% of the situations, and

3 - Disagreement was found in 6 of the 26 cases, i.e. in about 23% of them.

From a consideration of the results of the last column in Table 4.VIII it can be concluded that, in general, good agreement was reached between the different marks for the benefit of geophysical exploration, galleries and "in-situ" tests: ($\overline{V}$ lower than 25%). For boreholes and trenches ($\overline{V}$ equals 35.3% and 42.3% respectively) and even in the case of laboratory tests (where $\overline{V}$ is slightly greater than 25%), only a fair agreement was achieved. For the benefit from the reconnaissance, fundamental disagreements is evident ($\overline{V}$ = 64.5%).

In the last case, one of the assessors (No. 6) awarded, in general, marks greater and sometimes substantially greater than the others in the group - an average benefit of 20% for the 4 dams studied whilst the average marks given by the other three assessors varied from 5.7% to 12% (see table 4.IX). This trend is also noticed with assessor 10 and is verified in other types of projects as well (see tables 4.III - concrete gravity dams and 4.V - tunnels).

As far as the trenches are concerned it is assessor 8 who shows the tendency to assign lower marks, this being also observed with projects other than the 4 selected (e.g. Valhelhas dam, Alto Lindoso tunnel). High marks came from assessor 6 who, sometimes, gave higher marks to the benefit from trenches than to that from boreholes, and in some situations, even higher than the benefit from galleries (Foz-Côa and Torrão dams sites and also Alto Lindoso tunnel). Assessor 10 also awarded higher marks to the benefits from trenches than to those from boreholes in 2 of the 4 selected dams, but never greater than the benefits assigned to galleries.

TYPE OF WORK	ALTO LINDOSO DAM			FOZ-CÔA DAM			FRONHAS DAM			TORRÃO DAM			MEAN OF THE COEFFICIENTS OF VARIATION $\bar{V}$ (%)
	MEAN	STANDARD DEVIATION	COEFFICIENT OF VARIATION V(%)	MEAN	STANDARD DEVIATION	COEFFICIENT OF VARIATION V (%)	MEAN	STANDARD DEVIATION	COEFFICIENT OF VARIATION V (%)	MEAN	STANDARD DEVIATION	COEFFICIENT OF VARIATION V (%)	
Reconnaissance & gathering existing inf.	15.4	10.9	70	9.3	5.3	57	9.4	8.1	86	9.8	4.4	45	64.5
Geophysics				16.4	2.5	15	8.3	0.6	8	13.0	2.9	22	15.0
Trenches	10.9	8.2	75	16.3	4.1	25	9.3	1.3	14	14.5	7.9	55	42.3
Galleries	21.8	3.1	14	12.9	2.7	20	18.8	3.8	20	16.0	1.9	12	16.5
Boreholes	27.3	14.0	51	21.2	5.2	24	22.0	6.0	27	12.8	5.2	40	35.5
"In-situ" tests				11.7	2.9	25	11.6	2.8	24	12.0	2.1	18	22.3
Laboratory tests	12.0	2.4	20	5.9	2.4	40	4.2	0.3	7	5.5	2.3	43	27.5

TABLE 4.VIII - Statistics on the marks given by the same group of 4 assessors, for the benefits of the works performed for 4 concrete arch dams

	ALTO LINDOSO, FOZ-CÔA, FRONHAS AND TORRÃO DAM SITES			
TYPE OF WORK	3	6	8	10
Reconnaissance & gathering exist. inf.	5.7	20.0	6.5	12.0
Geophysics	12.1	13.3	14.7	10.3
Trenches	11.3	16.8	7.9	15.1
Galleries	20.0	13.9	18.0	18.3
Boreholes	25.3	12.7	27.9	17.6
"In-situ" tests	13.5	11.8	13.3	8.7
Laboratory tests	9.1	6.7	6.5	5.4
Total knowledge at completion of S.I.	90.0	88.8	87.5	82.5

TABLE 4.IX - Average marks for the benefit from each work, given by 4 assessors, for a group of four dams

In relation to mechanical characterization tests, there was a tendency for assessor 3 to give the highest marks, and for assessor 10 to award the lowest ones.

According to the results, it seems that a certain affinity in judgement exists between assessors 6 and 10 on the one hand and between assessors 3 and 8 on the other.

For tunnels it was not possible to find so large a number of assessors who were connected with the same site investigations. Nevertheless, it was possible, for example, to group two pairs of tunnels in which the same assessors were involved. One of these pairs are the Castelo do Bode and the Funcho-Benaciate tunnels with marks given by assessors 5 and 8, and the other Sado-Morgavel and Sta. Clara - Mte. da Rocha tunnels to which marks were assigned by assessors 2 and 8. As only two assessors were involved in each site investigation, judgement of their attitudes is achieved by comparing the individual marks: Table 4.V. A different attitude can be noticed from the assessors involved in the studies for the Castelo do Bode and Funcho-Benaciate tunnels as revealed by the marks given to the benefits from "in-situ" tests. The two assessors for the Sado-Morgavel and Sta. Clara - Mte. da Rocha tunnels, reveal differences in respect of the marks dealing with both reconnaissance and boreholes.

Such differences in attitude among the assessors towards the benefits of site investigations, are being revealed in the relaxed atmosphere of their offices, but these differences would be probably greater if expressed under pressure. This is an important point to consider in cases like consultation and arbitration and is the inevitable consequence of being unable to define benefit in simple financial terms. Benefit is no longer a simple financial concept as soon as it is taken from a strictly financial setting. Hence, benefit becomes a subjective appraisal and as such it is unrelaistic to imagine that it can be defined in non-subjective terms and thus become a point of agreement. The benefits of site investigation will therefore always be a matter of discussion.

4.3 DIFFERENCES IN USE OF TECHNIQUES

One other important finding which emerged from this study deals with the frequency of utilization of each type of work in the

site investigations for the dams and the tunnels. This gives a reasonable close picture of the Portuguese practice in this respect as these projects cover nearly all the major projects studied during the last ten years.

4.3.1 DAMS

With regard to the 20 dam projects it was found that two types of works were always performed whatever the type of dam. These were reconnaissance and gathering existing information, and borehole drilling. Nevertheless it must be mentioned that they have never been the only works carried out.

The degree of utilization of each type of investigation work for each type of dam is presented in Table 4.X.

The use of geophysical exploration varied little with each type of dam. Galleries were almost exclusively utilized for concrete arch dam sites where they were used in 75% of the cases. For the other types of dams studied, galleries were performed only in 2 cases out of 12 and in one of these cases (an earth dam), the single gallery was excavated before the type of dam was selected. This was also the only dam site where shafts were used.

As for trenches, they were less used for studying gravity dam sites (employed in 67% of the cases) than for arch and earth dams (employed in 100% and 83% of the cases, respectively). These conclusions are summarized in Table 4.XI, where the degrees of use of each type of work are compared through an empirical scale established as follows:

DEGREE OF UTILIZATION	FREQUENCY OF UTILIZATION
Seldom or not used	Used in less than 30% of the cases
Moderately used	Used in between 30-70% of the cases
Highly or systematically used	Used in more than 70% of the cases

TYPE OF WORK	CONCRETE ARCH DAMS - TOTAL 8		CONCRETE GRAVITY DAMS - TOTAL 6		EARTH DAMS - TOTAL 6	
	NO. OF CASES	%	NO. OF CASES	%	NO. OF CASES	%
Reconnaissance & gathering existing inf.	8	100	6	100	6	100
Geophysics	4	50	4	67	3	50
Trenches	8	100	4	67	5	83
Shafts	-	-	-	-	1	17
Galleries	6	75	1	17	1	17
Boreholes	8	100	6	100	6	100
"In-situ" tests	5	40	-	-	-	-
Laboratory tests	6	75	1	17	-	-

TABLE 4.X - Frequency of utilization of each type of work for dam site investigations

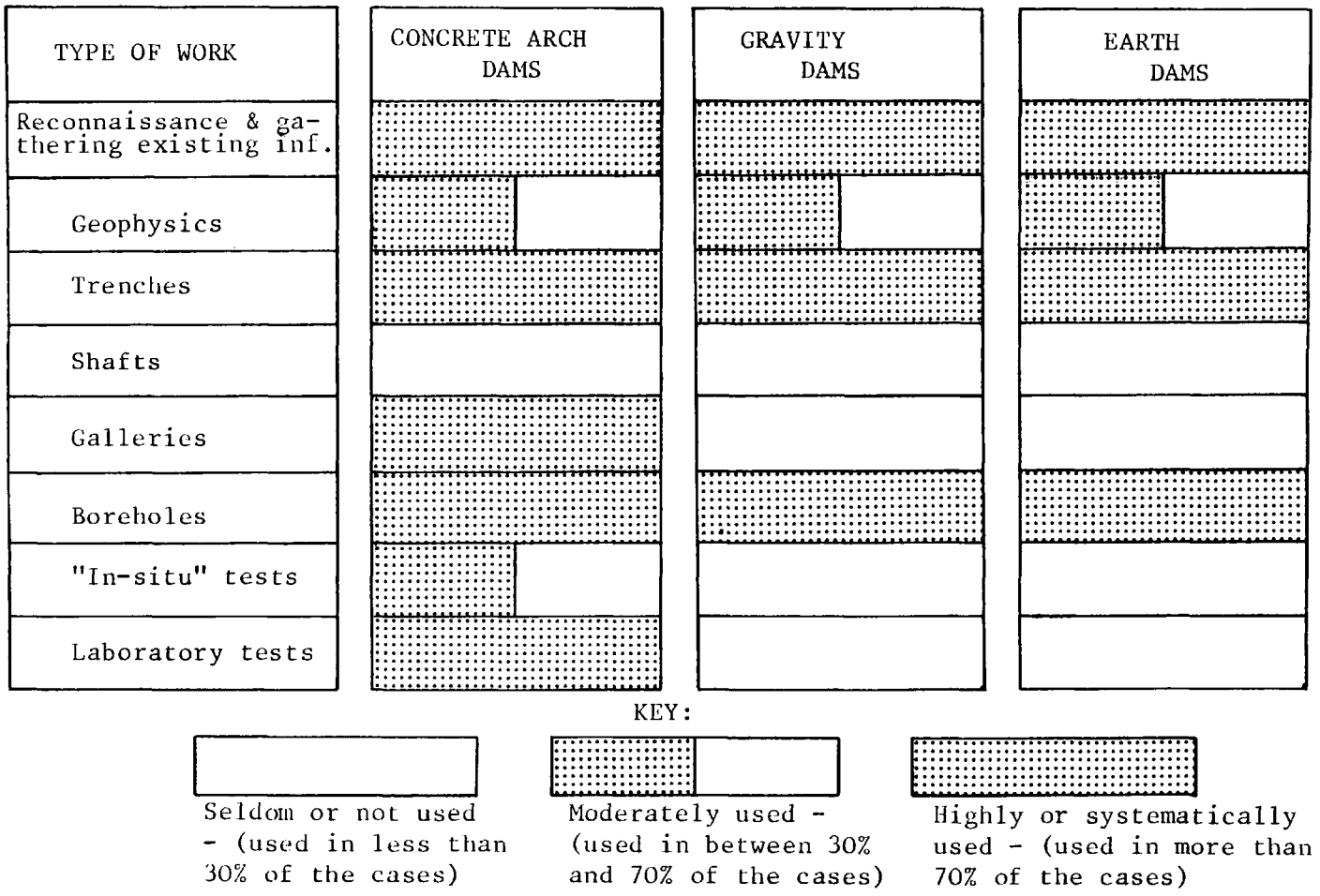


TABLE 4.X1 - Degree of utilization of each type of work in site investigations for dams

In preparing Table 4.XI, the author did not strictly follow the results for the utilization of trenches for the gravity dams studied. In this case the figure obtained (67% - see table 4,XI) is very close to the "highly or systematically used" degree established above which was adopted.

4.3.2 - TUNNELS

For the six projects studied (see table 4.XII) it can be verified that, as for the dams, reconnaissance and boreholes were the type of works always used in site investigations. Shafts and galleries have never been used and trenches were only utilized in one of the six projects. Geophysical exploration was used with similar frequency in site investigations for tunnels as it was for dams. "In-situ" and laboratory tests were both used for 83% of the sites. They were only omitted in one of the projects studied.

The frequency of utilization of "in-situ" tests is much higher than that in the case of concrete arch dams if all the dams of this type studied are accounted for. However, if only the higher dams (height > 40 m) are considered, the "in-situ" tests were used with equal frequency as for tunnels.

A comparison of the degree of utilisation of each type of work in the site investigations for 6 tunnels, using the previous scale, is presented in table 4.XIII.

4.4 - PARTICULAR BENEFITS FROM EACH TYPE OF WORK

4.4.1 - DAMS

Statistics on the benefits from each type of work in the site investigations for the 20 dams studied is presented in Table 4.XIV. The calculations, based upon the data in Table 4.VI deal with the benefits gained through the performance of each type of work, as defined in section 4.1.1.

The values for shafts, and those for galleries for gravity dams are meaningless due to the fact that they were very rarely utilized. The same applies to laboratory tests in relation to gravity dams.

The greatest part of the knowledge obtained, as can be seen from the above table, was from performing boreholes. This happened

TYPE OF WORK	NO. OF CASES OUT OF 6	%
Reconnaissance & gaterhing existing inf	6	100
Geophysics	4	67
Trenches	1	17
Shafts	-	-
Galleries	-	-
Boreholes	6	100
"In-situ" tests	5	83
Laboratory tests	5	83

TABLE 4.XII - Frequency of utilization of each type of work for tunnel site investigations.

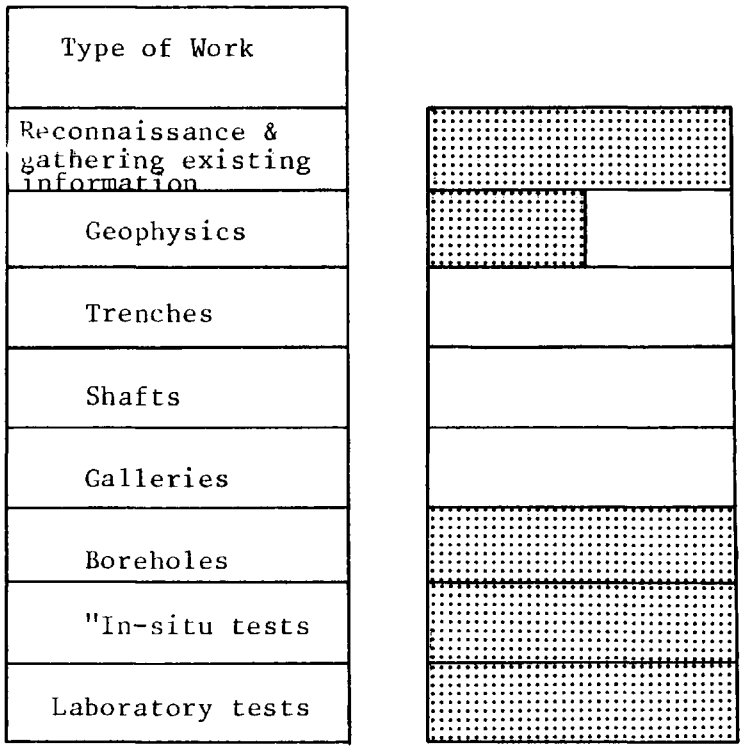


TABLE 4. XIII - Degree of utilization of each types of work on site investigations for trenches

TYPE OF WORK	CONCRETE ARCH DAMS NO. OF PROJECTS - 8				CONCRETE GRAVITY DAMS NO. OF PROJECTS - 6				EARTH DAMS NO. OF PROJECTS - 6			
	RANGE (%)	AVERAGE (%)	COEFFICIENT OF VARIATION V (%)	NO OF PROJECTS WHERE W/K HAS BEEN USED	RANGE (%)	AVERAGE (%)	COEFFICIENT OF VARIATION V (%)	NO OF PROJECTS WHERE W/K HAS BEEN USED	RANGE (%)	AVERAGE (%)	COEFFICIENT OF VARIATION V (%)	NO OF PROJECTS WHERE W/K HAS BEEN USED
Reconnaissance & gathering existing inf.	5.6 - 20.0	11.7	43	8	5.2 - 25.3	14.7	51	6	6.7 - 28.0	12.2	65	6
Geophysics	8.3 - 16.4	11.2	38	4	10.5 - 18.9	15.4	24	5	13.5 - 17.8	15.7	-	2
Trenches	1.9 - 16.3	8.7	60	8	8.1 - 8.6	8.4	3	3	6.2 - 34.0	16.1	66	6
Shafts	-	-	-	-	-	-	-	-	-	5.4	-	1
Galleries	12.9 - 29.2	19.2	30	6	-	15.9	-	1	-	23.8	-	1
Boreholes	12.8 - 74.6	35.8	61	8	32.8 - 57.7	44.4	24	6	31.5 - 60.8	47.0	21	6
"In-situ" tests	7.1 - 18.5	12.2	33	5	-	-	-	-	-	-	-	-
Laboratory tests	3.8 - 12.0	6.8	48	6	-	8.5	-	1	-	-	-	-

TABLE 4.XIV - Statistics on the benefits from each type of work - DAMS

in the three types of dams and for the gravity and the earth dams studied, about 50% of total knowledge obtained during site investigations was achieved by this type of work. This figure is lower in the case of concrete arch dams and can be explained by the fact that for these structures important information required for the design can only be achieved by performing galleries and "in-situ" tests. This does not apply to the case of small arch dams where large amounts of the information can be obtained through boreholes.

In the projects dealing with the same type of dams, the variability of benefit derived from boreholes, as given by V (coefficient of variation) is small for both the gravity and earth dams studied in comparison to arch dams: (V equals to 24%, 21% and 61% respectively). The higher variation for arch dams is because the projects studied include dams with heights from 23.75 m to 145 m. The greater benefits from boreholes coincide with the smaller dams. Thus, it is possible to conclude that the amount of knowledge obtained from boreholes in relation to the design needs, decreased with an increase in the dam height. This is supported by the fact that if the dams studied are divided into two groups, one comprising the lower ones and the other the higher ones with heights above 40 m, the average marks for the benefit from boreholes are 60.6% and 20.8% respectively.

It is interesting to point out that not only the average marks but also the marks for the minimum and maximum benefit from boreholes were similar in the case of gravity and earth dams. The eight of these two types of dams varied from 23.6 m to 60 m (gravity) and from 18 m to 80 m (earth).

After boreholes, the highest benefits of site investigations for concrete arch dams came through galleries (19.2% as an average). When considering the group of arch dams higher than 40 m, the average benefit from galleries was very similar to the figure derived from boreholes (19.7% and 20.8% respectively). In the particular case of Funcho and Torrão dams, meanwhile, the galleries gave more information than the boreholes (29.2% as against 20.9% in the first case and 16% as against 12.8% in the second).

Average benefits of 12.2% and 6.8% were given for performing "in-situ" and laboratory tests, respectively, for arch dams. In the case of the group of the higher dams, both types of tests were performed for each site except for Alto Lindoso dam where "in-situ"

tests were not carried out. Although this is a 110 m high dam this was justified by the fact that results were extrapolated from a neighbouring site with the same geology (rock mass) where the dam was previously intended to be located. Even for the smaller dams studied, both "in-situ" and laboratory tests were carried out for the 25 m high Alvide dam and the results were afterwards extrapolated to the nearby Atrozela dam site which is located in similar geological formations. No mechanical characterization tests were performed for the 27 m high Fagilde dam. This, possibly explains, at least in part, why the mark for the total knowledge obtained for the site conditions at completion of site investigations is the lowest value of all the concrete arch dams studied (see Table 4.VI).

Neither "in-situ" nor laboratory tests were performed for the gravity and the earth dams studied. However, Valhelhas dam site was an exception. Although it can be accepted that for earth dams on rock masses, the mechanical characterization of the geological formations is generally not required, this is not so in the case of concrete gravity dams. Comparing the total knowledge of the site conditions gained at completion of the site investigations (Table 4.VI) one finds that an average of about 90% for the arch dams and about 80% for the gravity dams were achieved. The absence of mechanical characterization may have been one of the factors responsible for such a low mark.

The benefits from geophysical exploration were, on average, slightly lower in the case of arch dams than in the case of the other two types, however for earth dams it must be said that this type of work was only performed at two sites. The reason for those lower marks, may be the same as that explained for the case of the lower marks from boreholes for arch dams.

The benefit obtained by excavating trenches was on average, higher for earth dams (16.1%) than for both arch and gravity dams (8.7% and 8.4% respectively). The variability in benefit is very considerable with V equaling 60% and 66% for arch and earth dams respectively. This is understood to be due to the particular characteristics of this technique. The knowledge which it is possible to obtain from excavating trenches depends to a large extent upon the conditions of the rock mass near the surface, and upon the vegetation and topography of the site. The minimal variation in the marks assigned for the benefit from excavating trenches, in the case of the gravity dams studied, may reflect the fact that the sites where this technique was used deal with similar rock

masses.

The benefit from "reconnaissance and gathering existing information" was, on average, similar for the three types of dams. As would be expected, the variaiblity of the marks is large in all cases, because the amount of knowledge which it is possible to obtain through this type of investigation work depends upon the amount and quality of available information and also upon the local conditions (rock type, soil cover, density of vegetation, type of topographic maps available, etc.). Even so, it is interesting to see that the minimum benefit from reconnaissance for the three different types of dams only varied between 5.2% to 6.7%.

The author is aware of the reasons for some of the higher marks assigned and some of them will be quoted as examples.

In the case of Alto Lindoso dam site there was, at the beginning of the studies, very useful information derived from previous investigations undertaken for a 650 m downstream river section, on the same rock mass, where the dam was to be constructed. This explains the assigned mark of 15.4%.

Alvide and Atrozela dam sites are very close to each other and are on similar geological formations. When the studies on Atrozela began, advantage was taken of the information already gathered on Alvide a couple of years before. This explains the different marks of 7.1% for Alvide and 16.6% for Atrozela.

When the actual Touvedo dam site was suggested, four other sites were considered as well and a preliminary assessment of the geological conditions was undertaken for each site by means of a geological mapping and in some cases with a limited number of trenches and boreholes. As all these sites are on the same geological formation the accumulated knowledge gathered through those works explains the high mark of 25% given to the benefit from "reconnaissance and gathering existing information" for Touvedo.

As a similar example for the earth dams, the case of Morgavel dam should be quoted. This dam is located in a zone (Sines) where comprehensive site investigations had already been performed for several other projects dealing with the Sines industrial complex - tunnel, harbour, quarries, etc. Furthermore, preliminary investigations had been performed for a nearby alternative dam site by means of field mapping and some borehole drilling. The large amount of information

available when the actual Morgavel dam site began to be studied supports the high mark (28%) assigned to the benefit from "reconnaissance and gathering existing information".

4.4.2 - TUNNELS

According to the results in Table 4.XV, the major part of the information about the geological and geotechnical conditions of the rock masses to be tunnelled was also gained from boreholes: 34.7% on the average.

The average value for the benefits from geophysical exploration (15.9%) is affected by the marks given to the works of this type performed for the Sta Clara - Mte da Rocha tunnel. Here, very comprehensive and useful geophysical exploration works were performed in three programmes which were assessed to give a total knowledge of 26.8% (see Table 4.VII). For all the other tunnels studied, the minimum, maximum and average marks given for the benefits from geophysical exploration were 8.7%, 16% and 12.3% respectively.

The average value of 9.3% obtained for the benefit from laboratory tests in the case of site investigations for tunnels is high, this being influenced by the marks assigned to the benefit from these works for Castelo do Bode tunnel (19.2%), in which two comprehensive programmes of tests were performed.

4.5 - BENEFITS OF THE SITE INVESTIGATIONS OVERALL

The assessors involved in the studies for each of the 26 projects discussed, were asked about the level of information regarding the geological and geotechnical conditions achieved at the end of the site investigations. The answers were given in terms of percentage of knowledge in relation to that which they considered as desirable for design: i.e. total knowledge. The range and average answer for each type of project are presented in Table 4.XVI and were assessed from the data on Tables 4.VI and 4.VII.

For dams, the greatest knowledge of the site conditions was achieved from site investigations for concrete arch dams. Thus, it is appropriate to conclude that there is a trend towards more detailed studies for concrete arch dams than for the other types of dam. On the other hand it seems that the site investigations performed for

TYPE OF WORK	RANGE (%)	AVERAGE (%)	COEFFICIENT OF VARIATION (%)	NO OF PROJECTS WHERE WORK HAS BEEN USED
Reconnaissance & gathering existing inform.	6.1 - - 19.1	12.8	38	6
Geophysics	8.7- - 26.8	15.9	49	4
Trenches	-	11.7	-	1
Shafts	-	-	-	-
Galleries	-	-	-	-
Boreholes	25.7 - - 41.8	34.7	20	6
"In-situ" tests	12.0- - 19.7	13.7	25	5
Laboratory tests	4.0 - - 19.2	9.3	66	5

TABLE 4.XV Statistics on the benefit from each type of work - TUNNELS

	Concrete Arch Dams (8 projects)		Concrete Gravity Dams (6 projects)		Earth Dams (6 projects)		Tunnels (6 projects)	
	RANGE	AVER.	RANGE	AVER.	RANGE	AVER.	RANGE	AVER.
% of knowledge at the completion of S.I. in relation to that considered desirable for design (100%)	80.0 to 95.0	88.9	70.0 to 85.0	80.3	75.0 to 95.0	85.4	50.0 to 91.6	79.2

TABLE 4.XVI - Benefits of the site investigations overall

gravity dams were, in general, limited.

It would be expected that the knowledge of the ground conditions gathered through the site investigations for tunnels was poorer when compared with that achieved for dam sites. This situation is regarded as normal in Portugal. The detail of final design of tunnels is made during construction taking into account the additional knowledge provided by excavation, mainly in the first stage. The requirement of site investigation for tunnels in Portugal is that they provide designers with information about the various options that are appropriate for the ground that is to be expected. The choice of options (e.g. lining thickness) is normally decided once construction commences.

The poorer knowledge of the ground conditions is due to the inherent difficulty of gaining information from investigations which usually involve long lengths and great depths leading, normally, to great expenditures in order to obtain high levels of information.

Of the six tunnels, three of them show deficiency, when compared to the others, in the level of knowledge reached at completion of the site investigations. They are the Sabugal - Meimoa tunnel, Alto Lindoso tunnel and Sado-Morgavel tunnel. For these, the total knowledge of the rock mass conditions at the end of the studies was 50%, 78.7% and 80% respectively.

Different attitudes from the people involved, with respect of the knowledge gained at the end of the studies in relation to that considered as desirable for each project, can also be noted as in the case of the benefit from each type of work (see section 4.2). A quick survey of tables 4.II to 4.V brings this to light.

For example assessor 10 tends to be more conservative than the others involved in the same site investigations. The same trend is also noted for assessor 2 in relation to assessor 8 for the studies in which both were involved.

In spite of the low number of cases studied, table 4.XVI leads to some conclusions. Site investigations for concrete arch dams seem to be close to becoming as good as they are likely to be, in practice. It can be said that the same applies for earth dams.

The level of knowledge of the ground at completion of site investigations for tunnels also seems to be good under the particular

approach used in the site investigations for these structures: the values in Table 4.XVI are significantly influenced by the very low level of knowledge reached in the case of Sabugal-Meimoa tunnel (only 50%). If this tunnel is not accounted for, the range and the average would be 78.7% - 91.6% and 85% respectively.

As regards the concrete gravity dams it seems that site investigations could be improved. This would not require even unusual expenditures as proved by the fact that for the gravity dams the cost of the site investigations/cost of construction relationship was found to be, in average terms, the lowest of the four different types of structures studied (see 5.3). Perhaps this is the explanation: i.e. the site investigations for gravity dams studied were an "inexpensive" exercise. This can result in the designers for gravity dams using a level of geotechnical information that is less than that used by designers of arch dams, earth dams and tunnels, in that regards the design needs.

The types of works used in site investigations and the extent to which each technique is used will obviously be reflected in the cost of the studies. This aspect of the problem will be considered in detail for each of the twenty six projects studied (Chapter V).

CHAPTER V

THE COSTS OF SITE INVESTIGATIONS

5.1 - GENERAL

The costs of site investigations for each of the 26 projects studied are presented in Appendices A and C - vol. II. There, they are quoted in detail with account taken of the time when they were performed.

In specific cases where the same type of work was performed in more than one programme (e.g. boreholes, geophysical exploration), quotations are separately provided.

The way in which the costs were obtained is also explained in Vol. II (introduction).

Any comparison between the costs of the performed works, either within one individual project or between different projects, cannot be done without reducing them to one common basis. Such comparison would not be reliable due to price differences created by inflation over a number of years. Curve 2 in Fig. 5.1 is an example of this variation in cost. It represents the costs assessed for a medium size borehole programme, in Portugal, between 1970 and 1980, based upon prices obtained through a Portuguese contractor (Tecnasol Ltd.).

In order to achieve a common standard for evaluation, it was decided to convert all the costs to 1980 prices, details of which are also presented in appendix C - Vol. II. To achieve this, the Bank of Portugal's price index represented by curve 1 in Fig. 5.1 was adopted. The costs so obtained are the ones which will be quoted or used in any calculations throughout the text, except where otherwise stated.

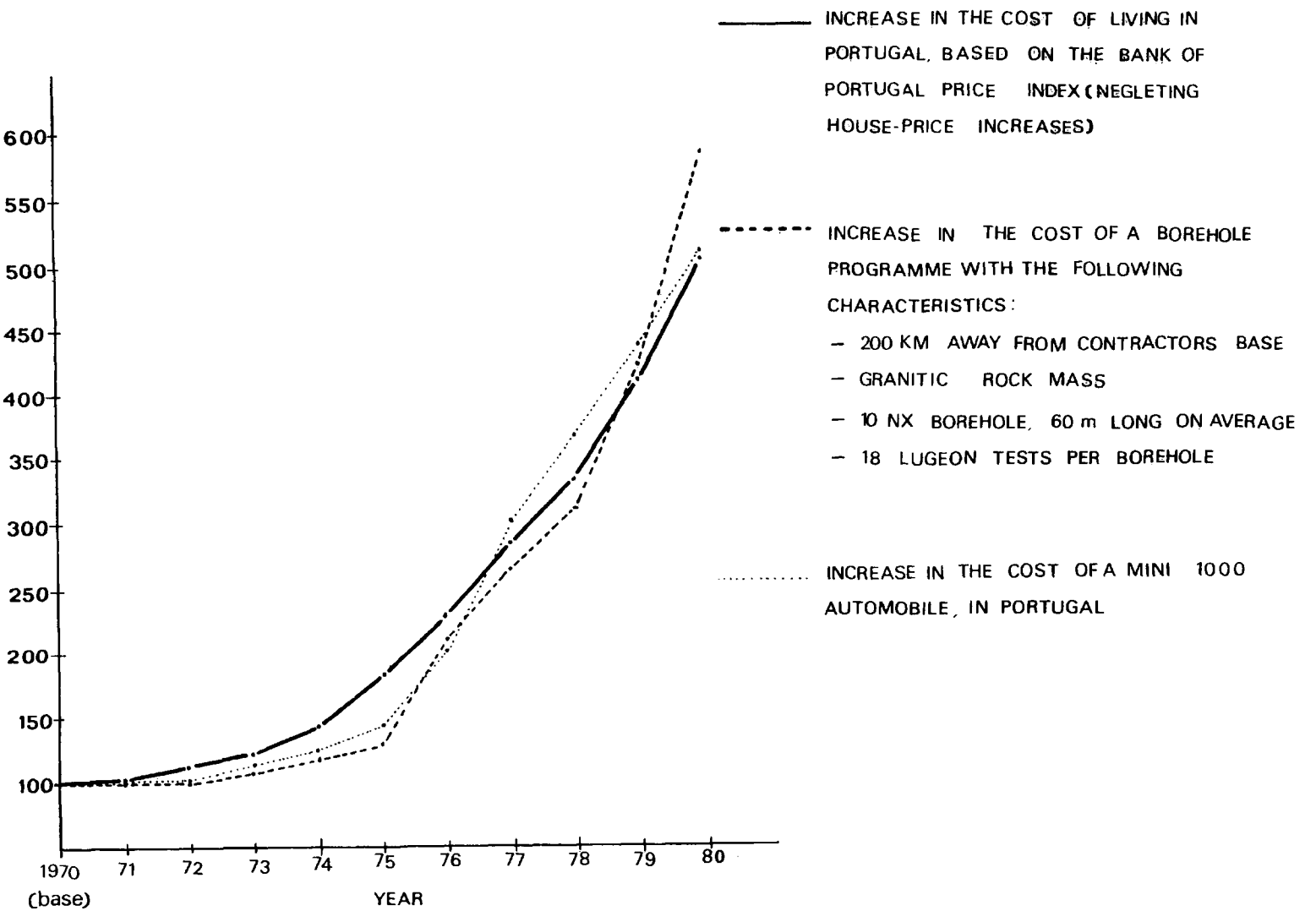


Fig. 5.1 Cost increase comparisons

By comparing curves 1 and 2 in figure 5.1 it can be concluded that between 1970 and 1980 the evolution in the costs of borehole programmes followed the inflation rate, in Portugal. The author also wanted to compare the increase in costs of the exploration works quoted above with that of a popular item in the Portuguese industrial market. For this, a mini 1000 car was selected. This choice seemed to be a good one because it is assembled in Portugal and the components are partially imported and partially made there. Curve 3 in Fig. 5.1 enables a comparison to be drawn between its cost evolution and that of the two already mentioned cases, and it was based on data provided by British Leyland de Portugal - Automoveis, Limitada.

Only the evolution in costs of the borehole programmes is accounted for in Fig. 5.1, although in the author's opinion these curves can also give a reasonable picture of the evolution of prices in the other types of works used in site investigations, over the last decade. This contradicts the opinion that site investigations are becoming more and more costly as can be heard some times. Their costs tend to follow the general trend in cost increases and so this justified that the Bank of Portugal's price index is used for the purpose explained above.

The reduction of costs to a common basis is also necessary if an assessment of the relationship between the cost of the site investigations and the cost of the construction is required. For this, it is also a mistake to use the costs of the works carried out during the studies at the time they were performed, using the total for comparison with the cost of the construction which, normally, is calculated just before the release of the design. This would give a lower relationship and should be avoided. An accurate assessment could be achieved either by converting the costs of the investigations to the time when the budget was assessed or by reducing all costs to other common basis as previously illustrated.

5.2 - THE COSTS OF THE SITE INVESTIGATIONS FOR THE PROJECTS STUDIED

5.2.1 - THE COSTS OF THE ENGINEERING GEOLOGICAL STAFF

First of all it is necessary to explain what is intended by the costs of the engineering geological staff. This is the money spent in payment of the geological (geotechnical) staff involved in the site

investigations, who either form part of the owner's staff and produce their own designs or belong to the consultant's staff. In some special cases both owner's and consultant's engineering geological staff can be involved.

The task of the engineering geological staff is to gather relevant information already available and also to undertake the reconnaissance; to plan the programmes for subsequent exploration and testing; to follow the progress and closely supervise the execution of those works which, in general, are contracted out; to suggest alterations or modifications to the established programmes when advisable, and to assess the information and produce appropriate geotechnical assessment reports to assist in design.

The costs of the geological staff who work for the contractors form part of the cost of the works contracted, and thus are not accounted for in the engineering geological staff costs previously mentioned.

The costs of the engineering geological staff have been assessed for the 26 projects studied and are presented in Tables 5.I to 5.IV, arranged according to the types of projects included. In the two cases (Raiva dam and Valhelhas dam) where a special joint assessment was contracted, costs have been accounted for as costs of the engineering geological staff. The tables also show some interesting details and useful comparisons can be drawn here.

The details of the staff costs are approximate and were assessed with the assistance of the people involved in the relevant site investigations. In quoting these costs it is only intended to provide a relative picture of the cost distribution in Portugal.

The average values related to both field studies and desk studies for each project type were plotted in Fig. 5.2. They show that within the projects studied the concrete gravity dams present the greatest cost with regard the field works, in relation to the total of the engineering geological staff costs. On the other hand, the tunnels and the earth dams are relatively close to each other with the distribution of the engineering geological staff costs and the costs related to field studies exceeding, in general terms, only slightly the costs for desk studies. A fifth point was plotted in Fig. 5.2, still dealing with the concrete arch dams, which shows increase in field work as the height of the dam increases. The general site invest-

PROJECT	STAFF COSTS				TOTAL COST OF S.I. ($\times 10^3$ ESC) D	STAFF COSTS/ COST OF S.I. A/D (%)	TOTAL COST OF CONSTRUCTION ($\times 10^6$ ESC.) E	STAFF COSTS/ COST OF CONSTRUCTION A/ E (%)	COSTS OF S.I./ COST OF CONSTRUCTION D/E (%)
	TOTAL ($\times 10^3$ ESC.) A = B + C	DESK STUDIES AND REPORT PREPARATION B (% OF A)	FIELD STUDIES C (% OF A)	FOR GATHERING EXISTING DATA AND RECONNAISSANCE (% OF A)					
ALTO LINDOSO	700	25%	75%	13%	9370	7.5%	2420	0.03%	0.4%
ALVIDE	450	40%	60%	15%	4000	11.3%	45	1.00%	8.9%
ATROZELA	440	44%	56%	12%	2180	20.2%	50	0.88%	4.4%
FAGILDE	50	50%	50%	25%	468	10.7%	50	0.10%	0.9%
FOZ-CÔA	1000	25%	75%	10%	31250	3.2%	6000	0.02%	0.5%
FRONHAS	900	25%	75%	20%	14570	6.2%	1130	0.08%	1.3%
FUNCHO	750	65%	35%	8%	7860	9.5%	240	0.31%	3.3%
TORRÃO	1500	30%	70%	24%	26410	5.7%	2000	0.08%	1.3%

TABLE 5.I - Engineering geological staff costs: details and comparisons - CONCRETE ARCH DAMS

PROJECT	STAFF COSTS				TOTAL COST OF S.I. ($\times 10^3$ ESC.) D	STAFF COSTS/ COST OF S.I. A/D (%)	TOTAL COST OF CONSTRUCTION ($\times 10^6$ ESC.) E	STAFF COSTS/C COST OF CONSTRUCTION A/E (%)	COST OF S.I./ COST OF CONSTRUCTION D/E (%)
	TOTAL ($\times 10^3$ ESC) A = B + C	DESK STUDIES AND REPORT PREPARATION (% OF A)	FIELD STUDIES C (% OF A)	FOR GATHERING EXISTING DATA AND RECONNAISSANCE (% OF A)					
JAMBA-IA-MINA	930	38%	62%	21%	7480	12.4%	960	0.10%	0.8%
JAMBA-IA-OMA	950	52%	48%	5%	6570	14.5%	790	0.12%	0.8%
TOUVEDO	500	26%	74%	26%	3440	14.5%	710	0.07%	0.5%
RAIVA	680	26%	74%	11%	2510	27.1%	640	0.10%	0.4%
VALEIRA	1400	28%	72%	10%	9800	14.3%	2800	0.05%	0.4%
VALHELHAS	1480	24%	76%	12%	5990	24.7%	1150	0.13%	0.5%

TABLE 5.II - Engineering geological staff costs: details and comparisons - CONCRETE GRAVITY DAMS

PROJECT	STAFF COSTS				TOTAL COST OF S.I. ($\times 10^3$ ESC) D	STAFF COSTS/ COST OF S.I. A/D (%)	TOTAL COST OF CONSTRUCTION ($\times 10^6$ ESC.) E	STAFF COSTS/ COST OF CONSTRUCTION A/E (%)	COST OF S.I./ COST OF CONSTRUCTION D/E (%)
	TOTAL ($\times 10^3$ ESC.) A = B + C	DESK STUDIES AND REPORT PREPARATION B (% OF A)	FIELD STUDIES C (% OF A)	FOR GATHERING EXISTING DATA AND RECONNAISSANCE (% OF A)					
AZIBO	1580	54%	46%	8%	4905	32.2%	520	0.30%	0.9%
CAPINHA	280	31%	69%	9%	1225	22.9%	50	0.56%	2.5%
MEIMOA	1600	44%	56%	12%	13150	12.2%	930	0.17%	1.4%
MORGAVEL	190	43%	57%	14%	2305	8.2%	530	0.04%	0.4%
ODELOUCA	880	54%	46%	10%	4860	11.2%	1100	0.08%	0.7%
SABUGAL	610	38%	62%	12%	4520	13.5%	360	0.17%	1.3%

TABLE 5.III - Engineering geological staff costs: details and comparisons - EARTH DAMS

PROJECT	STAFF COSTS				TOTAL COST OF S.I. ($\times 10^3$ ESC.) D	STAFF COSTS/ COST OF S.I. A/D (%)	TOTAL COST OF CONSTRUCTION ($\times 10^6$ ESC. E	STAFF COSTS/ COST OF CONSTRUCTION A/E (%)	COST OF S.I./ COST OF CONSTRUCTION D/E (%)
	TOTAL ($\times 10^3$ ESC. A = B + C	DESK STUDIES AND REPORT PREPARATION B (% OF A)	FIELD STUDIES C (% OF A)	FOR GATHERING EXISTING DATA AND RECONNAISSANCE (% OF A)					
ALTO LINDOSO	1000	33%	67%	17%	4100	24.4%	4300	0.02%	0.1%
CASTELO DO BODE	2220	76%	24%	3%	8665	25.6%	690	0.32%	1.3%
FUNCHO-BENACIATE	700	50%	50%	7%	3675	19.0%	300	0.23%	1.2%
SABUGAL - MEIMOA	380	35%	65%	12%	1550	24.5%	230	0.17%	0.7%
SADO-MORGAVEL	800	40%	60%	16%	6270	12.8%	1060	0.08%	0.6%
S.TA CLARA-M.TE DA ROCHA	410	43%	57%	11%	5200	7.9%	1130	0.04%	0.5%

TABLE 5.IV - Engineering geological staff costs: details and comparisons - TUNNELS

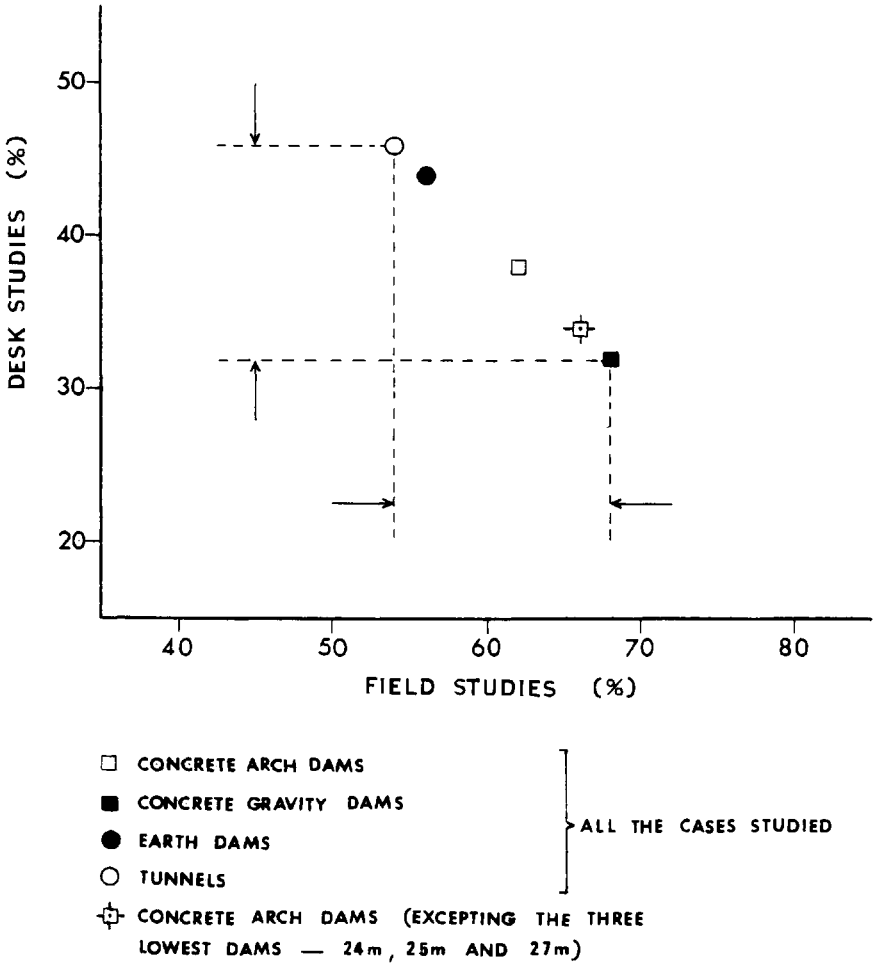


FIG. 5.2 - Engineering geological staff: relationship between the costs of field works and the costs of desk studies. (Percentage refers to the total cost of engineering geological staff).

igation approach in Portugal suggests that for the type of projects studied no less than 55% and no more than 70% of the money involved in site investigations is expected to be spent in field studies. Similarly no more than 45% and no less than 30% can be expected to be spent in desk studies.

The average amount of money spent for gathering existing information and reconnaissance was found to be 16% of the total costs of the engineering geological staff for the concrete arch dams, and 14% for the concrete gravity dams. For the earth dams and for the tunnels that value was found to be lower - 11% in both cases.

The costs of the engineering geological staff in relation to the total costs of site investigations showed a clear separation between the concrete arch dams and the other types of projects. The average values obtained for this relationship were about 9% for the concrete

arch dams, 18% for the concrete gravity dams, 17% for the earth dams and 19% for the tunnels. In the case of the arch dams studied, this value would fall to near 6% if the three lowest dams (25 m, 23.75 m and 27 m) were not considered.

The money spent in site investigations for geological staff is, in general only a very small percentage of the total construction costs except where projects are carried out in difficult terrain and where the projects are small.

5.2.2 - THE COST OF EACH TYPE OF WORK PERFORMED

A given type of work, e.g. boreholes, geophysical exploration, etc., can be performed more than once during site investigations. The data in Appendices A and C - vol. II reflect this point and there the costs are quoted separately for each instance.

Those costs were worked out in order to enable an assessment to be made of how much money was spent for each type of work during the entire site investigations in each one of the projects studied. These results, presented according to project type, are given in tables 5.V and 5.VI.

In the previous section it was pointed out that the costs of the engineering geological staff include the costs of special studies of jointing at Raiva and Valhelhas dam sites where they were undertaken by contract: this is unusual, for such studies are normally made by the geological staff of either the owner or the consultant. Also the integral sampling which is accounted for as a separate item in Vol. II (Valhelhas dam site and Sado-Morgavel tunnel) is accounted for here, with the borehole's cost.

Tables 5.VII and 5.VIII summarize the data in Tables 5.V and 5.VI and provide a statistical account of the costs of each group of works when actually performed. For these calculations, the cost of the works for Meimoa dam were not considered because the cost of trenches was exceptionally high. There was, apparently, no specific reason for such high cost and this caused distortion in the percentage of money spent with the other works undertaken.

It must be pointed out that although the average money spent in boreholes and galleries (which are revealed to be the most expensive works) for the concrete arch dams is very similar (Table 5.VII) in the

TYPE OF WORK	CONCRETE ARCH DAMS								CONCRETE GRAVITY DAMS					
	ALTO LINDOSO	ALVIDE	ATROZELA	FAGILDE	FOZ-CÔA	FRONHAS	FUNCHO	TORRÃO	JAMBA-IA-MINA	JAMBA-IA-OMA	TOUVEDO	RAIVA	VALEIRA	VALHELHAS
GEOPHYSICS	-	1.3	-	-	3.7	3.1	-	6.0	18.4	7.2	-	13.1	-	4.5
TRENCHES	2.2	5.3	0.9	1.7	23.2	11.6	4.6	15.1	-	-	11.9	3.6	3.0	13.7
SHAFTS	-	-	-	-	-	-	-	-	-	-	-	-	-	-
GALLERIES	48.9	42.5	-	-	30.0	24.6	36.1	36.3	-	-	-	-	23.7	-
BOREHOLES	36.7	28.8	78.9	87.6	21.5	19.1	17.3	9.7	69.1	78.4	73.5	56.2	59.1	54.8
"IN-SITU" TESTS	-	9.3	-	-	17.1	33.2	30.0	25.2	-	-	-	-	-	-
LABORATORY TESTS	4.7	1.8	-	-	1.3	1.0	2.4	2.1	-	-	-	-	-	-
ENG GEOL STAFF	7.5	11.3	20.2	10.7	3.2	6.2	9.5	5.7	12.4	14.5	14.5	27.1	14.3	24.7

TABLE 5.V - The cost of each type of work performed, as a percentage of the total cost of site investigations - CONCRETE DAMS

TYPE OF WORK	EARTH DAMS						TUNNELS					
	AZIBO	CAPINHA	MEIMOA	MORGAVEL	ODELOUCA	SABUGAL	ALTO LINDOSO	CASTELO DO BODE	FUNCHO--BENACI-ATE	SABUGAL--MEIMOA	SADO--MORGAVEL	S.TA CLA-RA - M.TE DA ROCHA
GEOPHYSICS	-	-	-	2.8	8.1	9.1	-	5.7	1.5	-	10.7	8.5
TRENCHES	13.6	6.9	64.2	-	5.1	22.6	7.1	-	-	-	-	-
SHAFTS	-	-	-	-	16.7	-	-	-	-	-	-	-
GALLERIES	-	-	-	-	26.8	-	-	-	-	-	-	-
BOREHOLES	54.2	70.2	23.7	88.9	32.1	54.9	39.8	51.1	50.6	75.5	61.7	63.3
"IN SITU" TESTS	-	-	-	-	-	-	24.1	16.2	26.9	-	12.8	19.4
LABORATORY TESTS	-	-	-	-	-	-	4.6	1.4	1.9	-	2.1	1.0
ENG GEOL STAFF	32.2	22.9	12.2	8.2	11.2	13.5	24.4	25.6	19.0	24.5	12.8	7.9

TABLE 5.VI - The cost of each type of work performed, as a percentage of the total cost of site investigation - EARTH DAMS and TUNNELS.

TYPE OF WORK	CONCRETE ARCH DAMS NO. OF PROJECTS - 8				CONCRETE GRAVITY DAMS NO. OF PROJECTS - 6				EARTH DAMS NO. OF PROJECTS - 6			
	RANGE (%)	AVERAGE (%)	COEFFICIENT OF VARIATION (%)	NO OF PROJECTS WHERE WK HAS BEEN USED	RANGE (%)	AVERAGE (%)	COEFFICIENT OF VARIATION (%)	NO OF PROJECTS WHERE WK HAS BEEN USED	RANGE (%)	AVERAGE (%)	COEFFICIENT OF VARIATION (%)	NO OF PROJECTS WHERE WK HAS BEEN USED
GEOPHYSICS	1.3 - - 6.0	3.5	55.0	4	4.5 - - 18.4	10.8	57.5	4	2.8 - - 9.1	6.7	50.8	3
TRENCHES	0.9 - - 23.2	8.1	97.8	8	3.0 - - 13.7	8.1	68.9	4	5.1 - - 22.6	12.1	65.8	4*
SHAFTS	-	-	-	-	-	-	-	-	-	16.7	-	1
GALLERIES	24.6 - - 48.9	36.4	23.8	6	-	23.7	-	1	-	26.8	-	1
BOREHOLES	9.7 - - 87.6	37.5	78.7	8	54.8 - - 78.4	65.2	15.1	6	32.1 - - 88.9	60.1	35.1	5*
"IN SITU TESTS"	9.3 - - 33.2	23.0	42.5	5	-	-	-	-	-	-	-	-
LABORATORY TESTS	1.0 - - 4.7	2.2	59.5	6	-	2.3	-	1	-	-	-	-
ENG GEOL. STAFF	3.2 - - 20.2	9.3	55.8	8	12.4 - - 27.1	17.9	35.1	6	8.2 - - 32.2	16.7	55.9	5*

* Works performed at Meimoa dam site not considered (see text)

TABLE 5.VII - A statistical account of the money spent in each type of work when actually performed - DAMS. (% refer to the total cost of site investigations).

TYPE OF WORK	RANGE	AVERAGE	COEFFICIENT OF VARIATION	NO OF PROJECTS WHERE WK HAS BEEN USED
GEOPHYSICS	1.5 - -10.7	6.6	60.1	4
TRENCHES	-	7.1	-	1
SHAFTS	-	-	-	-
GALLERIES	-	-	-	-
BOREHOLES	39.8 - -75.5	57.0	21.9	6
"IN SITU TESTS"	12.8 - - 26.9	19.9	28.8	5
LABORATORY TESTS	1.0 - - 4.6	2.2	64.0	5
ENG. GEOLOGICAL STAFF	7.9 - -25.6	19.0	38.2	6

TABLE 5.VIII - A statistical account of the money spent in each type of work when actually performed - TUNNELS
(% refer to the total cost of site investigations).

cases where these two types of exploration works were both used, the cost of the galleries exceeded that of the boreholes - 36% against 22%. It should also be mentioned that the lowest percentage of money spent in boreholes - 9.7% at Torrão dam site - seems to be unusually low. The next lowest value was 17.3% (Funcho dam site).

The other large amount of money spent for arch dams was, in average terms, on "in-situ" tests. Nevertheless, in two dams in particular (Fronhas and Funcho) their costs even exceeded the cost of the boreholes performed, which also happened in the case of Torrão dam site.

In the other types of projects considered, the greater part of expenditure during the studies went for boreholes whose costs, in relation to the total cost of site investigations are reasonably close to each other. In the case of tunnels, the "in-situ" tests costed consistently less than the boreholes.

It is worthwhile to mention again the costs of the engineering geological staff for gravity dams, earth dams and tunnels took, on average, a similar percentage of money (16.7% to 19%), however, in the case of concrete arch dams this average value was 9.3% of the total cost of site investigations. This difference may probably be explained by the greater amount of money that is generally spent in exploration and testing for this type of dam.

The coefficient of variation shown in Tables 5.VII and 5.VIII reflect the different percentages of money spent on each type of work and this is decided by the people responsible for the site investigation. In order to make their decision they have to consider the money available, the site conditions, and the characteristics of the structure, and then select the investigation methods which seem to be the most adquate, and the extent of their use. Thus, it is not surprising that the coefficients of variation in Tables 5.VII and 5.VIII are in some cases quite great. Note for example that the two most significant cost variations were assessed for works dealing with trenches and boreholes for the concrete arch dams, which present a very large range of heights.

5.3 - SITE INVESTIGATION COSTS VERSUS CONSTRUCTION COSTS

From previous experience of the relationship between the amount

of money spent in site investigations and the total cost of civil engineering works, it is possible to obtain some idea, in advance, of the capital which may or should be allowed for site investigations. Equally this may, in some way, be an indicator of the extent and standard of the site investigations which have been performed.

This experience has led to the acceptance of "orders of magnitude" for this relationship for the current cases of site investigations.

ATTEWELL and FARMER (1976), stated that in England it is usually found that 1% or 1.5% of the capital cost of the works is allocated for the site investigations. In a paper published in 1980, ATTEWEL and CLARK quoted the range of 0.5% - 1.5% as being the more usual one. TAYLOR (1975) referring to studies for tunnels, quoted percentages varying between 1.3 and 6.3%, stating that the tendency was towards the lower figure. In Portugal it is usual to hear that up to 3% of the construction costs should be allocated for site investigations; however, this value may rise to up to 5% in special cases.

It should be stressed that all the above mentioned figures are general quotes because the extent and level of site investigations will vary according to the requirements and characteristics of the design and also with the geological and geotechnical complexity of the site. The owners should be aware that although these figures can be adopted in a first assessment, flexibility is subsequently required.

The values obtained for the relationship between the costs of the site investigation and the costs of the construction for each of the 26 projects studied is presented in Tables 5.I to 5.IV. The figures used for the construction costs are the budgets included in the main design reports. These figures and the costs of site investigations have been corrected to a common basis - the equivalent to 1980 prices - using the Bank of Portugal's price index. The costs of construction only include the civil engineering works, and exclude the acquisition of hydromechanical and hydroelectrical equipment for the dams, as well as the costs of acquiring land.

If all the 26 projects of dams and tunnels studied are considered, the relationship between cost of site investigations and the cost of construction varies considerably between 0.1% and 8.9%. Nevertheless, if the different types of projects are accounted separately, the values vary as follows:

CONCRETE ARCH DAMS	- 0.4% to 8.9%, AVER. 2.6%
CONCRETE GRAVITY DAMS	- 0.4% to 0.8%, AVER. 0.6%
EARTH DAMS	- 0.4% to 2.5%, AVER. 1.2%
TUNNELS	- 0.1% to 1.3%, AVER. 0.7%

The greatest values in the case of the concrete arch dams, are specially influenced by two of the dams studied. They are planned for areas of difficult terrains (mainly karst limestones) and are, coincidently the two lowest dams studied - Alvide dam and Atrozela dam. If these are excluded from consideration, the other six dams in this group give a cost for S.I./cost of construction relationship that is found to vary only between 0.4% and 3.3%, with an average value of 1.3%.

The values of the relationship for the concrete gravity dams show a narrower range. The variation in size of the gravity dams included in this study does not vary so much compared with the case of arch dams. It is interesting to note that in the arch dams category the lowest values found for the relationship under consideration are exactly those obtained for the highest dams (Alto Lindoso dam and Foz-Côa dam).

For earth dams also the greatest value for the relationship under consideration (2.5%) is that for the lowest dam studied (Capinha). The lowest value assessed (0.4% - Morgavel dam) is certainly due to the existence of a great amount of information previously available from site investigations carried out for several other projects in the same area - Sines - and in the same geological formation. These studies included those for a previously selected site for this dam; a site which was abandoned after preliminary investigations. Thus, less money was needed in the investigations for this dam. For the other four earth dams, the values varied from 0.7% to 1.3%, the first belonging to the highest dam.

For tunnels, the lowest value (0.1%) was found in the case of Alto Lindoso. This tunnel clearly possesses the greatest thickness of overburden (300-400 m over the major part of its length) and this, together with the information available from one route previously studied should have influenced the approach to site investigations. It should also be noted that the next two lower values belong to the two longest tunnels. Thus it seems that both length and overburden can

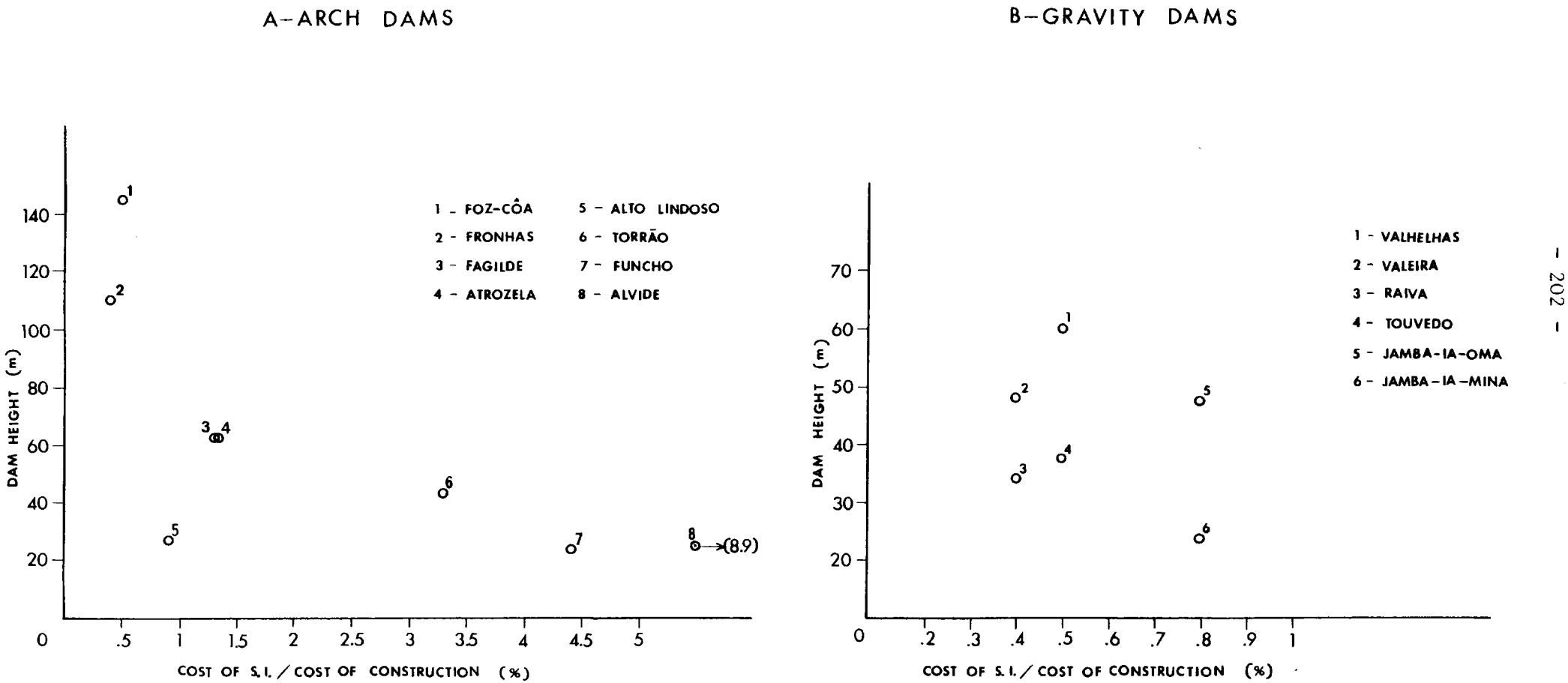


Fig. 5.3 - Cost of site investigations/cost of construction versus height, for (A) arch dams and (B) gravity dams.

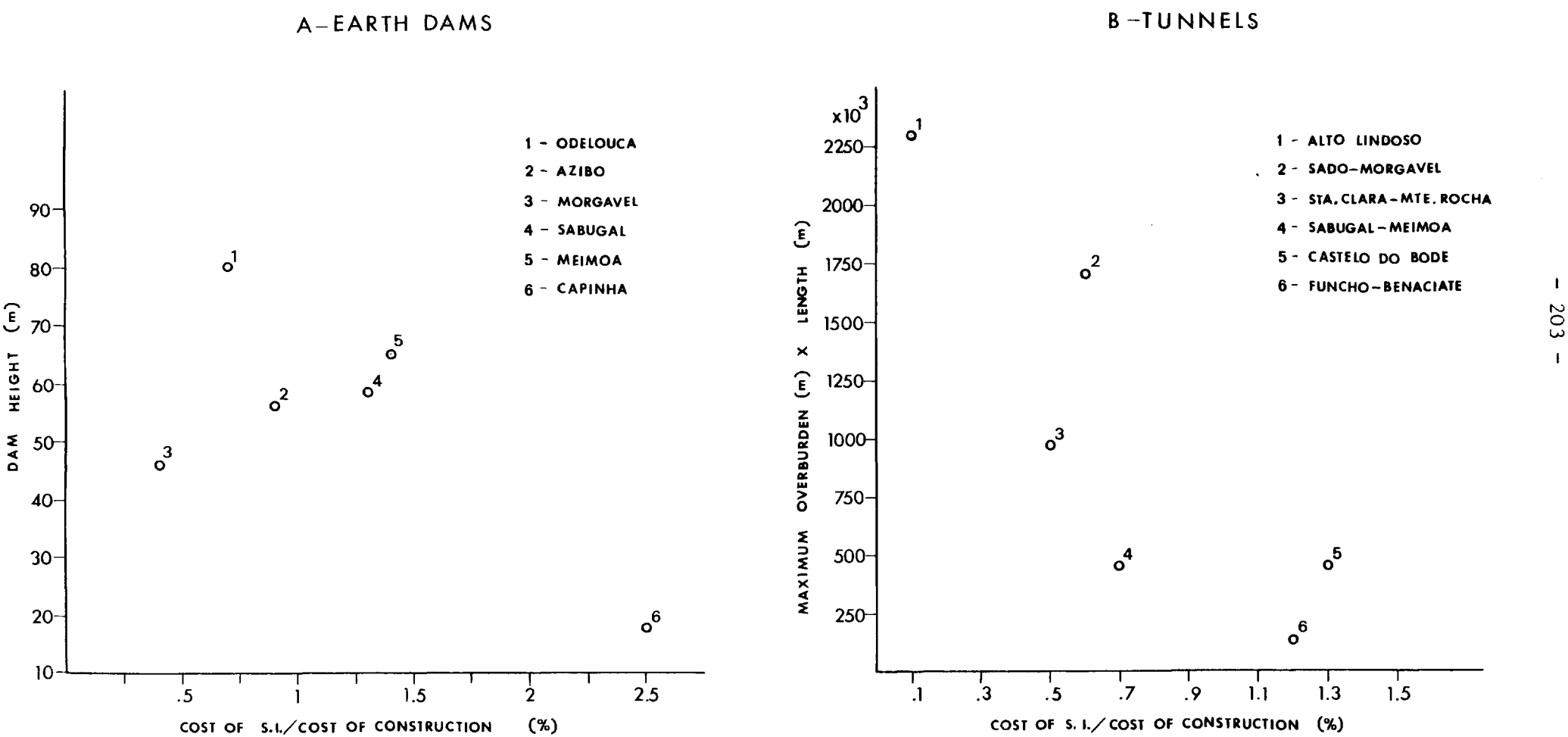


Fig. 5.4 - (A) Cost of site investigations/cost of construction versus height, for earth dams; (B) cost of site investigations versus maximum overburden x length, for tunnels.

influence the value of the cost of S.I./cost of construction relationship.

In Fig. 5.3 and 5.4, the values of that relationship are plotted against height for the three types of dams dealt with. Also, the appropriate values are plotted against the values for maximum overburden x length for the case of each of the tunnels.

CHAPTER VI

DISCUSSION

6.1 - GENERAL

In undertaking this research, the author had three main objectives in mind which, in some way could be useful to those involved in site investigations in Portugal, particularly the engineering geologists the consultants and the owners of projects.

These objectives were:

i) - To illustrate the Portuguese approach to site investigation on rock masses.

ii) - To assess the benefits of site investigations under the Portuguese approach.

iii) - To assess the costs for the benefits obtained.

To achieve the proposed objectives it was decided to restrict the research to dams and tunnels designed over the last decade.

Point (i) could have been dealt with in a more superficial way e.g. that of presenting the Portuguese approach to site investigations in a concise but descriptive manner. That would obviously be general in nature and would fail to fulfil the author's goal where detail in some instances is particularly important. The approach finally adopted was to present the individual histories for the 26 projects under study. The reader can then assess, by himself, those aspects of the Portuguese approach which are relevant to his particular interests, e.g. what was done, how and when.

The 26 case histories are presented in appendix A, Vol. II. In each case history it was decided to include information about

lithology, structure, state of weathering, hydrogeological aspects and the results of geotechnical tests performed. This inclusion of such detailed information may be justified by its usefulness for future site investigations. It is broadly recognized that, when assessing a new site, very valuable comparative data can be obtained from available information dealing with sites previously studied. This helps by saving time and money.

In chapters II and III, where the most common exploration and testing methods for site investigations on rock masses are discussed, the Portuguese practice is also considered.

Point (ii) the second aim of this research - assessing the benefits of site investigations under the Portuguese approach - was carried out by considering each of the 26 projects studied. For this purpose it was necessary to devise a special approach for weighting the benefits (explained in Chapter IV) because only in exceptional circumstances is it feasible to evaluate them in terms of money.

The weighting procedure adopted deals with the benefit in terms of the amount of knowledge of the site conditions required for design that was provided by the works performed. The people involved in the site investigations for the different projects (referred to as assessors, here) were personally interviewed and asked to assign marks for the benefit of each type of work, as well as for the site investigations as a whole, as explained in Chapter IV. The individual marks given by the assessors are presented in appendix B of Vol. II. Further calculations and pertinent discussion is included in Chapter IV, leading to some relevant conclusions.

The frequency of use of the different exploration and testing methods for site investigations on rock masses, as derived from the projects studied, is also presented and an analysis is made of the differences in the assessors attitudes towards the benefits.

In Chapters II and III, the benefits of the different site investigation methods, in terms of the information they can provide, are also generally discussed.

Point (iii) the third of the proposed objectives of this work - "what the Portuguese approach costs" - is initially considered in Chapters II and III. The prices currently used for the different exploration and testing methods as well as the time required for the

explorations and the number and qualification of the people involved in their performance, are quoted, and the factors influencing the costs are discussed. A common basis for comparing the costs of the most widely used methods has been provided where it was thought to be useful. The common basis chosen was the 1980 price for drilling one metre of a NX borehole (note, this covers only drilling operations).

In appendix A of Vol. II, the actual cost of each programme of works at the time of its performance is quoted for the different sites studied.

In appendix C, Vol. II, the money spent on each type of work at the completion of each site investigation is recorded. These costs have also been corrected to a common basis - 1980 equivalent prices - in order to allow a comparison to be made and enable discussion on the costs of either the different programmes of works carried out or the costs of the different site investigations (see Chapter IV) to be made. Here, special account is also given to the costs of the engineering geological staff and for the cost of site investigations as part of the total cost of construction.

From a survey of each of the 26 Portuguese projects studied a number of relationships between benefits and costs can be identified.

6.2 - RELATIONSHIPS BETWEEN BENEFITS AND COSTS

6.2.1 - APPROACH

The use of benefit/cost relationships, as they are commonly understood in economic terms, cannot be used here. For it to be meaningful, the benefits of the different works performed should be quantified in the same way as the costs, i.e. in terms of money. This is not possible and the method selected for their quantitative assessment (see Chapter IV) is purely subjective, dependent upon the feelings of the individuals involved in the site investigations; that is, the assessors.

Any conclusions which can be derived from relating the benefits to the costs can only be of a general character with respect to the Portuguese case. They are conditioned: a) by the feelings of the assessors as previously stated; b) by the prices demanded in Portugal; c) by the Portuguese approach in conducting the site investigations for

dams and tunnels on rock masses and d) by the small number of projects of each type which it was possible to consider.

The benefits used to work with throughout this chapter are the averages of the marks assigned by the assessors to the benefits obtained from each type of work (see Chapter IV) and the costs are expressed as a percentage of the total costs of the site investigations for the respective project (see Chapter V).

6.2.2 - RELATIONSHIP WITH DAMS AND TUNNELS

Initially an attempt was made to investigate the position of the points representating the different types of works in a benefit - cost graph , for the 26 projects studied. This is presented in Fig. 6.1.

The different zones illustrated in the graph were roughly defined by points representing the same investigation method. The large scatter in the case of boreholes reflects the versatility of this technique and the fact that it is the site investigation method most influenced, in relation to the money spent and the benefits derived, by the type and importance of the projects.

Although the normally accepted benefit/cost relationship cannot, as previously stated, be used here, the benefit/cost results as assessed in this research can be regarded as a coefficient, the RETURN COEFFICIENT (RC), which shows the return for the money spent in a given type of work in terms of the amount of actual knowledge of the site conditions it provided. This will serve for comparing the behaviour of the different work types in that it considers the return for the money invested in their performance. As an example, let us consider a programme of boreholes whose benefit assigned by the assessors was 30%. If the cost of this programme was the equivalent to 20% of the total cost of site investigations, then RC would be $30 \div 20 = 1.5$. For a given amount of money spent RC will increase as the benefit increases and for a fixed benefit RC will decrease as the money spent increases.

The return coefficient (RC) connected with each type of work performed in the site investigations for each of the four specific project types studied, were assessed and plotted against the costs of the same works. This is presented in Figs. 6.2 to 6.5.

These figures clearly show great differences in the RC values within the different types of works performed but in general show consistency for each type of project.

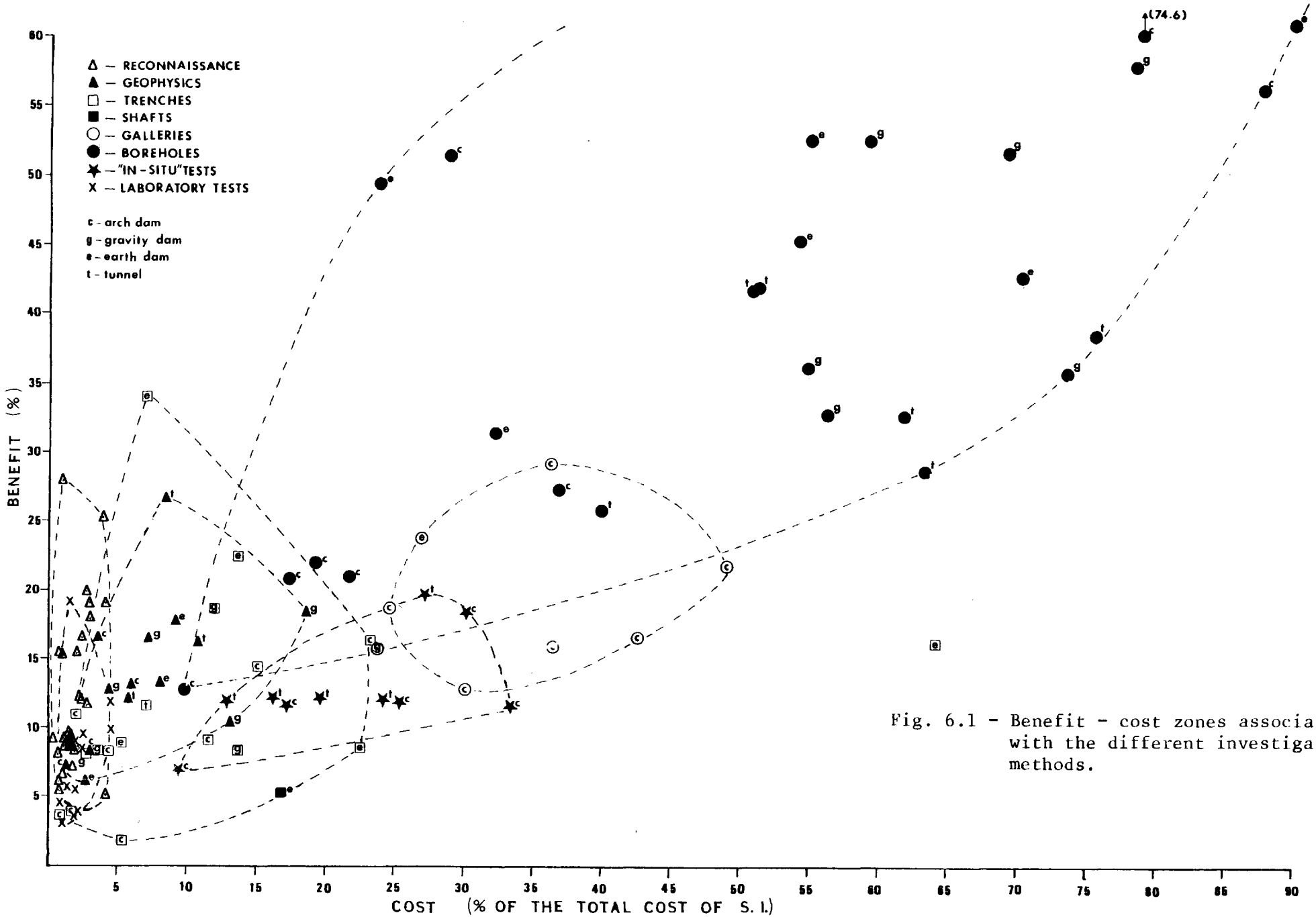


Fig. 6.1 - Benefit - cost zones associated with the different investigation methods.

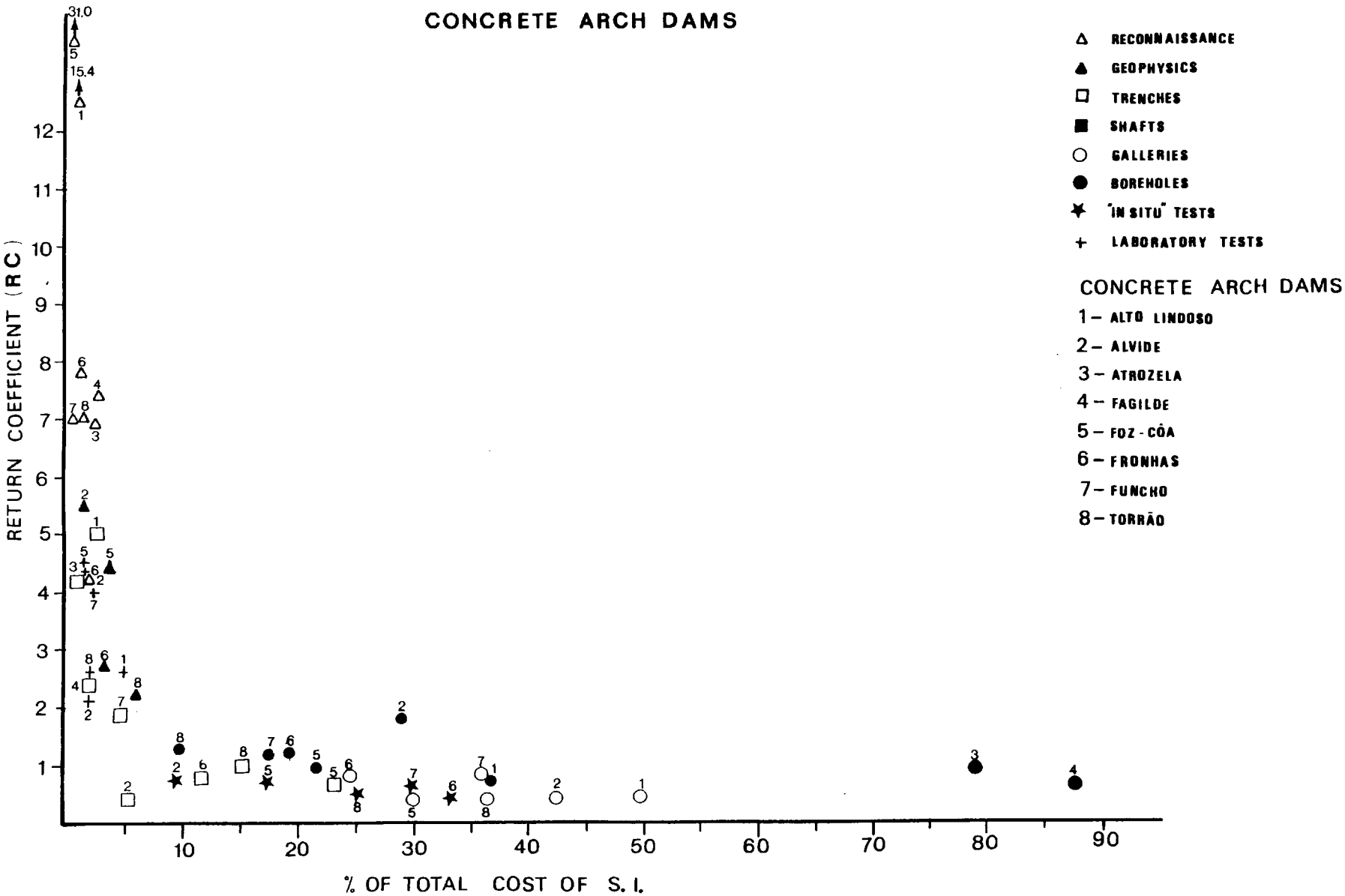


Fig. 6.2 - Return coefficient for the works used in each type of project.

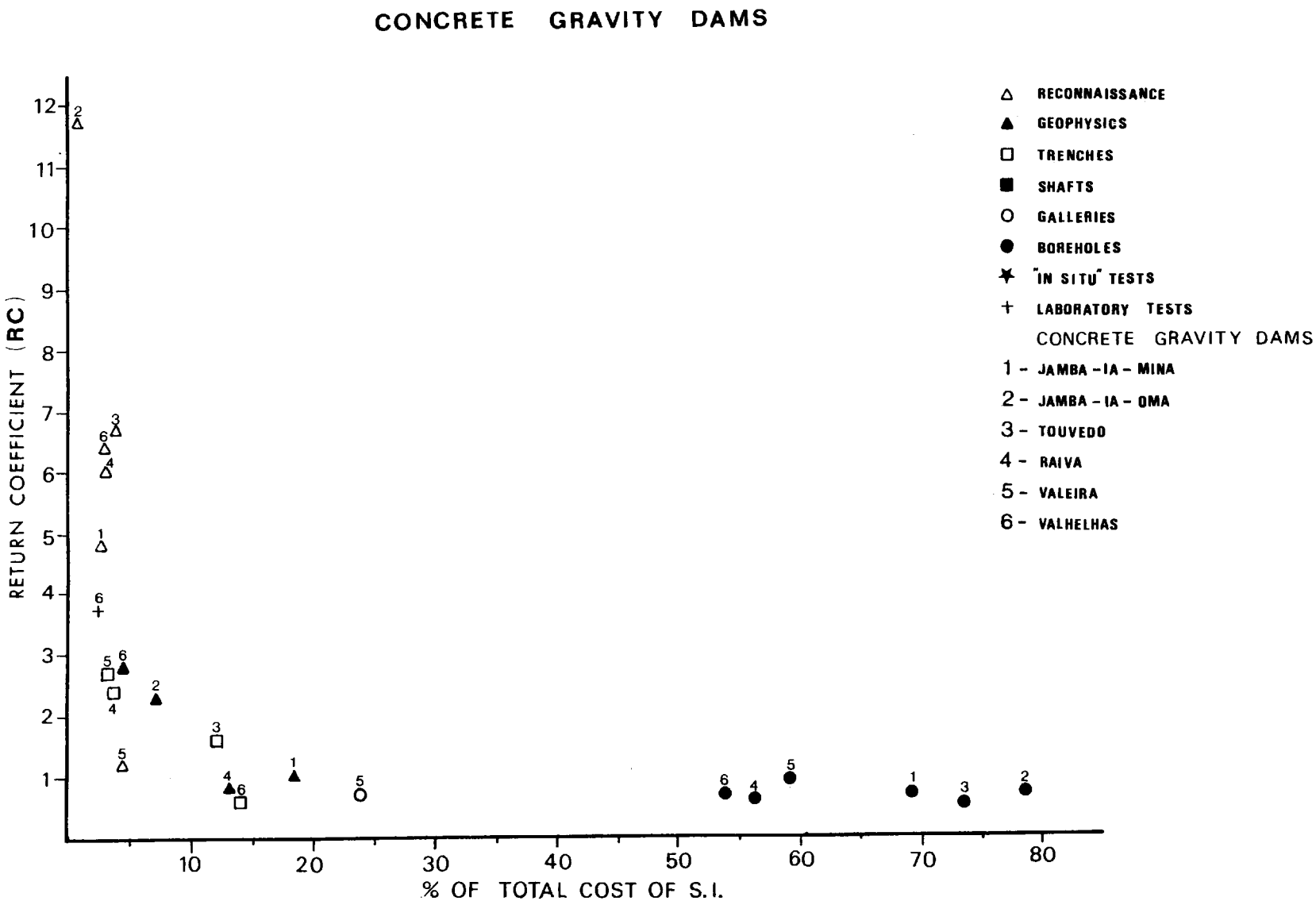


Fig. 6.3 - Return coefficient for the works used in each type of project.

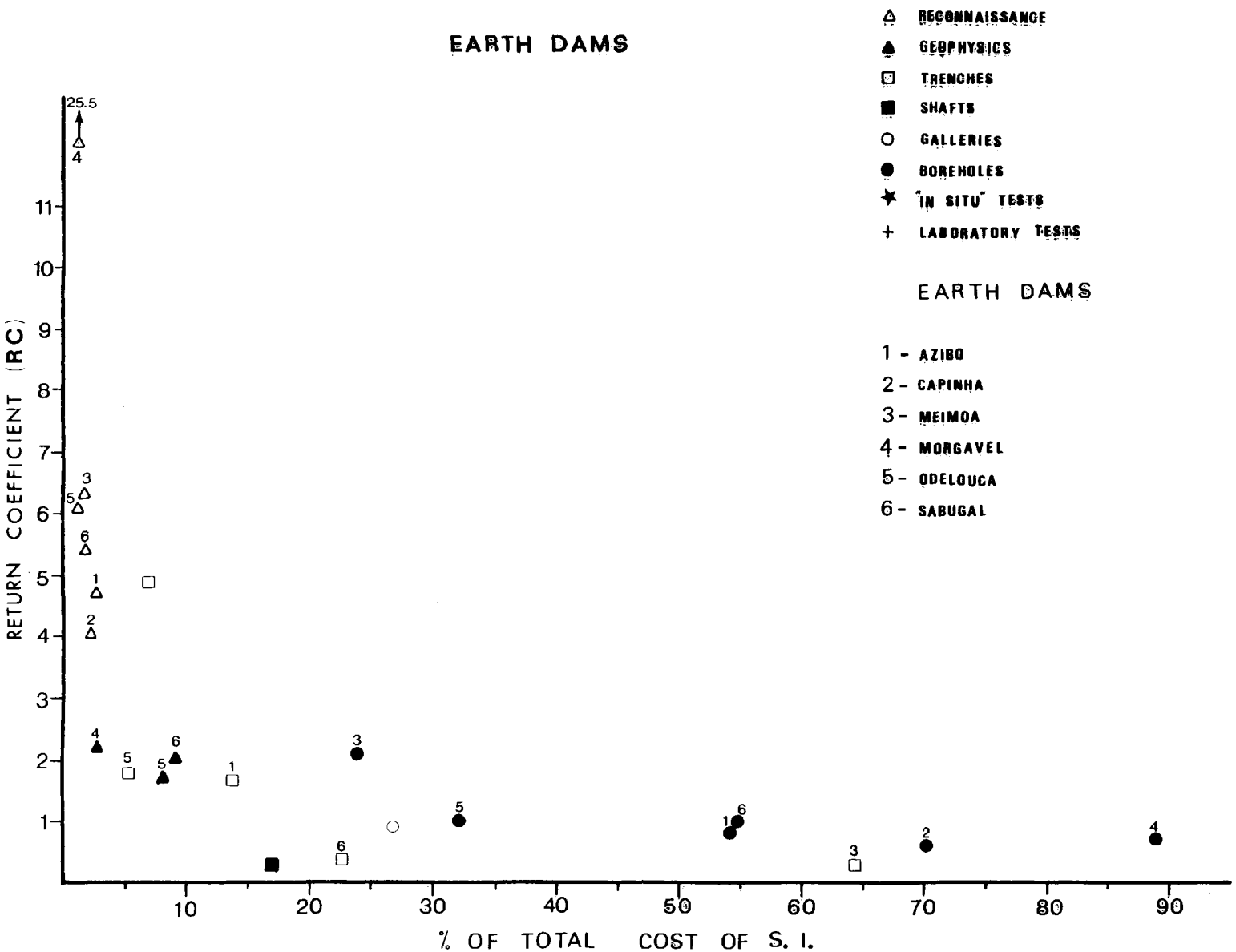


Fig. 6.4 - Return coefficient for the works used in each type of project.

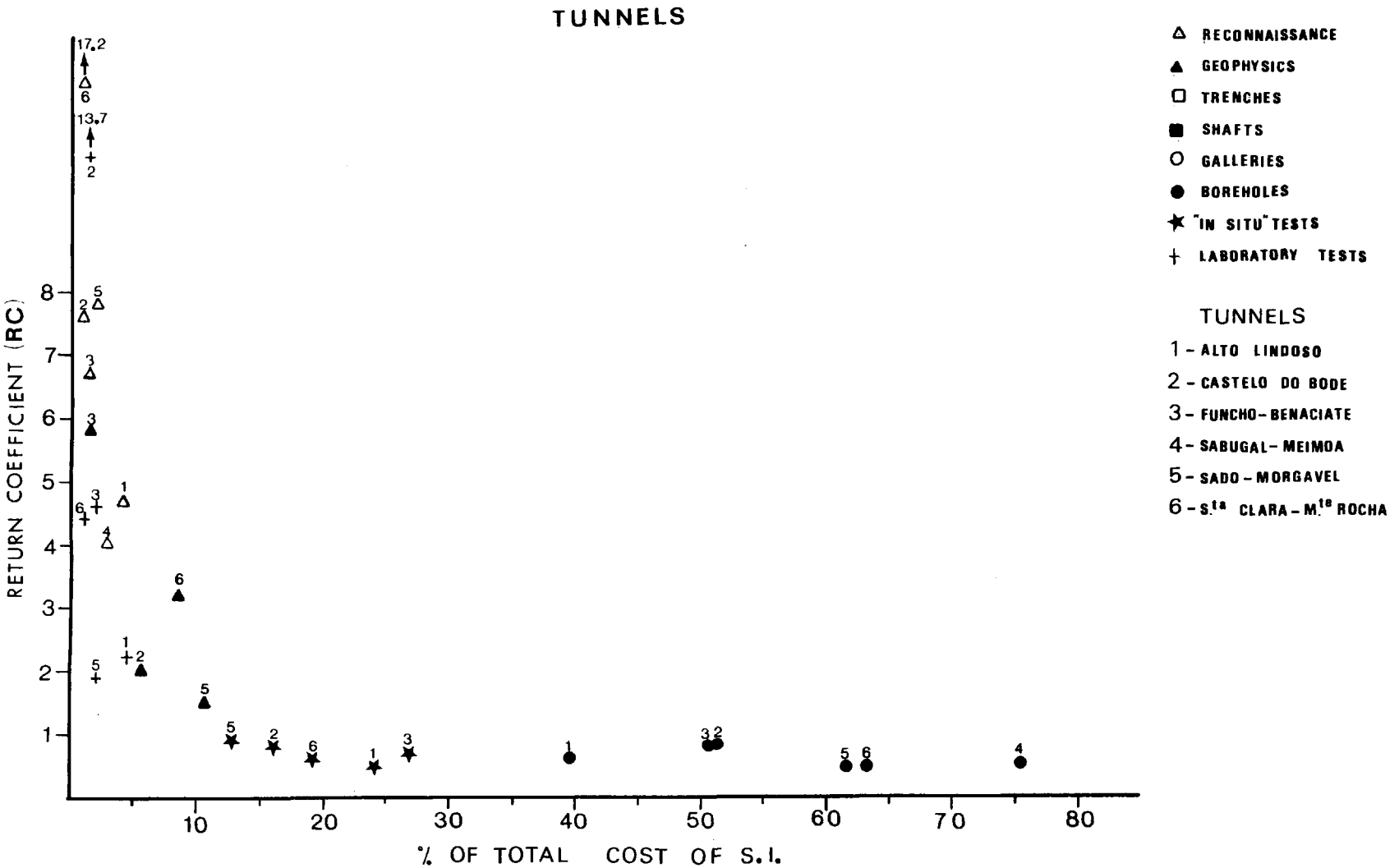


Fig. 6.5 - Return coefficient for the works used in each type of project

The most expensive works (galleries, boreholes, "in-situ" tests) which involved greater expenditures during site investigations, are those which present lower RC's. This contrasts with the cheaper works which normally take less money in site investigations ("reconnaissance and gathering existing information", geophysical exploration, trenches and laboratory tests). This explains why these types of works are used in the early stages of site investigations when a compromise exists between attempts to obtain the general picture of the rock mass and the amount of money usually allowed to be spent.

Trenches revealed in some cases relatively low RC values (see Fig. 6.2 - concrete arch dams), in the range of the boreholes, galleries and "in-situ" tests. This is, apparently, in opposition to the idea which people commonly possess about trenches, where they are regarded as being an inexpensive exploration method providing reasonable levels of information.

Two main reasons can give rise to situations like the one quoted above: a) the excavation took a far greater amount of money or b) the amount of information derived from these works was relatively poor. Within the trenches performed for the arch dams studied three of the four which gave lower RC's correspond to those which involved more money in their excavation (see Chapter V), although the benefits obtained were higher than the average benefit from trenches in site investigations for the type of projects quoted (see Chapter IV). The greater amount of money spent in these cases was due either to their long length (they correspond to the situations where the longest total lengths of trenches were excavated), or to an expensive excavation.

Taking one example from the earth dams considered (Fig. 6.4) the low RC assessed for the trenches at Meimoa dam site reflects the fact that they were extraordinarily expensive.

6.2.3 - RELATIONSHIP WITH INVESTIGATION TECHNIQUES

The graphs in Figs. 6.6 to 6.8 were prepared to complement those presented above (Figs. 6.2 to 6.5). Each graph deals with the RC's assessed for each type of work performed and includes all the results from all the types of projects. It appears that the trend of each graph would not be very different if a greater number of results were available allowing each type of project to be considered separately.

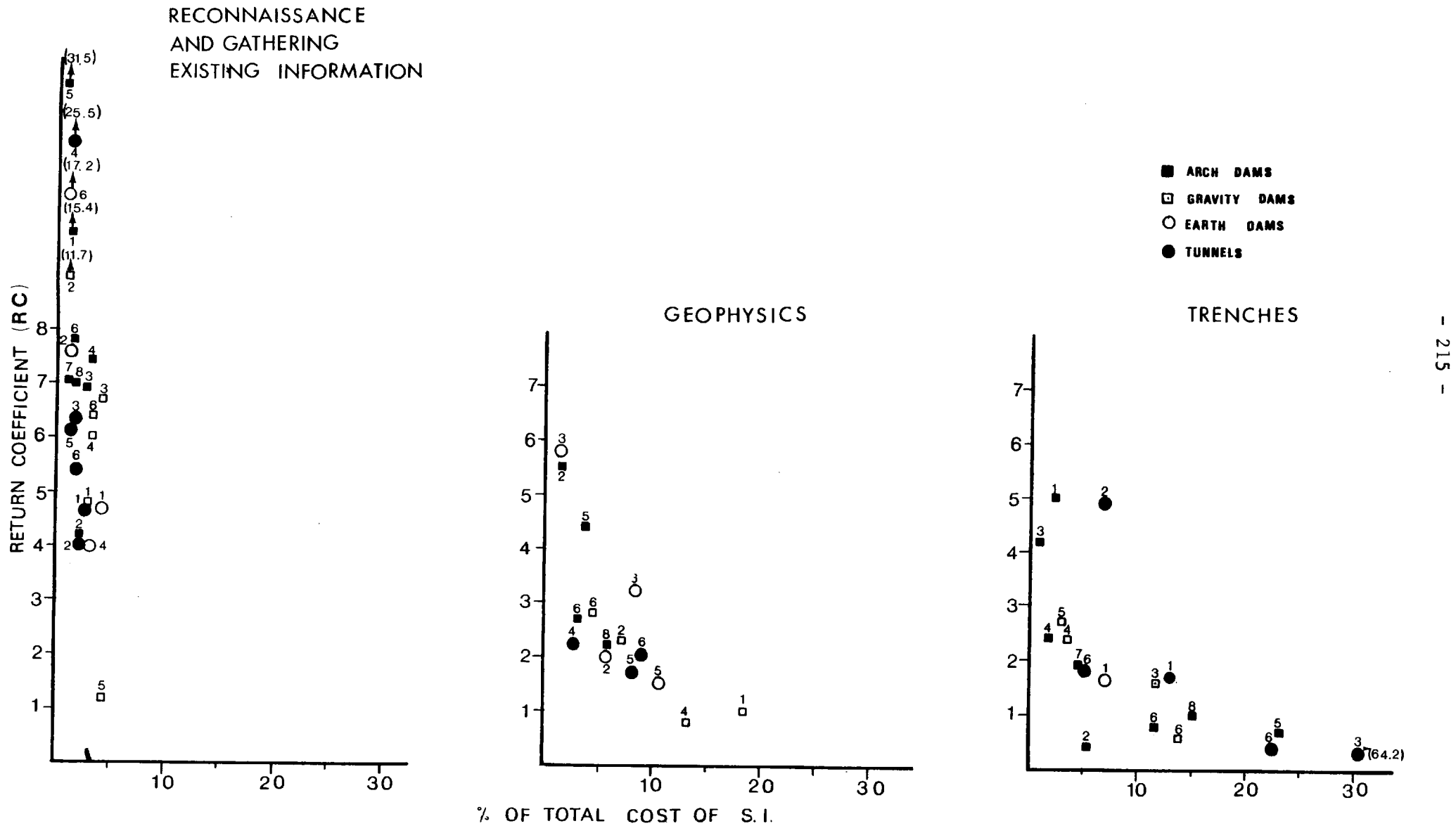


Fig. 6.6 - Return Coefficients for each type of work in site investigations.

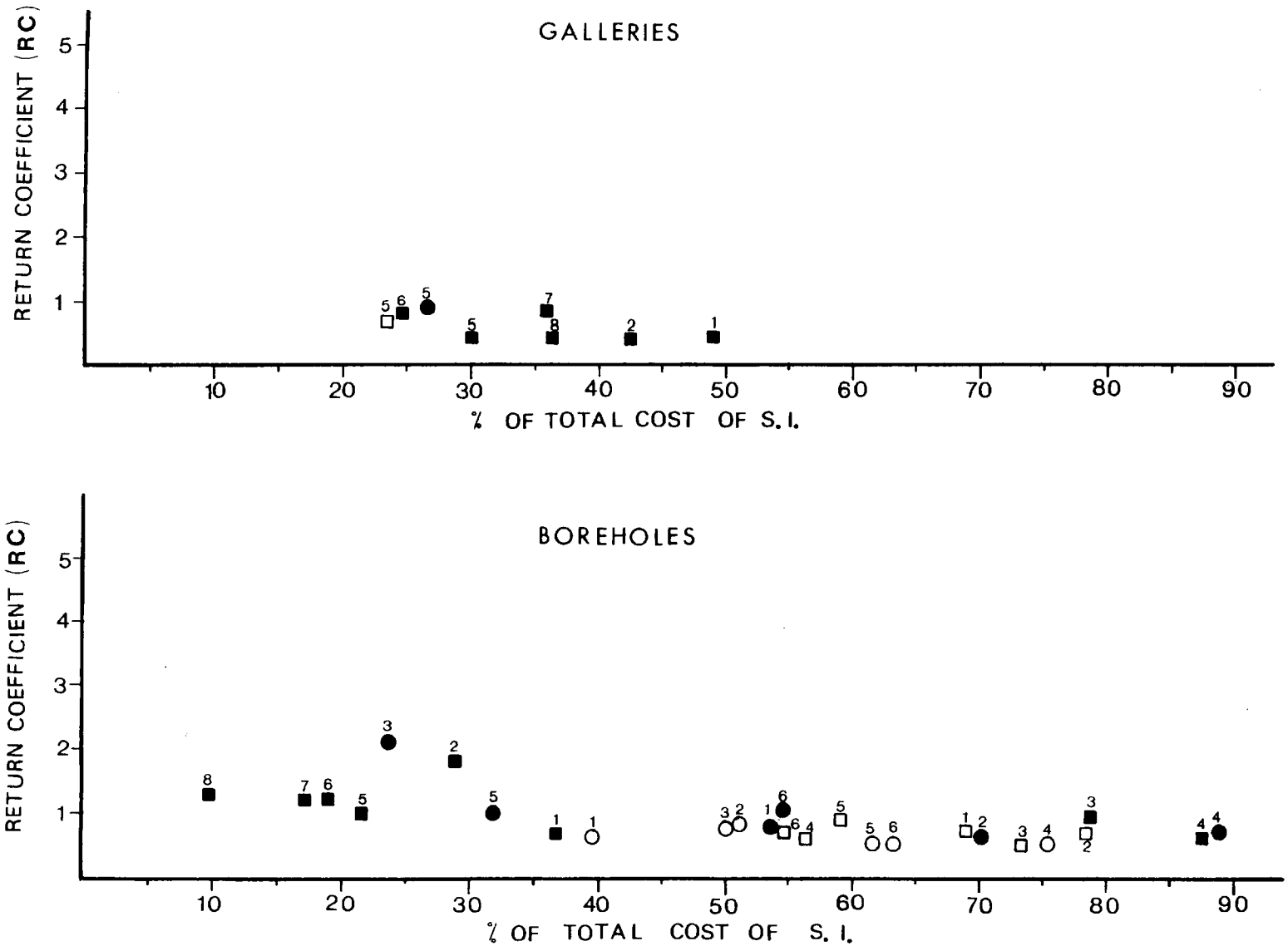


Fig. 6.7 - Return Coefficients for each type of work in site investigations.

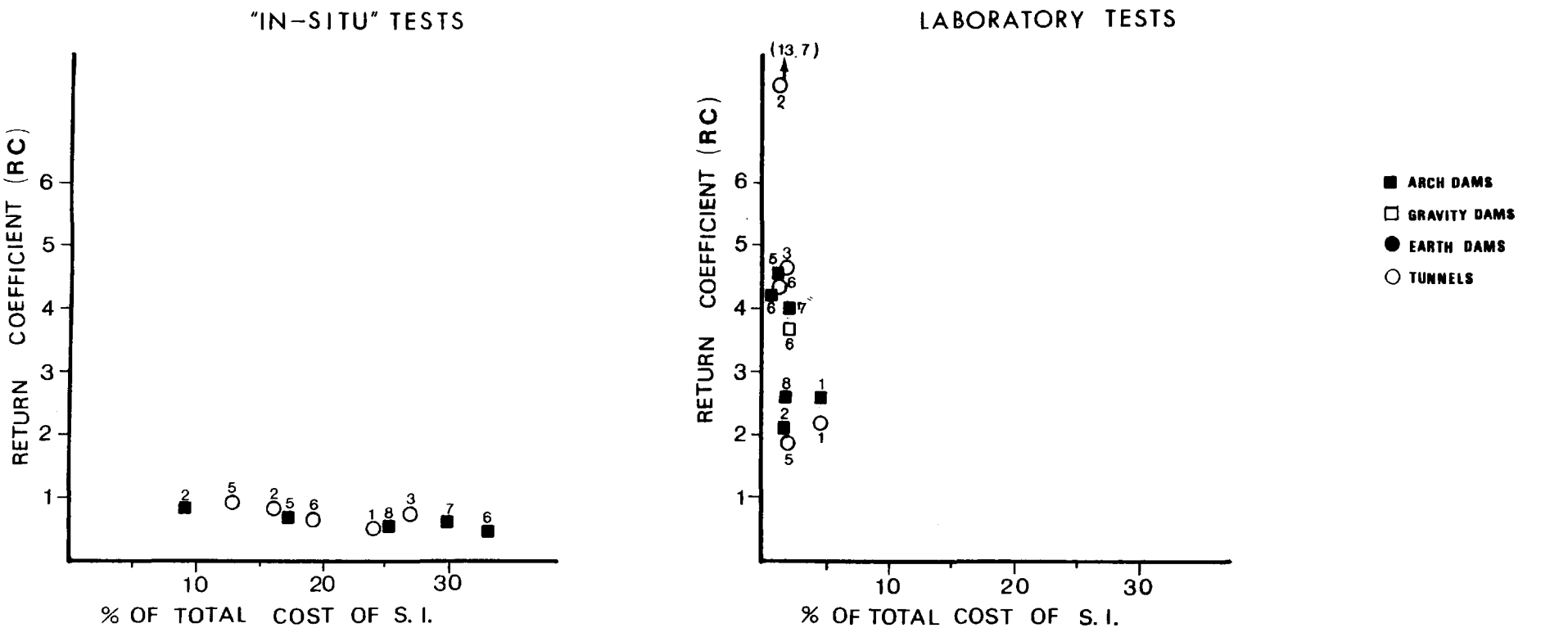


Fig. 6.8 - Return Coefficients for each type of work in site investigations.

The above graphs were used to define four groups, according to the RC's assessed:

A - Works with high to very high RC - " Reconnaissance and gathering existing information".
B - Works with high to moderate RC - Laboratory tests.
C - Works with high to low RC - Geophysical exploration and trenches.
D - Works with low RC - galleries,boreholes and "in-situ" tests.

The four groups above were established based upon the following RC values (see fig. 6.6 to 6.8):

RC > 6 - Very high
4-6 - High
2-4 - Moderate
< 2 - Low

It is understood by people involved in site investigations that, in general, as the amount of a given type of work performed increases (which normally can be assumed as an increase in the money spent in such work), the amount of information it provides does not necessarily increase proportionally. Beyond a certain amount of work carried out (cost), the additional information derived from a given subsequent capital investment begins to decrease. This is not only true for a given investigation technique but also for the site investigations overall.

The above is conceptually represented in Fig. 6.9, adapted from WALLACE (1974) which assumes an increase in the knowledge of the geological and geotechnical conditions of the site with a resultant decrease in risk costs from unpredicted ground.

The return coefficient as defined in this thesis,is knowledge ÷ costs, hence a plot of RC against costs (Fig. 6.10) is a plot of the rate at which graph on Fig. 6.9 changes with costs. Hence the RC graphs shown in Figs. 6.2 to 6.8 are a genuine reflection of a well established conceptual appreciation of the development of knowledge during a site investigation. Considering a given type of work, its return coefficient tends to decrease as the money spent in its performance increases. This is also valid for the site investigations overall.

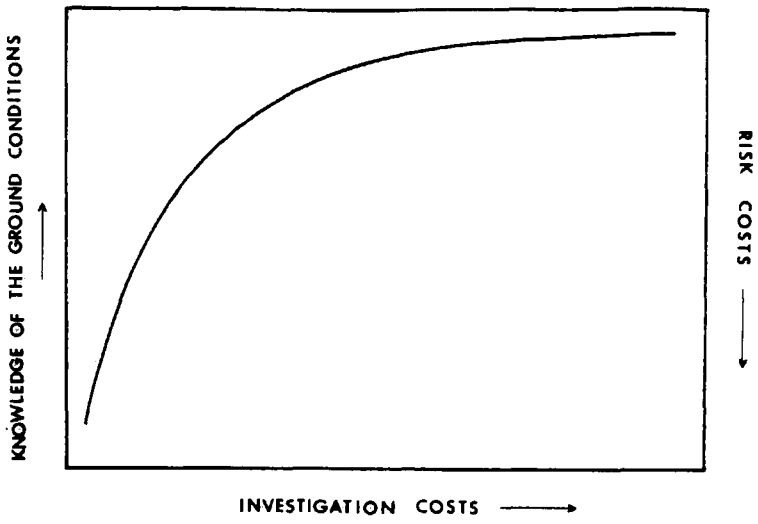


FIG. 6.9 - Variation of the knowledge of the ground conditions with the costs of investigation.

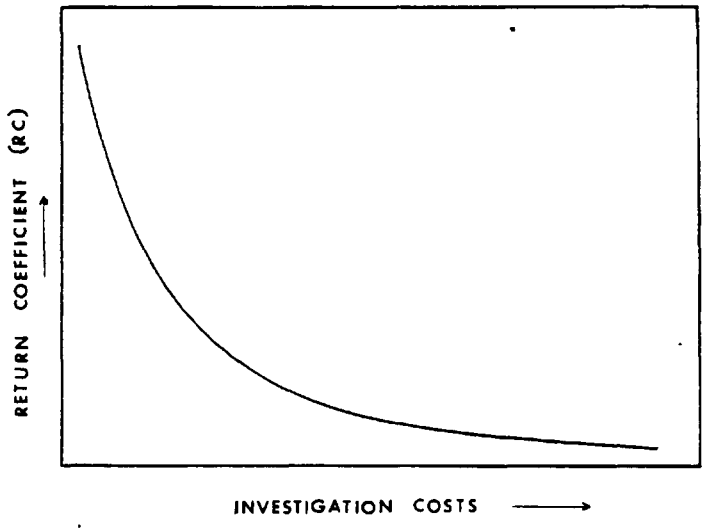


FIG. 6.10 - Variation of RC with the costs of investigation.

In the graph concerning the item "reconnaissance and gathering existing information" (Fig. 6.6) this trend is not noticed. The component "gathering existing information" can provide a substantial amount of information about the site conditions, at low prices; this influences considerably its RC value. In addition to this, reconnaissance only involves, as a general rule, a very small fraction of the total cost of site investigation due to the specific goal of this work. Thus, the

lower zones of curve 6.10 are usually not reached.

In the case of laboratory tests (Fig. 6.8) although they also show considerable variation with respect to RC values, the decreasing trend of this as the cost increases is either non existent or of little importance.

This may mean that "reconnaissance and gathering existing information" and laboratory tests, when used in the site investigations under study, have costs and associated benefits that suggest they lie in the upper part of the curve in Fig. 6.10.

As for geophysical exploration and trenches, the graphs (Fig. 6.6) suggest varied situations in relation to RC values for the total money spent for these types or works, in each of the projects studied. A clear decrease in RC in relation to cost increase can be demonstrated. Comparing these graphs with Fig. 6.10 it is apparent that while in some situations the work lies in the upper part of the curve 6.10, in others it falls mid way. In other cases this midway section was passed with less favourable RC values being reached. The detail reasons for these variations have already been discussed.

The galleries, the boreholes and the "in-situ" tests show, as a rule, low return coefficients whatever the amount of money spent in their performance. It can be said that the aims of performing these types of works are, in general, different from those previously quoted. Reconnaissance, trenches and geophysical exploration have commonly a broad exploratory character. They are used mainly for a first and general picture of the rock masses with regard to their geological and geotechnical characteristics, and to assist in the establishment of further investigation programmes. These later programmes involve the more expensive techniques like boreholes, galleries and "in-situ" tests, which will then provide information on the ground conditions at the levels needed for design. Once commissioned they have to be as extensive as is necessary to achieve information of the standard and reliability needed for design, in order to decrease the risk costs due to unpredicted ground conditions. In the case of "in-situ" tests it must be pointed out that in general the number of tests performed is already so low that cutting down their number will seriously reduce or nullify the reliability of the information they can provide.

From the above it is acceptable that, considering Fig. 6.10, the zone of work for galleries, boreholes and "in-situ" tests fall

within the lower part of the curve. It must be remembered that low values on the RC curve of Fig. 6.10 coincide with high knowledge from Fig. 6.9. RC reflects benefits and costs.

An attempt to improve the RC values related to these techniques, mainly in the case of galleries and boreholes can be achieved by improved planning of the works in order to save costs and through a more accurate and economic recording and interpreting the results.

In relation to "in-situ" mechanical characterization tests, an improvement can be attempted by associating the traditional tests with quality indices which can reflect the mechanical behaviour of the rock masses.

Considered on large scale an improvement in RC requires more than an improvement on site. Four main areas can be identified as being necessary for improvement if RC values are to improve overall. These are:

1 - Better organization or planning of work
2 - Better execution and recording
3 - Better use of geology in design
4 - Better use of geology in tendering

If geology is better used in this way, the benefits of the site work would improve with no significant increase in site costs.

Just as the concept of RC can be applied to a large scale it can also be applied to a small scale, for example to analyse individual types of works as the site investigation progresses; this is now discussed.

6.2.4 - RC VARIATIONS WITHIN DIFFERENT PROGRAMMES OF IDENTICAL WORK TYPES

In the previous sections of this chapter the exploration and testing methods were described in terms of their total costs and the benefits derived from each type of work, as assessed on the completion of the site investigations.

There are types of works which can be performed more than once (i.e. in different programmes) as the site investigations progress. This can be done either to improve the general previous knowledge of the site conditions or specifically to clarify particular problems which have emerged during investigations.

It can be said that both galleries and "in-situ" tests for mechanical characterization are seldom undertaken in more than one programme. In the case of these tests this is due to the high costs involved, (especially in the case of galleries), and also because of the high level of knowledge of the rock masses which is required in order to establish appropriate specifications (location, type and number of tests to be carried out).

The performance of a given type of work in different programmes does not necessarily mean that the return coefficients attached to each programme are similar. On the contrary, they will tend to decrease especially when a considerable amount of work had already been performed and the subsequent programmes are carried out to improve the general knowledge on the rock mass previously obtained. A different situation may occur when a new programme is undertaken to solve a specific and important problem, in which case the new programme may have a better RC value than the previous programmes.

Table 6.I shows the return coefficients obtained for boreholes and trenches performed in different phases of site investigations.

In preparing this table there was no intention to select particular examples. Instead, all the examples which were available within the 26 projects studied were included; even so they only provide limited data. Examples for geophysical exploration were not quoted due to the different methods utilized (e.g. seismic, resistivity etc.).

From table 6.I two main groups of examples have emerged. A first group (Group 1) where the RC's are almost identical for the different programmes of a given work type within the same project, and a second one (Group 2) in which they decrease in relation to that of the same works performed before.

In general, it may be concluded that when a great amount of work has already been performed, the tendency is for the return coefficient to decrease in subsequent programmes. This can be attributed to the fact that a great amount of knowledge about the rock mass conditions have already been gathered and additional information is more costly to obtain.

A single example of a clear increase in the RC value was found, related to the trenches at Azibo dam site. Here (situation A in table 6.I), the last two trenches were performed to clarify the existence of faults suggested by previous works. The very high RC value for these

	PROJECT	TYPE OF WORK	PHASE OF S.I.	BENEFIT (%)	COST (% OF TOTAL COST OF S.I.	RETURN COEFFICI-ENT (RC)
GROUP 1	ALTO LINDOSO DAM	9 boreholes 6 boreholes 5 boreholes	2nd 3rd 4th	12.0 8.8 6.6	19.1 10.0 7.6	0.6 0.9 0.9
	SABUGAL DAM	10 boreholes 8 boreholes	2nd 3rd	30.8 21.5	35.6 19.2	0.9 1.1
	FOZ-COA DAM	6 trenches 9 trenches	2nd 3rd	5.9 10.5	9.5 13.6	0.6 0.8
	CAPINHA DAM	1 borehole 7 boreholes	2nd 3rd	8.5 34.0	19.6 50.6	0.4 0.7
GROUP 2	ALVIDE DAM	6 boreholes 1 borehole 3 boreholes	2nd 3rd 4th	44.2 2.4 4.8	18.0 3.0 7.8	2.5 0.8 0.6
	CAPINHA DAM	1 trench 1 trench	2nd 3rd	25.5 8.5	4.1 2.9	6.2 2.9
	MEIMOA DAM	20 boreholes 12 boreholes + 3 auger borings	2nd 3rd	28.2 21.4	12.4 11.3	2.3 1.9
	ODELOUCA DAM	14 trenches 1 trench	2nd 4th	8.1 0.9	3.7 1.4	2.2 0.6
	ODELOUCA DAM	18 boreholes 12 boreholes	2nd 4th	22.5 9.0	18.6 13.5	1.2 0.7
	AZIBO DAM (A)	1 trench 1 trench 2 trenches	2nd 3rd 4th	9.3 8.1 5.4	8.0 4.7 0.9	1.2 1.7 6.0
	AZIBO DAM (B)	1 trench 1 trench 2 trenches	2nd 3rd 4th	9.3 8.1 5.4	7.8 4.6 2.4	1.2 1.8 2.3

TABLE 6.I - RC variations within different programmes of a same type of work.

two trenches in relation to that assessed for the previously excavated ones was in part due to their lower cost; they were excavated by machine whereas the other trenches listed in Table 6.I were excavated manually. If the costs of excavation for the trench performed during the third phase are considered the situation changes (see B) and the exceptionally high RC value encountered in situation (A) is now reduced by more than half. This situation of RC increasing as the site investigation programme progresses occurs normally when it is used to solve specific and important problems. The example quoted also emphasized the importance of selecting the cheapest methods to perform a given type of work providing they allow the information required to be obtained.

6.3 - CONCLUSION

The author is aware of the fact that much more could be done with the data gathered and presented. Within the limits of a research project with a fixed length of time, an attempt was made to undertake a job which will be of most use to those who are involved in performing site investigations. If this has only been achieved in part, the effort made has been fully compensated.

The factual information which has been recorded can, perhaps, give rise to other types of analysis and can constitute a first step in subsequent research. In this particular case, the information included in appendix A of vol. II, can form a basis for comparing the site conditions anticipated by the site investigations for the 26 projects studied, with the actual conditions encountered during construction taking advantage of the fact that the majority of these projects have not yet been constructed. One of the most interesting aspects connected with this would be to assess the costs due to the appearance of unanticipated unfavourable conditions and comparing them with the costs of the related site investigations as well as with the knowledge of the site conditions derived by the marks assigned by the assessors for this work.

Finally, the author wished to present a contribution dealing with the question of how much is likely to be spent on each type of work in current site investigations for dam foundations and tunnels on rock masses. This is shown in Table 6.II. This table was empirically prepared based on the studies undertaken and taken into account the knowledge that the author possesses of the Portuguese practice.

	GEOPHYSICAL EXPLORATION	TRENCHES	GALLERIES	BOREHOLES ✱	"IN-SITU" TESTS	LABORATORY TESTS	ENG. GEOLOG. STAFF
CONCRETE ARCH DAMS	UP TO 5 %	2 – 15 %	10 – 40 %	20 – 70 %	10 – 30 %	1 – 5 %	5 – 20 %
CONCRETE GRAVITY DAMS	UP TO 15 %	2 – 15 %	UP TO 20 %	40 – 80 %	5 – 15 %	1 – 5 %	10 – 25 %
EARTH DAMS	UP TO 10 %	5 – 15 %	–	50 – 80 %	–	–	10 – 25 %
TUNNELS	UP TO 10 %	–	–	40 – 80 %	UP TO 25 %	1 – 5 %	10 – 25 %

✱ COSTS OF BOREHOLES INCLUDE WATER ABSORPTION TESTS

TABLE 6.II – Costs distribution – a model for discussion (percentages refer to the total cost of site investigation).

The range of values used are wide enough to fit the most common situations except those concerning very small projects.

The proposed table should not be regarded as a code of practice; the author does not believe that such a code is desirable. It is intended solely to form a basis for discussion in order to find a better model of cost distribution which can be of help, in the first stage of planning site investigations, especially when less experienced people are involved.

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