# Geotechnical considerations in tunnel design and contract preparation

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### Introduction

> 'A major change occurred at 2400 feet inside the tunnel, where the contractor encountered an unstable material, which proved extremely difficult to support. All work was suspended for 4 months while the owner and the contractor disputed whether a changed condition had been encountered and discussed how to proceed with construction'.

This quotation from a case history, discussed by the US National Committee on Tunnelling Technology Subcommittee on Contracting Practices (The Academy, 1976), highlights two critical aspects of modern tunnelling:

(1) Geological unpredictability which is guaranteed to provide some surprises in even the simplest tunnelling job and

(2) Contractual problems which arise out of 'changed conditions' when these surprises occur.

In today's climate of rampant inflation and high interest rates, an extremely important consideration in all underground works is the completion of the project on schedule. An underground hydroelectric project under construction may be one of the most exciting places for an engineer to visit but, until it is complete and is producing power, it can also be a financial black hole into which huge amounts of public money can disappear. The same is true of almost all underground civil engineering works, the majority of which are owned by public utilities and funded by the average taxpayer.

The pressure to complete projects on time calls for the utilisation of the most modern excavating equipment and techniques and for the highest possible rates of advance. Unfortunately, many of the contractual practices and the attitudes of owners, engineers and contractors which can still be found on many underground projects around the world are more appropriate to the more leisurely days of hand tunnelling. Operating a multi-boom jumbo in a drill and blast tunnel or a high speed tunnelling machine under these conditions is rather like flying a modern jet under the air traffic control system of the 1940's - when things go well they go very well but when problems are encountered there is no reserve capacity in the system to deal with them.

> 'In the Brooklyn-Staten Island water tunnel, the mole progressed 75 feet in three successive shifts, but only 300 feet in 12 months before it was removed.' (Feld, 1974)

Most of the author's background and experience is in drill and blast hard rock tunnelling and hence the examples given and much of the discussion in these notes are related to this type of tunnelling. It is hoped that some of these discussions will be of interest to those concerned with other types of tunnelling.

**Geological investigation and geotechnical interpretation for underground works**

An important question which requires careful consideration at the start of any underground excavation project is: What constitutes a realistic site investigation for that project and to what extent can such a site investigation be expected to minimise the unknowns which could give rise to tunnelling problems and consequent contractual difficulties?

The prime purpose of any tunnel site investigation should be to obtain the maximum amount of information on rock characteristics, structural systems and groundwater conditions. This information is important to the tunnel designer in that it should enable him to anticipate the behaviour of the rock surrounding the tunnel and the type of support required to maintain the tunnel in a stable condition. The information is also important to the contractor in that it should provide him with a basis for establishing the optimum tunnelling method and the type of services which he will require in order to meet the construction schedules.

The first fact which must be recognised when planning a site investigation programme for a tunnel is that there is no such thing as a standard tunnel site investigation. Consider, for example, the contrast between a shallow tunnel in sedimentary materials in a European urban area which has been inhabited for several centuries and a tunnel through volcanic rocks in the South American Andes. The amount of previously available information and the information likely to be obtained from any site investigation differs by orders of magnitude in these two cases. In addition the amount of time and money allocated to one site investigation may differ so much from that of another that there is no hope that the end results will be comparable. Consequently, each site investigation programme must be carefully tailored to the specific site conditions, end results required and amounts of time and money available.

The linear extent of a tunnel means that it will probably traverse a greater variety of geological conditions than would be encountered in the excavations or foundations for most other engineering structures. Consequently, very careful consideration must be given to the amount of information which can be accumulated from a site investigation programme and the accuracy of the projections which can be made from this information.

Robinson (1972) has discussed the accuracy of geological projections in different geological environments and his comments are summarised below:

*Sedimentary rocks*

Mostly, these rocks are formed under relatively uniform conditions over relatively large areas. Subsequent metamorphism, folding and faulting, even though relatively complex, do not change the sedimentary rock to the extent that its original composition and areal extent cannot be recognised. This permits relatively accurate interpretation between data points and projection to the depth of a tunnel.

*Extrusive igneous rocks*

Like sedimentary rocks, but with more lithological variation, these layered rocks permit interpretation between data points and projection to depth with an accuracy related to the origin of the feature. Hence a basaltic flow may be expected to retain homogeneity between widely spaced data points and can be projected to depth with considerable accuracy. On the other hand, a rhyolitic ash flow may be of relatively limited extent and projection to depth may be much less reliable.

*Intrusive igneous rocks*

These are less predictable than extrusive igneous rocks and the accuracy of projection is of the same order as their areal extent. Hence a dyke can be interpreted between data points and projected to depth to the approximate extent of its surface expression. The least reliable feature of an intrusive igneous rock mass is the contact of a cross-cutting intrusive body. Robinson quotes an example from the Roberto tunnel in which the intersection of intruded igneous Montezuma Stock was about 2 miles more than projected from surface information (tunnel depth 3000 to 4000 feet).

*Structural systems*

In general, the extent of rock formations and their composition can be interpreted between data points and projected to depth with more accuracy than can structures or structural systems. A major fault formed at shallow depth may be projected horizontally and vertically over considerable distances while small faults and shear zones can only be projected over limited distances. As a general rule, the dimensions and characteristics of a structural feature, such as a fault, can be projected into the vertical dimension with no more accuracy than they can be interpreted at the surface. These comments are obviously rather general and the reader should not assume, for example, that tunnelling is automatically simpler in extrusive than in intrusive igneous rocks. In particular cases the reverse may be true.

Wahlstrom (1964), in discussing the problems of geological projection stated:

> 'Surface studies of geology, geophysical measurements, and exploratory drilling yield useful direct information, but equally important to the geologist may be a knowledge of the regional geology and the geologic history of the area, and a thorough appreciation of the manner in which rocks respond to changing geological environments. Such considerations permit him to make a very useful semi -quantitative estimate of the *kinds*, but not the exact locations, of geological features that will be encountered at depth'.

In addition to geological complexity, the importance of the general topography of the proposed tunnel route has been discussed by Rawlings and Eastaff (1974). They consider the influence of topography on site investigation programmes under the heading of low relief, moderate relief and severe relief.

In areas of low topographic relief, tunnels will tend to be shallow and the cost of drilling (including mobilization) will be relatively low. Geophysical techniques can be used effectively in such areas and, in general, it should be possible to carry out a reasonably comprehensive site investigation programme at a moderate cost. Two examples are quoted in which the site investigation costs were about 1.3 per cent of the final contract price for constructing the tunnels. Both tunnels encountered some difficulties related to geology and Rawlings and Eastaff suggest that, with hindsight, the problems could have been predicted if a little more site investigation work had been carried out.

In areas of moderate relief, for example, hilly country in which the tunnel cover may vary between 100 and 500 metres, the extent of a site investigation programme depends largely upon access. Costs of both mobilization and drilling may be considerable and these costs may decide the amount of drilling which can be carried out along the tunnel line. Where the geology has a very strong bearing upon tunnel construction and it is essential to obtain data, alternative means of investigation such as probing ahead of the face have to be considered.

Areas of severe relief, in which the cover over the tunnel exceeds 500 m, are the most difficult for tunnel site investigations. In addition to the cost of diamond drilling, the cost of making and maintaining access to the drilling sites must be considered. In some cases, any access except by helicopter may be impossible. Under such conditions, reliance has to be placed upon the interpretation of air photographs and whatever surface mapping can be carried out.

The following example from Peru, quoted from Rawlings and Eastaff (1974), illustrates the difficulty of obtaining data, of carrying out a meaningful analysis of these data and of the subsequent tunnelling problems in an area of severe relief:

> ‘The Trans-Andean Tunnel is just over 10 km long and was driven at approximately 4300 m above sea level. It took 4.75 years to drive and was finally pumping 425 gals/sec at the eastern portal, and was delivering 345 gals/sec at the western portal. The maximum inflow at the face was 900 gals/sec. Water with a temperature of 35 degrees F was encountered necessitating ‘the workers to be given woolen socks, gloves, turtle neck sweaters and trouser-boots’. But through another stretch, the workmen had to be furnished with frogmen’s outfits to erect arches while partially submerged. The geology comprised some 4 km of intensely folded and fractured marl and 5 km of volcanic rock. Faults were common and outbursts of water frequent. A particularly unusual phenomenon was the appearance of lenses of volcanic rock that disintegrated as they came into contact with the air and ‘turned into powder’. No exploratory drilling was done, although predictions could be made from the surface geology concerning the rock types and structure. Water was anticipated, but not at the rates actually encountered. It is doubtful whether any drilling could have given data on the magnitude of these problems’.

It will be clear, from the examples quoted, that the information provided by a site investigation can range from adequate, permitting reasonably precise design and scheduling, to totally inadequate for a conventional contract. In the latter case, only the general methodology can be established and a ‘design-as-you-go’ method adopted in which all parties have to work in close cooperation in order to overcome whatever problems are encountered.

**Disclosure of geological data and geotechnical interpretations**

In discussing 'foreseeable' ground conditions a CIRIA report on tunnelling contract practices (No. 79, 1978) stated:

> 'Historically, little or no site investigation information was made available to tenderers and that given was specifically disclaimed. The tenderer was free to make such investigations as time and his inclination allowed and took the full risks arising from the ground not behaving in accordance with the model he had formulated'.

While this system was suitable for very simple tunnels in familiar ground, it proved unworkable in more complex geological environments. This lead to the concept of 'unforseen conditions' and the introduction of the 'changed-conditions' clause in tunnelling contracts.

The purpose of a changed-conditions clause is to permit the owner to assume the risk of unknown subsurface conditions and to reduce the need for large contingencies in contractors' bids. Obviously a changed-conditions clause can only operate effectively if a set of reference conditions can be established against which any 'change' can be judged.

In the United Kingdom, the CIRIA working party recommended that a set of Reference Conditions be established by the engineer and, after discussion with, and modification by the contractor, these be used as a basis for the settlement of disputes. These reference conditions should cover one or more of the following (CIRIA Rep. 79, 1978):

*Geological*
Description of the strata in terms of engineering geology, including geological structure, a lithological description of the materials and the groundwater regime.

*Method*
Description of a defined method of construction.

*Response*
Behaviour of the ground in response to tunnelling or to geotechnical expedients.

*Rate*
Rate of advance.

A major factor in establishing reference conditions and agreeing to these conditions with the contractor, before tunnelling commences, is the extent to which geological information and interpretation is disclosed.

To quote from the CIRIA report (No. 79, 1978):

> 'To be effective, the Reference Conditions should be presented in a rational and systematic manner so that the physical facts, and whatever interpretations of them are made, are explicit'

The US National Academy of Sciences report (1976) states:

> 'Geological exploration is not an exact science and expert opinions must sometimes be based upon fragmentary evidence. Nevertheless, both parties in a contract must have an understanding of the conditions likely to be encountered. This common understanding of the design considerations involved can only be reached by providing all data, including professional interpretations thereof, to prospective bidders'.

Both the CIRIA report and the US Academy of Sciences report were prepared by working parties made up of very experienced tunnelling men. Their recommendations on the disclosure of geological information and the interpretations thereof are clear and unambiguous - all relevant information must be made available to bidders on a tunnelling contract.

In the author's experience, an increasing amount of information is being supplied to bidders. However, it is not the quantity but the quality of this information which is a source of concern.

In a recent paper on the causes of claims in tunnel contracts, Waggoner (1981), an experienced consulting engineering geologist, states:

> 'Geological data are often too generalized, not sufficiently site specific. Frequently they use terms that are misleading or ambiguous and do not include data that a contractor really needs'.

The same concern is expressed by D'Appolonia (1981), an equally experienced consulting engineer:

> 'This factor - data for construction - is often ignored in the planning and design of a tunnel and in the preparation of contract documents'.

These comments highlight a particular problem in tunnel site investigations - execution of a site investigation programme is one thing, interpretation of the data from such a programme in terms of tunnel behaviour is another. Many companies which are thoroughly competent in the techniques of geological data collection and geotechnical site investigation have limited experience in tunnelling. In the author's opinion, one of the most effective means of overcoming this problem is to establish a review panel of one or more experienced tunnel engineers to assist in the preparation of or the review of specifications and contract documents. While this may appear to be a duplication of effort, the investment made in a competent review panel at an early enough stage in an underground construction project will probably pay for itself many times over during the course of the project.

During the past decade, a number of rock mass classification systems have been introduced, notably by Wickham et al (1972), Bieniawski (1974), and Barton et al (1974). These classifications represent a serious attempt to collect together relevant geological and geotechnical data and to organise these into a rational system for predicting tunnel behaviour and estimating support requirements. All of these systems have some limitations and the predictions made on the basis of the classifications are not always accurate. Nevertheless, when used intelligently, these classifications can provide an effective basis for establishing reference conditions and for subsequent discussions between engineers and contractors. The author is optimistic that, with greater familiarity, these rock mass classification systems will play an increasingly important role in underground construction.


Figure 1 : Types of tunnel contract arranged in a hierarchy dependent upon the allocation of risk (after CIRIA, 1978).

**Types of contract**

Figure 1 lists the types of contract which can be considered for underground construction. The contract types are arranged in a hierarchy which depends upon the allocation of risk between the owner or promoter and the contractor. The types of contract listed in figure 1 are defined below:

*Turnkey*
The contractor is responsible for site investigation, design and construction of the project for a fixed price.

*Lump Sum*
A single price is given for all the work or for completed sections of the work. This type of contract is referred to as a firm-fixed-price contract in the United States and is the most common type of tunnelling contract used there. Provision can be made for limited changes by the inclusion of a contract price adjustment clause.

*Admeasurement*
These contracts are based upon Bills of Quantities or Schedules of Rates and payment is determined by measurement of the completed work at initially tendered or subsequently negotiated rates.

*Target*
These contracts are based on the setting of a probable cost for the work which is adjusted for changes in the work and escalation of cost.

*Cost-reimbursable*
The contractor is paid the actual costs incurred in carrying out the work. A separate fee may be negotiated for management overheads and the profit element.

A full discussion on these types of contract exceeds the scope of this paper and the reader is referred to the comprehensive discussions and bibliographies given in the CIRIA report (79, 1978) on tunnelling contract practices and the US National Academy of Sciences report (1974). The latter contains a particularly useful summary of tunnel contracting practices in the United Kingdom, France, Italy, Norway, Sweden, Switzerland and West Germany.

Returning to the earlier discussion on the influence of geology and topography upon the amount and quality of subsurface information which can be obtained during a site investigation, it is useful to consider the relationship between this information and the type of contract which should be considered. In the case of a tunnel site for which a substantial amount of subsurface information is available, a turnkey or lump sum type of contract may be appropriate - as, for example, in a tunnel that is being driven parallel to an existing tunnel for which good construction records are available or where reliable subsurface information is available from a well planned preliminary contract involving a pilot tunnel or exploratory adits.

At the other end of the spectrum, the use of a turnkey or lump sum contract would make very little sense for a tunnel in steep mountain terrain of volcanic rock such as the Trans-Andean tunnel discussed earlier. In such a case, a target price or a cost reimbursable type contract would be much more suitable. These types of contract are particularly suitable for situations involving major or unquantifiable risk and deserve more serious consideration than they have been given in the past. A useful summary of the use of target and cost-reimbursable contracts has been published by Perry and Thompson (1975).

Intermediate between these two extremes is the admeasurement type of contract which is frequently used by United Kingdom based engineers for underground projects. The degree of success which can be achieved when using this form of contract is directly proportional to the adequacy of the bill of quantities estimates. For example, the quantities of different kinds of tunnel support are estimated and included in the bill of quantities and the support is then installed on the instruction of the engineer on a ‘design-as-you-go’ basis. This provides excellent flexibility when working in tunnels in which the ground conditions ahead of the face are unknown but it carries the risk that some of the quantities may have been grossly over- or underestimated and the owner may face claims based on these variations. Some of these risks can be minimised by including a percentage over- or underrun from the estimated quantities within which the contractor’s bid unit prices will still apply. In addition, a check on the bill of quantities by a competent review panel of one or more experienced tunnel engineers can help to ensure that estimated quantities are realistic.

Whatever type of contract is decided upon, few would argue with the following two quotations from the US Academy of Sciences executive presentation on Contracting for Underground Construction (1976):

> 'The Contract drawn between the owner and contractor seeks to define the requirements of the underground project, to assign the responsibility for its accomplishment, and to establish its cost. A good contract does not merely divide the responsibilities of the project; it is a unifying force, an agreement committing both parties to a single common objective. Every provision in the contract must be an acknowledgement not only of the legitimate interests of the individual parties but their common goal.
>
> The system works best when the engineer, contractor, and owner establish the attitude through their organisations that each party is knowledgeable, fair-minded, cooperative, competent, and willing to see equitable payment made for the work'.

These quotations highlight the fact that it is not only the form of contract but also the attitudes of all of the parties involved in negotiating the contract which determine whether an underground project will be completed on schedule and within budget or end up in court with an endless succession of claims. The importance of attitudes in dealing with tunnel problems is dealt within an excellent two page paper by Kuesel (1975) in which he describes how an unexpected fault zone would be dealt with in Swedish and American tunnelling environments respectively.

As a final comment on tunnel contracts, the author strongly recommends that, whenever permitted by law, a contract should contain provision for arbitration. In some cases it may be possible to resolve difficulties by setting up an arbitration panel comprising one member chosen by the owner, a second by the contractor and a third chosen by the first two members. In other cases, more formal arbitration is required but, as a general rule, arbitration is an attractive alternative to long and costly litigation as a means of resolving disputes.

**Provision for water**

The entry of large quantities of water from the tunnel face or from the rock surrounding the tunnel is one of the most troublesome problems which can be encountered in underground construction. Although it is sometimes difficult to predict the location and the extent of water problems and to specify how these problems should be dealt with, it is essential that provision should be made to pay the contractor for dealing with these problems.

Some tunnelling experts maintain that the contract should include a separate pay item for the handling of all water with a rate of payment calculated on the basis of the quantity of water being handled. Others prefer the inclusion of a pay item which only becomes effective when the quantity of water being handled exceeds a specified amount for a given period of time. For example, a flow of one gallon per minute per foot of tunnel (15 litres/minute/metre) for a period of 24 hours has been used as a basis for initiating separate payment for water handling. The intent of this type of provision is that the handling of 'normal' water quantities should be included in the contractor's overall bid but, in order to avoid large contingencies, unusual quantities of water are paid for separately by the owner. Clearly, the quantity of water and the length of time chosen as a basis for deciding when water handling ceases to be 'normal' will depend on the circumstances which apply in each case.

Consideration must also be given to payment for water treatment since, in many parts of the world, water from a construction site can only be released into the regional water system if its quality meets specified standards. In some cases, water treatment to ensure that these standards are met can be very expensive and it may be prudent to include this treatment as a separate pay item.

Where there is a possibility of unexpectedly encountering significant quantities of water, it is important to specify that a probe hole should be drilled ahead of the face. Typically, such a probe hole should extend 2 to 3 tunnel diameters ahead of the face at all times. The hole can be percussion drilled, preferably as an off -line activity, and the drilling should be very carefully monitored by a tunnel inspector or a geotechnical engineer. Penetration rate and the quantity and colour of the water return should be recorded on a probe hole log. Sudden changes in penetration rate will indicate the presence of hard or soft zones while deviations from normal water quantity and colour may indicate a water bearing fault or fracture zone. When serious water problems are indicated, it may be necessary to stop the tunnel advance and to probe the rock mass ahead of the face with one or more diamond-drilled boreholes. This probing is particularly important if a major fault zone which acts as a water barrier is to be traversed. This type of problem can only be solved satisfactorily if there has been sufficient advanced planning, by both the engineer and the contractor, to ensure that an agreed course of action has been mapped out and that appropriate equipment has been mobilised before the fault is exposed.

When water problems ahead of the face are anticipated as a result of probe hole drilling, a controversy can sometimes arise between the engineer and the contractor on whether to drain or to grout. Resolution of this problem depends upon an understanding of the fundamental difference between these two processes.

Drainage is an effective method when water pressure is likely to induce instability in the rock surrounding the tunnel. For example, a large fault zone containing substantial quantities of gouge and clay can act as a water barrier. If this fault is exposed in the face, the water pressure difference across the fault may be sufficient to cause a 'blowout' of the fault into the tunnel. In such a case, drainage of the water trapped behind the fault is an obvious means of reducing the water pressure and hence the potential for instability. This drainage can be achieved by drilling a number of probe holes through the fault or, in extreme cases, by advancing a small pilot tunnel through the fault. In this example, grouting, assuming that it could be achieved in a clay-filled fault, would be the wrong treatment since it would inhibit drainage and increase the chances of instability.

Grouting is an effective treatment when water quantity rather than water pressure and instability is the major problem. For example, when tunnelling through water bearing limestones or dolomites, a heavily jointed zone may be found to contain large quantities of water. If this fractured zone is connected to a large water reservoir within the rock mass it may be impossible to drain the zone without inducing major changes in the regional groundwater system. In this case, grouting ahead of the face in order to isolate the tunnel from the surrounding rock mass would be the correct solution.

## Blasting control

> 'Blasting for underground construction purposes is a cutting tool, not a bombing operation'.

This quotation from a paper by Svanholm et al (1977) emphasises an important factor in drill and blast tunnelling - the quality of blasting can have a major influence upon the amount of damage inflicted upon the rock surrounding a tunnel. A good tunnel blast is one which results in good fragmentation of the rock within the tunnel, a loose and easily diggable muckpile of limited lateral extent and minimal damage to the rock surfaces around the tunnel. All of these results can be achieved in a single blast if sufficient care is taken with the design of the blasthole pattern, the charge distribution and the detonation sequence (Svanholm et al, 1977, Langerfors & Kihlstrom, 1973, Holmberg & Persson, 1980, Hagan, 1980, Holmberg, 1975).

Figure 2 illustrates the results achieved with carefully designed and controlled blasting in a 7.2 metre diameter tunnel in massive gneiss in the Victoria project in Sri Lanka. Details of the blasthole pattern and of the explosive distribution and detonators are give in figure 3 and table 1 respectively. An advance of 3.7 metres per round with cycle times of 8 to 10 hours (depending upon the distance between the face and the loading bays) was achieved using this blasting design.

Figure 2 : Results achieved using well designed and carefully controlled blasting in a 7.2 metre tunnel in gneiss in the Victoria project in Sri Lanka. Photograph reproduced with permission from British Overseas Development Administration and from Balfour Beatty-Nuttall.

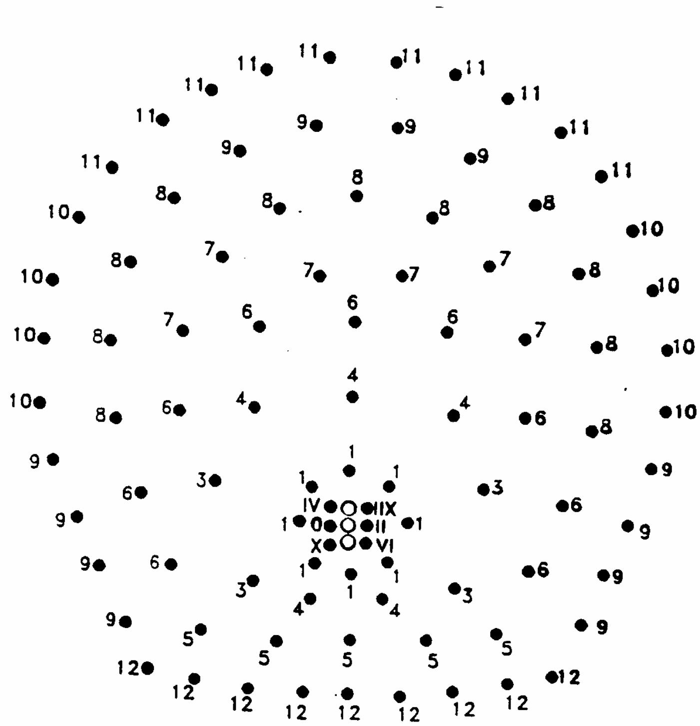


Table 1: Details of explosive distribution for blasthole pattern shown above.

| Holes | no | Dia mm | Explosives | Total wt. kg | Detonators |
|---|---|---|---|---|---|
| Burn | 14 | 45 | Gelamex 80, 18 sticks/hole | 57 | Millisec |
| Lifters | 9 | 45 | Gelamex 80, 16 sticks/hole | 33 | Half-sec |
| Perimeter | 26 | 45 | Gurit, 7 sticks/hole and Gelamex 80, 1 stick/hole | 26 | Half-sec |
| Others | 44 | 45 | Gelamex 80, 13 sticks/hole | 130 | Half-sec |
| Relief | 3 | 75 | No charge | | |
| Total | 96 | | | 246 | |

Figure 3 : Blasthole pattern used by Balfour Beatty-Nuttall on the Victoria project in Sri Lanka. Roman numerals refer to the detonation sequence of millisecond delays in the burn cut while Arabic numerals refer to the detonation sequence of half-second delays in the remainder of the blast.

Figure 2 shows that there is minimal damage to the rock surrounding the tunnel. When blasting of this quality is achieved it may be possible to eliminate most of the support which would normally be required to stabilize blocks and wedges loosened by blasting. Note that the results illustrated in figure 2 may be impossible to achieve in heavily jointed rock masses because of preferential fracture along joint planes.

In the author’s experience, one of the most effective means of ensuring that blasting is adequately designed and controlled on a tunnel project is to specify that the contractor employ a specialist blasting consultant, approved by the engineer, with a separate pay item to cover the services of this specialist. This arrangement ensures that the best advice is available to the contractor and that the responsibility for the blasting remains where it belongs - in the hands of the contractor.

The introduction of computer blasting design services will make good tunnel blasting design available to a much wider range of contractors than in the past and these designs should result in a major improvement in the results achieved in drill and blast tunnels. The advent of computer controlled drilling jumbos, the first of which have come on the market in the past few years, will help in the execution of these blasting designs.

**Rock support**

In rock tunnelling, the choice of appropriate rock support systems is an important aspect of tunnel design and construction. In civil engineering, the philosophy of rock support tended to evolve from the use of steel sets in soft ground tunnelling. At the other end of the spectrum, mining engineers, working in hard rock at greater depth, have tended to use the absolute minimum of rock support with a preference for rock bolting when support is required. With the passage of time, the differences between these two approaches have become less pronounced and the approaches to tunnel support are now very similar in the two industries.

Hoek and Brown (1980) have published a comprehensive review on the use of steel sets, concrete lining, shotcrete and rockbolts for underground excavation support. Space does not permit a full discussion of these techniques here and the following remarks are restricted to the use of rockbolts for tunnel support. This subject has been chosen for discussion because of its importance in modern high-speed rock tunnelling and also because some of the rock bolt types and applications may not be familiar to the reader.

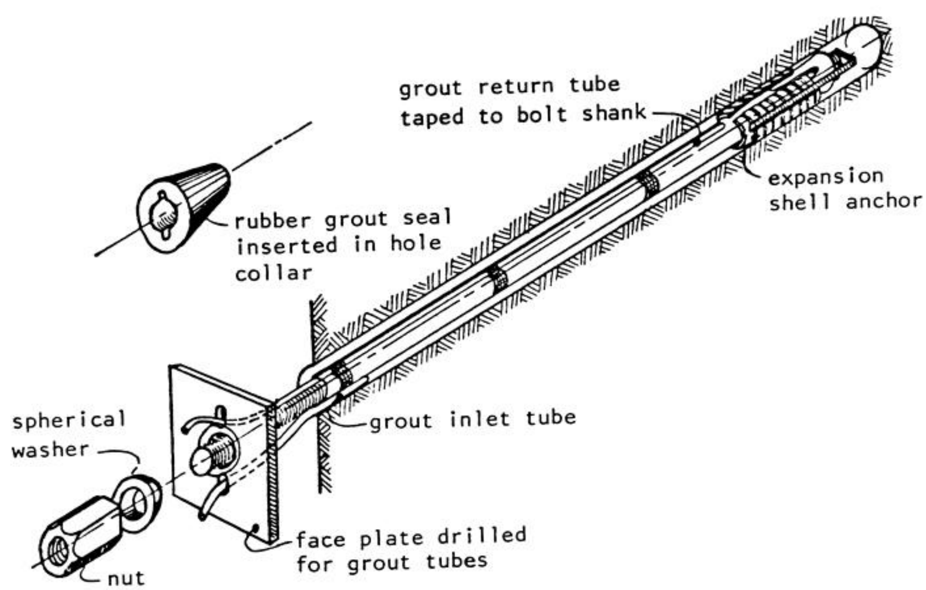


Figure 4 : Mechanically anchored, tensioned and grouted rockbolt system commonly used in civil engineering in hard rock tunnels.

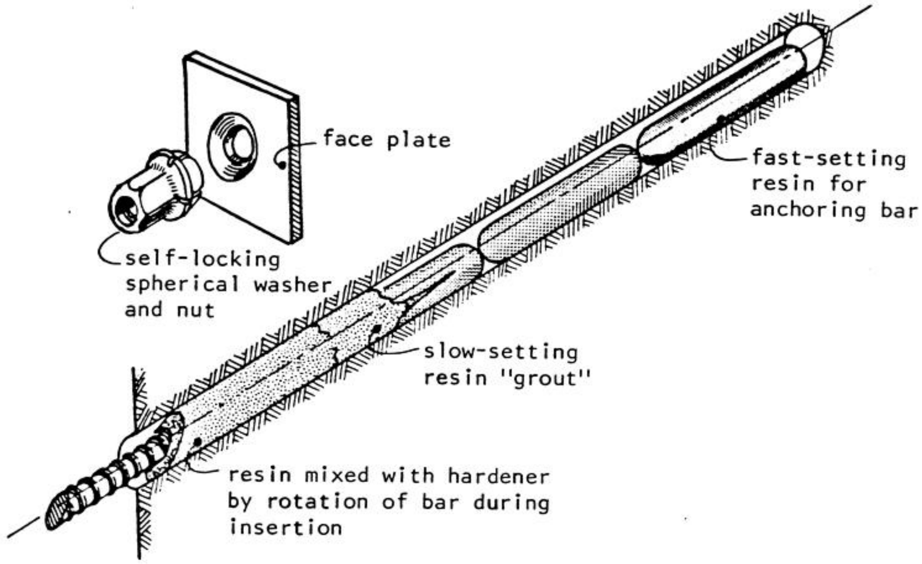


Figure 5 : Resin anchored and grouted rockbolt system which allows very rapid installation and tensioning of the bolt.

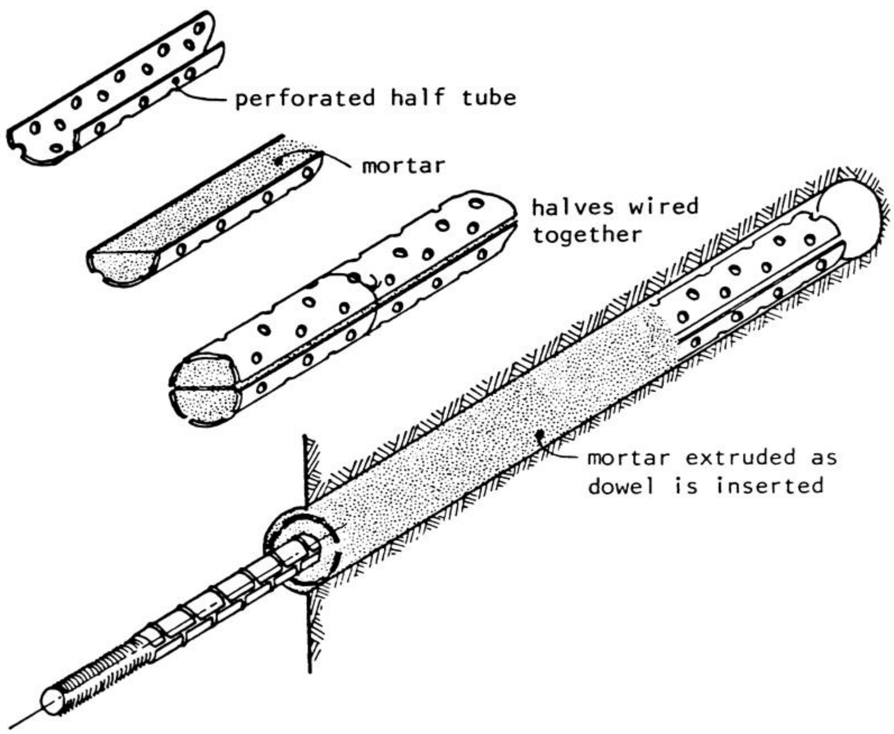


Figure 6 : Swedish ‘Perfobolt’ system of untensioned fully grouted dowels.

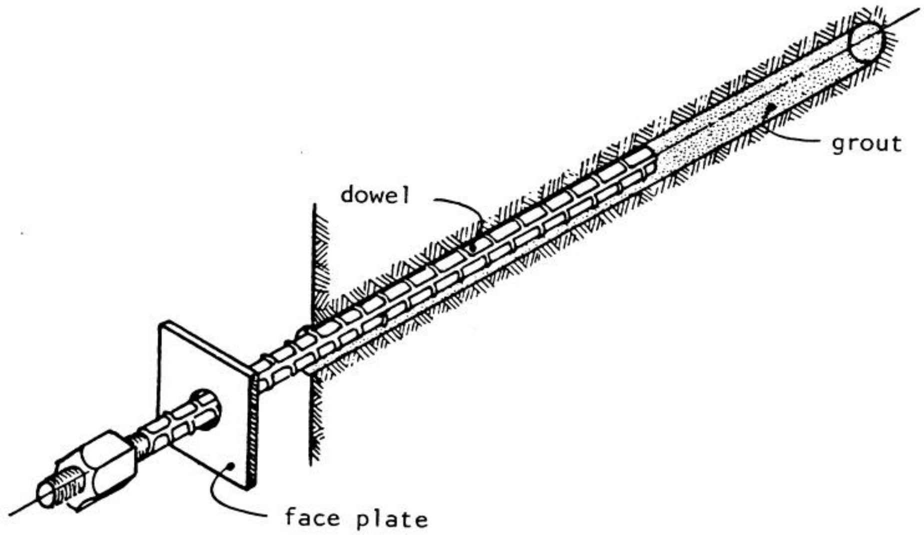


Figure 7 : Simple untensioned dowel installed in a grout-filled borehole.

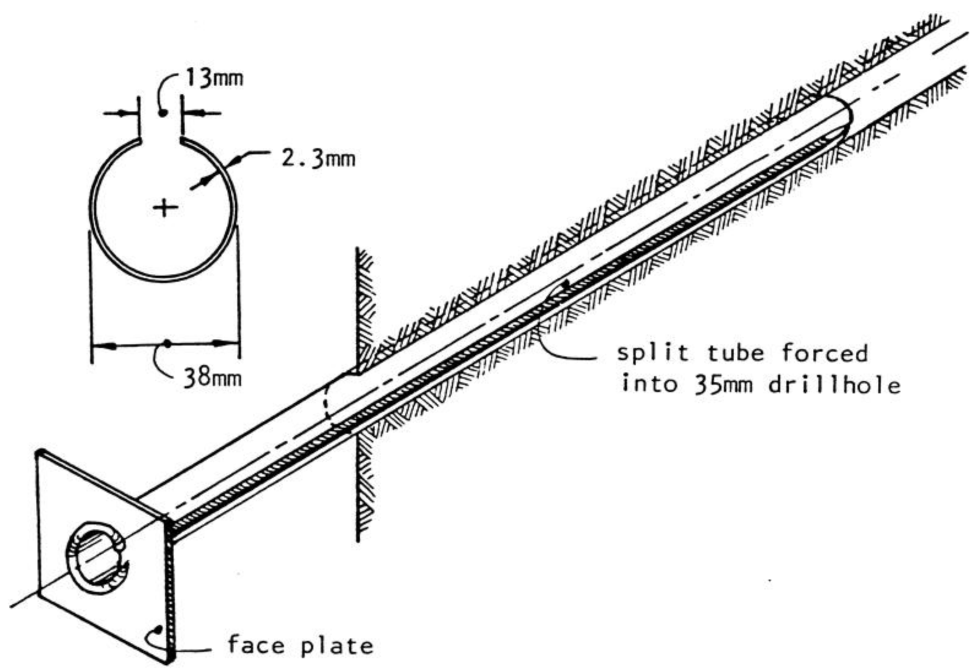


Figure 8 : Split set untensioned rock reinforcement system used by the mining industry for temporary support duties.

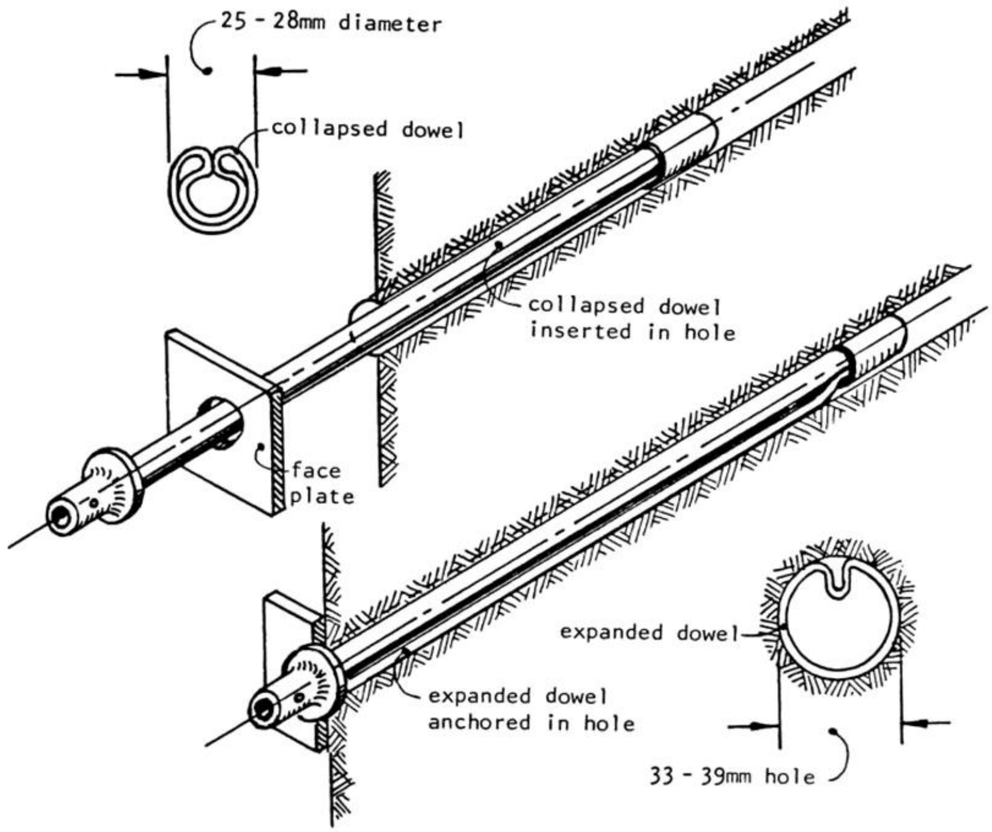


Figure 9 : Atlas Copco 'Swellex' system of untensioned rock reinforcement.

Figures 4 to 9 illustrate a number of rockbolt types which are commonly used in civil engineering and mining applications. A brief discussion on each of the rock bolt systems illustrated follows :

*Mechanically anchored, tensioned and grouted rockbolts* are probably the most common type of rockbolt system used in civil engineering tunnelling. A large number of expansion shell anchor designs are available and these work well in all but very soft rocks. Failure of the system generally occurs in the threaded portions of the bolt, at either the nut or the anchor ends. Grouting of the bolt is important for long term corrosion protection and also to improve the reliability of the system by eliminating the effects of anchor slip. Grouting is usually carried out through tubes taped to the bolt shank as illustrated in figure 4.

Damage to these grout tubes during installation of the bolt is a very common problem and repair of the damaged grout tubes can be a very time-consuming operation. This problem can be reduced by using a hollow shank bolt with the grout return routed through the bolt. The bolt is usually tensioned to about 70 percent of its working load on installation and retensioned to this load just before grouting. Because the installation of the bolt and the final grouting of the system are normally carried out as two separate operations, considerable duplication of effort is involved and the total time required to complete the installations, tensioning and grouting of this type of bolt can range from 1 to 2 hours.

*Resin anchored and grouted rockbolts* were introduced during the past decade as a result of the development of reliable synthetic resin systems which can be introduced into the borehole in sealed capsules. These capsules contain the resin and a catalyst in two separate compartments and mixing of these two components takes place when the plastic capsule is broken by insertion and rotation of the steel reinforcing rod. The chemical composition of the resin and catalyst can be varied to suit different ambient temperatures (including sub zero temperatures) and to control setting times.

In the application illustrated in figure 5, a fast-setting anchor cartridge (setting time about 30 seconds) is introduced into the bottom of the hole and the remainder of the hole is filled with a slow-setting resin 'grout'. Polymerization of the resins is initiated when the reinforcing rod is inserted in the hole and, when the resin anchor has set, the bolt is tensioned. The setting time for the resin 'grout' is about 30 minutes and hence, an hour after installation, a fully tensioned and grouted rockbolt is available for rock support.

The single operation required for the installation of this system should not require more than fifteen minutes (excluding drilling of the hole). Because this very rapid installation permits rockbolting to be carried out in conjunction with on-line activities such as blasthole drilling, this system of rock reinforcement has become very popular in situations in which installation of tensioned reinforcement close to the tunnel face is important. For example, when tunnelling through massive blocky rock in which wedges can be released from the roof or walls if the full tunnel span is excavated without the installation of support, the installation of tensioned rockbolts very close to the face will minimise the potential rock fall problem. In spite of the high material cost involved in using this system, the speed and simplicity of installation and the reliability of the overall system have made it the preferred choice in many tunnelling operations.

*Perfobolt system of untensioned grouted reinforcement*, which is used extensively in Scandinavian, particularly Swedish, underground construction, is a simple system for introducing cement grout into the annular space around a reinforcing rod in a borehole. As illustrated in figure 6, the grout or mortar is contained in a thin cylinder made up of two perforated half-tubes which are wired together. Insertion of the reinforcing rod into this column of mortar results in extrusion of the mortar through the perforations and this 'grouts' the entire assembly into the borehole. Obviously, tensioning of this system is not possible, and, in order to be effective, the Perfobolt must be installed very close to the tunnel face. This question will be examined further later.

*Untensioned grouted dowels* are even simpler than Perfobolts. These untensioned reinforcing elements are used extensively in underground mining. A very thick grout is pumped into a borehole by means of a positive displacement gear pump and the reinforcing rod is simply pushed into this grout column. In up-holes, a small wooden or steel wedge is sometimes used to secure the reinforcing rod in the hole during the setting of the grout. Alternatively, the reinforcing rod is given a very slight bend so that it holds itself in the borehole by a slight spring action. In many mining applications, the faceplate and nut are only installed where they are required to support steel mesh.

*The split set untensioned rock reinforcement system*, developed by Scott (1976) in conjunction with the Ingersoll-Rand Company, is very widely used in the North American underground mining industry. The reinforcing element consists of a thin-wall steel tube of 38mm diameter which is forced into a 35mm diameter drillhole. This spring action of the compressed steel tube induces a frictional force along the length of the tube and this frictional force anchors the reinforcing element in the rock. Provided that the borehole diameter is accurately controlled, this system can be used very effectively to provide temporary support in hard rock tunnels. Installation is very rapid and support performance is good provided that the split sets are installed close to a face and that stresses imposed upon the tunnel are not very high. These devices have not been used to any significant extent in civil engineering tunnels because they cannot be grouted and hence cannot be considered as permanent support. However, where temporary support is required in a tunnel in which a full concrete lining is to be placed for hydraulic reasons, the use of split sets could be considered.

*The 'Swellex' expanded rock reinforcement system*, developed by Atlas Copco, is rapidly gaining popularity in the underground mining industry. This untensioned rock reinforcement system offers many advantages as compared with other untensioned reinforcing systems. As illustrated in figure 9, the reinforcing element consists of a thin walled tube of approximately 42mm diameter which has been folded into a collapsed shape of between 25 and 28mm diameter. This collapsed dowel can be inserted very easily into a borehole of 33 to 39mm diameter and, once in place, is expanded by injection of water at a pressure of about 20 MPa (3000 lb/in$^2$) which is generated by a small portable pump unit. Expansion of the dowel results in an overall length reduction and this pulls the face plate tight against the rock and induces a small tension in the dowel. The anchoring force is very high and the strength of the system is limited by the strength of the tube. Although the system cannot be grouted, rusting is inhibited by the presence of a sealed volume of water inside the expanded dowel and by a protective coating which can be applied to the outside of the dowel.

The types of rock reinforcement illustrated in figures 4 to 9 and discussed in the preceding paragraphs fall into two distinct groups : tensioned and untensioned or, as some users prefer to call them, active and passive reinforcing systems. An understanding of the difference between these two systems is essential if they are to be used effectively in rock tunnelling.

*Tensioned rock reinforcement* is required when it is necessary to apply a force of known magnitude to a rock mass. The most obvious example is to be found in controlling structurally defined failure in underground excavations. If the joint systems in a hard rock mass intersect in such a way that blocks or wedges are released to fall or slide from the roof or walls of a tunnel, a restraining force at least equal to the weight of the block or wedge must be applied in order to prevent failure. In this case, the simplest solution is to install a series of anchored rockbolts and to tension them to specified loads by means of a torque wrench or a hydraulic tensioning device. Grouting the bolts after tensioning ensures that the loads will be maintained in the bolts. Either the mechanically anchored or the resin grouted bolts illustrated in figures

4 and 5 are suitable for this application.

Further examples of the need for tensioned rock reinforcement occur when the installation of rock bolts close to a tunnel face has not been possible or when deterioration of an existing tunnel requires that rockbolts be installed in order to improve the stability of the tunnel. In these cases, practically all of the elastic deformation of the rock mass surrounding the tunnel has already taken place and the only means by which loads can be induced in the bolts is by tensioning.

If the circumstances are such that installation of the rock reinforcement can take place before significant closure of the tunnel has occurred, the use of *untensioned reinforcement* is appropriate. Alternatively, if the stresses in the rock surrounding the tunnel change after construction of the tunnel, as is frequently the case in underground mining, untensioned rock reinforcement can be used very effectively.

*Untensioned rock reinforcement* works because it is anchored in the rock mass before significant movement of this rock has occurred. Once movement occurs as a result of stress changes in the rock, tension is induced in the reinforcement. Provided that the capacity of the reinforcement is not exceeded, further movement of the rock mass will be resisted by the reinforcement and hence, the stability of the tunnel will be improved. With increasing experience and confidence in the use of rock reinforcement systems and with an increasing variety of rockbolts and dowels becoming available, there are now relatively few tunnelling situations which require the use of other forms of rock support. In many cases, the addition of mesh and/or shotcrete to stabilize the near surface rock is all that is required for the permanent lining of an underground excavation.

**Responsibility for support**

In most civil engineering contracts it has been traditional for the contractor to carry responsibility for temporary support while the engineer is responsible for the design of permanent support. This tradition arises from the construction of concrete lined tunnels in which rockbolts and steel sets are used to provide stability and safety during construction and the concrete lining is designed to provide permanent support.

Fewer and fewer rock tunnels are being concrete lined and in many cases rockbolts and shotcrete are designed to provide both temporary and permanent support. In such cases the traditional allocation of responsibility must be modified so that the engineer is responsible for the design of all support while the contractor retains the responsibility for the safety of his men and equipment during construction.

In simple cases in which the geology and rock characteristics are predictable, the engineer may specify a pattern of rockbolts with mesh and shotcrete as required. This support will be installed according to specifications by the contractor. Additional bolts, considered necessary for safety, may be installed by the contractor with payment being approved by the engineer. In order to provide a disincentive for the the excessive use of these safety bolts, payment may be at cost or at some percentage (usually 70 percent) of the cost of specified bolts.

In more complex geological conditions, the engineer may adopt a ‘design-as-you-go’ approach in which support is installed on the instruction of the engineer after inspection of the rock conditions at the face. In this case it is essential that the engineer have a small field team of experienced tunnel engineers with a sound understanding of the mechanics of rock support. Members of this team must be on call to inspect the rock conditions at the face at any stage of the tunnelling operation. It is also advisable that before the commencement of tunnelling, the engineer and the contractor should agree upon a set of guidelines for the types of support to be used for different rock conditions. As in the previous case, the contractor must have the right to install additional support if he considers that this is required for the safety of his men and equipment.

## Conclusion

Tunnelling has always been and will continue to be an engineering activity which is associated with uncertainty and with consequent risk of cost over-runs, litigation and public indignation. There are no simple answers to these problems since, however thorough a site investigation, the rock ahead of a tunnel is unknown until it has been exposed in the face.

In most other forms of engineering, the contractor bids on clear and complete plans and, in general, relatively few surprises will be encountered during construction. Consequently, traditional forms of contract involving turnkey, lump sum or fixed price bids are appropriate. In the case of tunnels, the information available prior to construction is seldom adequate for the use of these types of contract without the inclusion of changed-conditions clauses and other forms of protection for all parties involved. In some cases, the use of completely different types of contract, for example, target or cost-reimbursable, may be a better solution than the use of traditional contracts.

Flexibility in both contract negotiations and in dealing with on site problems is the key to successful tunnelling. If one or more of the parties involved approaches the contract negotiations with preconceived and rigidly held attitudes, it is unlikely that the project will be completed without disputes, claims and perhaps long and costly litigation. Worse still, these preconceived views and rigidly held attitudes can extend to technical matters and can result in the lack of on site technical cooperation and even the use of incorrect remedial measures when technical problems arise.

Both the United Kingdom CIRIA working group (1978) and the US National Academy of Sciences subcommittee on tunnelling contract practices (1976) have emphasised this need for flexibility and cooperation in contract negotiations and in technical activities. It is unfortunate that the problems which prompted these recommendation can still be found on many tunnelling projects around the world. It is hoped that these notes will contribute a small share to the reduction of these problems.

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