Seminar on

Drilling and Blasting for Tunnelling Operations

10 - 11 September 2018, New Delhi

PROCEEDINGS

Border Road Organisation

Corps of Engineers

Tunnelling Association of India

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CONTENTS

Strategy for Site Investigation of Tunnelling Projects

ITA Working Group 2 Research

This study of site investigation for tunnelling projects began with a request from the Executive Council Meeting held in Kyoto, Japan on November, 2001 led by Professor André Assis, former President of the International Tunnelling Association (ITA).

As it is not possible to predefine the ground conditions in detail before a tunnel is constructed geological risks exist on any tunnelling project. The purpose of site investigation is to provide adequate and reliable information in early stages of the project in order to improve the knowledge of the subsoil, assess various design options and choose construction methods that better cope with the identified potential risks.

Site investigations have to be conducted within the global strategy of project risk management (see “Guidelines for Tunnelling Risk Management “, WG2, 2004) and should follow the ALARP (as low as reasonably practicable) principle to reduce risks - namely geological, geotechnical and hydrogeological risks.

The level of acceptable risk as defined by the ALARP principle can be specified in different ways depending on the design stage, and the site investigation strategy should take cognisance of this. The effort required during a site investigation (in terms of the scope of investigation and related cost) will vary with the project development, and has to focus on progressively improving the level of knowledge. The effort required at any stage will depend upon the complexity of the project and will have a direct impact on risk mitigation and project cost.

This document presents the strategy for site investigations based on international best practice, with the aim of maximising the benefit in terms of acquiring knowledge at the right project phase, while avoiding common misleading approaches in terms of investigation effort and responsibility. It is hoped that this document will be a useful guide for future tunnelling projects.

As Animateur and Vice-Animateur of ITA Working Group 2, Research, we wish to acknowledge the important contributions of the following persons: Eric Leca as former Animator and current WG2 Tutor who previously led this study; David Chapman, Elena Chiriotti, Giorgio Höfer-Öllinger and Emmanuel Humbert who drafted the text; all the WG2 Members who contributed to collect the relevant case histories and to finalise the document, the WG2 reviewers Ron Tluczek, Conrad Felice and William Hansmire, and the ITA reviewers Harvey Parker, Amanda Elioff , and Robert Galler. A special thank goes out to Randy Essex, a member of WG3, who gave valuable comments on geotechnical reports in preparation of this recommendation.

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1 >> General

Because the three-dimensional engineering geology for tunnelling and underground projects cannot be entirely defined prior to construction, there are more unknowns, and hence risk, in these projects than those involving superstructures such as bridges and buildings.

The need for good geological knowledge and engineering geology is essential for an underground project, and it should dominate investigations from the very beginning. Geology affects every major decision to be made in designing and constructing a tunnel, determining its cost, and even the performance of the final product.

Careful site investigation is essential to a successful tunnelling project. A thorough programme will not only include collecting and collating all information and data, but evaluating design parameters to be used to assess the project's feasibility, deciding on a reasonable and optimum alignment, designing the ground support and/or lining, and evaluating the construction method and resultant construction program.

More importantly, it should provide a baseline for bidding and predict possible construction difficulties so as to ensure safe and economic performance, and assess the impacts of the tunnel construction on the environment, local residents and surrounding structures.

1.1 SCOPE

The aim of this report is to share and disseminate existing approaches on site investigation for tunnelling projects in order to improve international practices by reviewing and assessing the geotechnical information required for design, while considering environmental and construction issues. This report provides a general guide for site investigation procedures which may be adapted to address the specific needs of each project which may include technical risks, local regulations, contractual framework, etc..

Since in-situ conditions may not be fully defined until they are encountered directly from within the tunnel, the site investigation programme must be phased to match the objectives of the subsequent design phases so that each phase reveals more data on specific uncertainties or queries. This guideline deals with the various phases which are required for site investigations from design stages prior to the start of construction, through to the systematic updating of the geological, geotechnical and hydrogeological model during construction.

The scope of site investigation should not be limited to geotechnical aspects, but must also consider the environment in the locality of the proposed tunnel and identify any associated potential risks.

This report will discuss the benefits of phasing the site investigation, while comparing known conditions vs uncertainties, and the value of additional information versus cost implications. This document sets out a general strategy on how to obtain the required site information to assist the client, engineer and contractor to meet the project goals. As each underground project will have individual requirements, as well as different risks and geotechnical profiles, these general guidelines will have to be developed and adapted to meet the specific project requirements.

The practical and technical details of conducting a site investigation (e.g., the boring methods, sampling methods, methods available for conducting in-situ testing and laboratory testing, the interpretation of the data and how to characterize, classify and analyse the various parameters obtained from the site investigation) are not covered by this document and the reader is advised to refer to specialized technical standards and books.

2 >> Key Reasons for Undertaking a Site Investigation

2.1 PURPOSE OF SITE INVESTIGATIONS

Site investigations should be viewed as an integral part of the risk management process of a tunnel project. Without sufficient data or information from site investigations, the inherent risks in construction and operation of the tunnel or underground works may be unacceptably high. Site investigations should therefore be considered to be the foundation on which the risks associated with the project are identified.

Through each phase of the site investigation for a project, the collected and interpreted data will form the basis for achieving the following design objectives:

- assessment of the technical and economic merits of alternative schemes;
- selection of the most suitable alternative and alignment;
- preparation of an adequate and economical design for the tunnel(s) and underground structure(s);
- selection of appropriate construction methods with low inherent risks;
- identify difficulties or risks that may arise during construction and assess potential mitigation measures
- assess impact on the environment, local residents and existing structures;
- evaluate the re-use or disposal for excavation material;
- predict productivity, schedule and cost;
- predict a geotechnical baseline or reference conditions for bidding.

All site investigations should be initiated by interrogating all existing data with respect to, the history of the site, the predicted geology, existing structures and their foundations, utilities in the area, historical geotechnical investigations, etc.

The information to be obtained should include geology, geomorphology, seismicity, hydrogeology, geotechnical laboratory and field testing results. This information must establish in three dimensions the geological structure, the succession and character of the strata present, the groundwater conditions and the presence of any special hazards. The array of data to be collected will be dictated by the specific construction and performance requirements of the proposed tunnel or underground structure.

An effective site investigation is best achieved by carrying out the work in various phases. Each phase aims to fill gaps in the existing knowledge of the site or to confirm or correct earlier predictions.

A rigidly prescribed programme should not be followed; the philosophy for the planning and execution of the site investigation should be:

a) to decide what information to look for – this will be derived from an appreciation of the geotechnical needs of the project with an understanding of the general geology, character and previous use of the area, compared to the detailed knowledge gained to date;

b) to design the site investigation to provide this additional information utilising the most suitable methods – while being alert to variations or anomalies which may occur that may require changes to the planned investigation.

And last, but not least, the reliability and robustness of the data should be continuously reviewed as new information is obtained, so that the investigation effort is maximised by adapting the programme to the encountered conditions. The detailed knowledge gained at each phase should be utilised to update the ground model and reduce the level of uncertainty, and to plan the scope of further investigations.

2.2 FACTORS INFLUENCING SITE INVESTIGATIONS

The following factors are identified as influencing the extent, reliability and development of Site Investigations:

- **Geology, hydrogeology and geomorphology**

As more complex ground conditions are encountered, extra effort will be required in order to attain a suitable level of confidence in the reliability of the data. This may be hampered in remote areas where in-situ investigation may be difficult to obtain and remote sensing techniques and/or geophysical investigations may be required.

- **Project characteristics**

The scope and focus of a site investigation will be defined by the constraints and geometry of the project (i.e., depth and layout of underground work, tunnel(s) and related ancillary works, such as cross-passages, egress and/or ventilation shafts, adits, galleries, etc.), as well as its locality (i.e. urban or high mountainous regions, complexity of portal or shaft construction and access, etc.).

- **Project use**

Each project will have individual needs as well as a unique risk and geotechnical profile which will dictate specific requirements, e.g. nuclear waste repository, mining exploitation, tunnelling beneath urban environments, etc.

- **Project stage / Investigation phase**

The effort to be put in site investigations has to be consistent with the scope of the project stage. The detailed knowledge gained at each phase of the site investigation should be then utilised to update the ground model in order to plan the scope of further investigations required to reduce the residual level of uncertainty in the next stage of the project.

- **Construction method**

Once appropriate construction method(s) are defined, additional field and/or laboratory investigations may be required to obtain design parameters for mechanised vs. conventional tunnelling.

- **Environmental considerations**

Environmental factors may trigger the type and extent of specific investigations that may be required with regard to the natural environment (e.g., groundwater quality, pollution factors) and/or the urban environment (e.g., noise, air quality, existing buildings, wetlands).

After considering all the above-mentioned influencing factors, at each stage of a specific project, it will be possible to define the optimum scope of investigation required. The level of site investigation required to reach specific goals may vary considerably. Even preliminary studies may require a non-negligible initial investment when the project risk and geotechnical profile are complex and may impact the feasibility of the underground work. Depending on the size and complexity of the project exploratory galleries/shafts may be excavated to achieve a sufficient level of information.

It is the Owner's responsibility to approve the scope of the site investigation and consent to the associated programme and cost. However, contingent factors often exist, which may influence the Owner's decisiveness which will have an impact on the optimum sequencing and effectiveness of the investigations. On the one hand this may be part of the Owner's role and responsibility.

However, on the other hand, the Owner must be fully informed and made aware of:

- the impact that his decision(s) may have on the robustness of knowledge gained;
- the risk related to insufficient investigation;
- the residual uncertainties that will be maintained;
- the level of risk his project will be exposed to.

2.3 STAKEHOLDERS

During the process of development of a tunnel project the following stakeholders are involved:

- **the Owner**
- **the Engineer** as the Owner´s Designer
- **the Contractor,** and his Designer depending on the contractual framework

Third parties, which include:

- owners / Managers of utilities, public underground structures and public surface structures which may be influenced by the tunnel construction;
- owners of land, buildings or housing which may be influenced by the tunnel construction;
- people who live/work within the zone of influence of the tunnel alignment;
- those who may benefit or be disadvantaged during and after construction of the tunnel.

2.4 ROLES AND RESPONSIBILITIES

The Owner, the Contractor and the Designers have different levels of responsibility with regard to the development of a project, and all have to fulfil their obligations and contribute – to different degrees – to the control of the project cost and schedule, and to the preservation of the environment.

The tasks and responsibilities of the different parties involved with the site investigations during the development of a tunnel project will be dependent on the contract model chosen for the project. However, it is recommended that the Owner retains the final responsibility for the ground conditions, irrespective of the contractual framework that is chosen for the project.

As stated in the "Geotechnical Baseline Report for Construction - Suggested Guidelines", ASCE, 2007:
"In traditional contracting, the Owner and his design Engineer will address the full scope of geotechnical investigation and design including exploration of subsurface conditions along the project alignment. Under DB (design and build) method the Owner may seek to transfer the responsibility for portions of this effort to the DB team, whether to achieve schedule efficiencies, transfer subsurface risks, or other reasons.

It is recommended that the same level of exploration be carried out in advance of DB procurement as would be accomplished under traditional method.
To "economize" on the amount of subsurface information provided in advance of DB proposals increases the risk that the Designer will have insufficient information upon which to base a reliable design".

These are strong statements which are generally shared among the technical community and should draw attention to the following aspects:

- The Owner retains the final responsibility for the accuracy of information on the ground conditions.
- The Owner has the final responsibility in approving the extent of investigations to be implemented at each stage of the design, which may be in conflict with what would ideally be required by the Designer. His or her decision has a direct influence on whether additional costs are incurred upfront during site investigations in order to minimise the uncertainties, or whether the costs of the effect produced by such uncertainties on the project will be potentially covered as a provisional sum for risk.
- Generally, it is far more cost effective to carry out the appropriate site investigations at the right project timing, rather than try to make provisions for investigations and risks at a later stage of the project.

In fact, in the former case the majority of uncertainties linked to the ground conditions are resolved prior to construction, which assists in preparation of an economical design and selection of appropriate construction methods with low inherent risks. Adequate information on the ground conditions contributes to the development of a proactive and positive relation among all parties involved in the project and control of the schedule and costs.

In the latter case, bigger residual uncertainties can lead to conservative design approaches, higher provisions for risks, or higher exposure to the risk of contractual claims.

- The Owner should allocate sufficient time, funding and resources to the Engineer to develop and coordinate the investigation programme, to interpret the results of investigations, to assess the residual uncertainties and to develop the design accordingly.
- The site investigation works should remain under the responsibility of the Owner, and they should preferably be excluded from the Engineer's contract. It is recommended that they are carried out through a dedicated bid for execution only. This allows avoiding the following negative effects that could be related to site investigation costs being included in a lump-sum engineering service contract:
  - the bidders for the position of Engineer may propose reduced investigation to remain competitive; as a consequence, the extent and quality of the site investigation could be insufficient;
  - the responsibility for the collected data could be shifted to the Engineer, while it has to remain with the Owner who has to approve and consent to the scope, programme and costs of site investigations.

Any apparent economy in terms of cost and/or technical involvement by the Owner could result in an overly conservative (or even too optimistic) design, bigger residual risks and/or unidentified geological/ geotechnical risks.

- The risk related to ground uncertainties should be properly managed, and may be shared among the Parties, in particular between the Owner and the Contractor. The frequently encountered practice among Owners worldwide of attempting to transfer the total geotechnical risk to Contractors, especially in DB contracts, does not facilitate the proper management of risks and does not liberate the Owner of his final responsibilities. This transfer of geotechnical risk – especially when accompanied by a reduced initial effort in ground investigations – maybe eventually paid by the Owner in terms of either conservative design and/or increased risk of contractual claims, revised design and schedule overruns.

Consequently, the best practices should take into account the following:

- the strategy employed for site investigations should, as far as possible, be independent of the contractual framework;
- information takes time to be obtained and design changes due to late availability of geological and geotechnical data will have more negative impact if they occur in the latest stages of the project;
- a concerted effort should be made to gather the maximum amount of information during the preliminary design stage, with the objective of completing the majority of the investigations prior to commencement of the detailed design stage;
- as the reliability of the data and knowledge of the ground conditions depends upon the amount of site investigations and the quality of interpretation, it is considered prudent to establish an appropriate contractual risk sharing framework (see § 4).

3 >> Strategy for Site Investigation at Various Stages of the Project

3.1 GENERAL

The scope and extent of any site investigation will depend on the status of the project design and on the associated investigation phase. With regard to underground work, the duration of a site investigation campaign from the time it is conceived, through procurement, execution and interpretation is – at each stage of the design – of the order of months to years.

Simple investigations will typically take 3 to 6 months but more extensive investigations can extend to one year or more, depending on the complexity and variability of ground conditions along the tunnel and associated underground structures. In extreme cases the investigation may extend for several years if exploratory galleries/shafts are recommended. Hence, not only the full scope and extent of the site investigation needs to be appreciated, but also its duration within the overall schedule of each project.

The following sections outline typical components, the various phases of site investigations and their purpose.

3.2 COMPONENTS OF SITE INVESTIGATIONS

Typical components of ground investigations are as follows:

- **Desk study**, i.e., literature search and collection of existing information, such as:
  - regional maps (topographic, geological, geophysical, hydrogeological, natural hazards, seismicity, etc.);
  - aerial photos, satellite images;
  - technical literature, studies and existing reports about ground conditions;
  - data related to neighbouring and/or similar projects;
  - existing land use and environmental factors;
  - seismic, climatic, rainfall and hydrological data.
- **Field mapping** and reconnaissance
  - geomorphological mapping;
  - geologic field mapping, geotechnical outcrop mapping, sampling;
  - hydrogeological mapping, water management survey, sampling.
- **Field investigations**
  - direct investigations: trial pits, boring and sampling, in-situ testing (i.e. in-situ stress tests, lugeon or permeability tests, etc.);
  - indirect investigations: geophysical methods, airborne surveys;
  - surveys: topography, building conditions and foundations, utilities, environmental, water wells;
  - monitoring: geotechnical, hydrogeological monitoring, monitoring of existing surface and underground structures.
- **Laboratory tests**
  - identification and classification tests (including mineralogical and petrographic tests, if required);
  - rock / soil mechanical laboratory tests to define strength and deformability properties, time-dependent behaviour, hardness, abrasivity, etc.;
  - hydrochemistry.
- **Exploratory/investigation** tunnel or shaft, which may include field trials for grouting, rock bolts installation, etc.

Further information on the technical details and test procedures for these methods may be obtained from existing standards and references.
Examples of typical information and data that can be collected through the above mentioned components are given in Annex 1.

3.3 PHASED INVESTIGATION OF PROJECTS

The flowchart in Figure 1 demonstrates how the various phases of a site investigation interlink or correlate with the design stages of a tunnel project. Three design phases are considered prior to construction:

- **feasibility studies** (including pre-feasibility, technical feasibility and conceptual design when applicable);
- **preliminary/basic design** (including any designs for permit applications or approvals, when applicable), referred to as preliminary design in the text;
- **detailed/final design**, referred to as detailed design in the text;

Furthermore, specific site investigations can be carried out during the construction stage. More detail on each of these investigations is discussed in following sub-sections.

The flowchart also illustrates the scope of work to be undertaken at each phase of the site investigation, namely:

- **feasibility studies:** to collect enough data to confirm the feasibility of the project;
- **preliminary design**: to determine quantitative characteristics of the ground so that technical solutions may be developed to a point where reliable costs and duration can be established;
- **detailed design:** to reduce the residual uncertainty and inherent risks to a level as low as reasonably practicable.

Since the scope and extent of site investigations depends upon the level of uncertainty and the complexity of the ground conditions, the flowchart gives a basic framework that may be adapted to suit each project profile.

The reliability and robustness of ground model has to match at each phase the design objectives defined in §2.1. This may require an iterative process of data collection, assessment, re-evaluation and redefinition of investigations within the same design phase. In complex projects where exploratory galleries/shafts are required, the results from such investigations become available progressively during the preliminary and detailed design phases, requiring additional design review phases. Although this data is collected for the duration of the design, the exploratory work should be complete prior to concluding the detailed design. In general, the earlier the exploration is made the greater the potential for savings and for cheaper and much better project.

Reference can be made to Annex 2 where various case histories are listed.

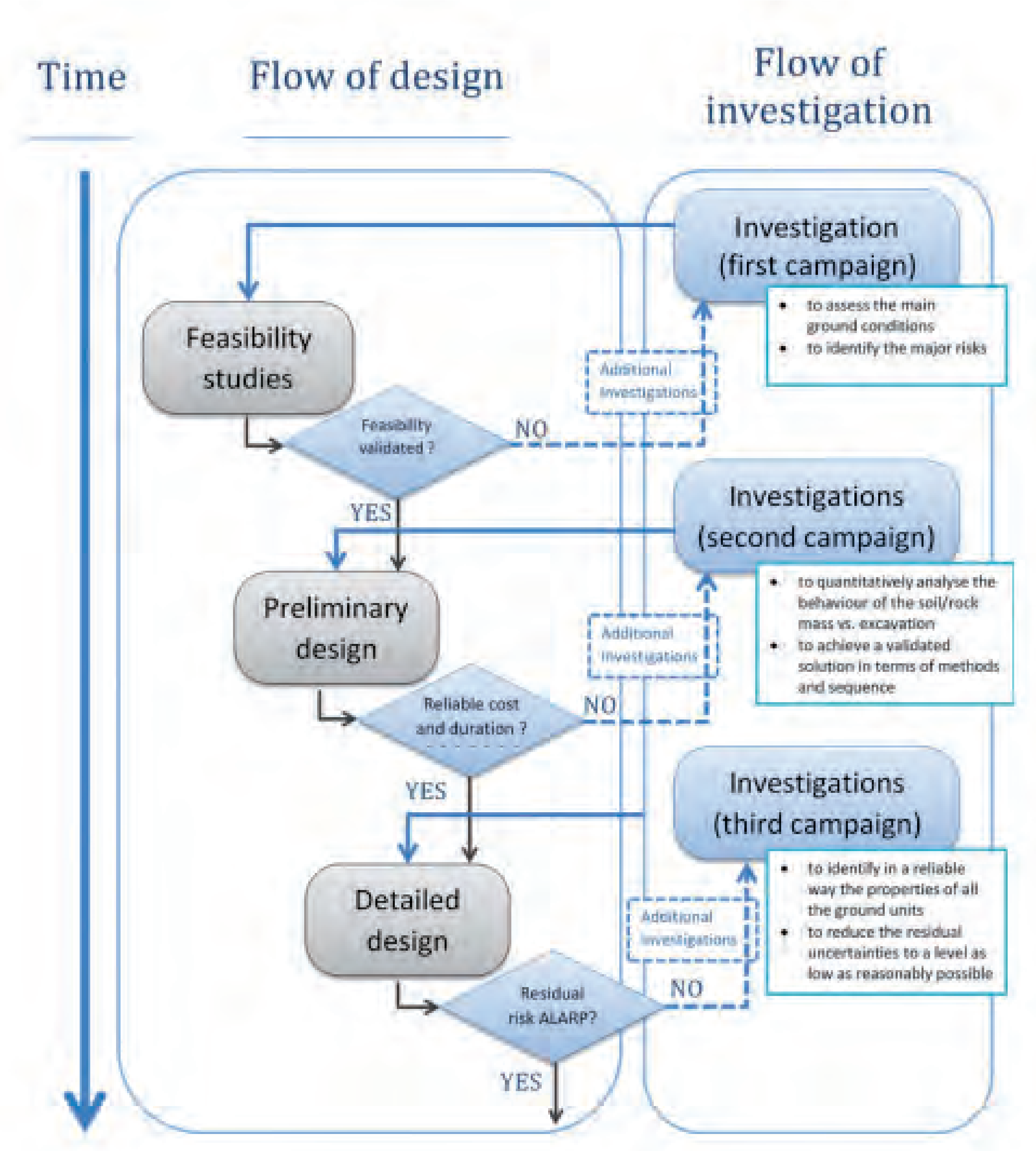

Figure 1 – Recommended phased strategy and scope of site investigations in relation to the design stages of a tunnel project.

3.3.1 Investigations for feasibility studies

Initial studies should be carried out to achieve the following goals:

- to assess the general suitability of the location of the site/tunnel;
- to achieve the best interpretation of the ground conditions based on existing data;
- to assess the technical and economic merits of alternative alignments and their respective ground conditions;
- to make conceptual level estimates of cost and schedule;
- to identify major risks and/or fatal flaws and propose a Risk Register;
- to assess the ground conditions and risks, if any, which could determine the feasibility itself.

3.3.2 Investigations for preliminary design

Design investigations should be carried out to achieve the following goals:

- to develop a 3D model of geological conditions which quantitatively characterises the ground and the hydrogeological regime to a level that permits:
  - selection of the most suitable alignment;
  - preparation of an adequate and economical design together with preliminary cost estimate;
  - selection of appropriate construction methods with ALARP inherent risk, including predicting the behaviour of the ground versus excavation method, determining the different temporary support classes and their distribution along the tunnel alignment, together with a possible range of variation, and to design the ancillary works and portals;
- to define the extent of the zone of influence and to estimate the impact this may have on adjacent structures or land forms;
- to quantitatively identify the risks, to assess their impact on the cost and potential delays to the schedule, and to decide on design measures to reduce the risk ;
- to give a reasonable range of probable cost and duration;
- to assess the level of residual uncertainty so that the need for additional ground investigation can be identified;
- to provide information for the EIA (Environmental Impact Assessment), depending on the legal requirements.

Project stage	Expected results	Investigation means
Feasibility study	Geological and hydrogeological maps. Natural risk map, when appropriate. Longitudinal geological profile. Longitudinal geotechnical and geomechanical profile with the qualitative identification of ground behaviour classes and the identification of the major hazards (with qualitative assessment). Preparation of Risk Register.	Regional topographic, geological, hydrogeological/ groundwater, seismic hazard maps. Information from field surveys and/or adjacent similar projects. Geophysics may provide useful information. Limited site investigations to confirm extremely critical geological or groundwater conditions (e.g., faults, karsts, aquifer , if needed.

Table 1 – Investigations for feasibility studies.

Project stage	Expected results	Investigation means
Preliminary design	Longitudinal geological profile (1:5000 to 1:2000). Longitudinal geotechnical-geomechanical profile (1:5000 to 1:2000) with the quantitative characterisation of ground behaviour classes and identified hazards. Geological and geotechnical cross-sections at the portals (1:500 to 1:200) Geological and geotechnical sections at access/ ventilation shafts Preliminary characterisation of the hydrogeological regime. Update of Risk Register	Geophysics and boreholes at the portals and shafts Boreholes along the alignment. Water sources and groundwater monitoring. Laboratory tests. Outcrop and surface mapping In situ stress measurements and permeability tests, when appropriate. Exploratory galleries / shafts, if needed.

Table 2 – Investigations for preliminary design.

3.3.3 Investigations for detailed design

Design investigations shall be carried out to achieve the following goals:

- to reduce the residual uncertainty to a ALARP level;
- to plan and execute the field and laboratory investigations to confirm the geotechnical and hydrogeological properties of the various ground units;
- to develop a reliable 3-dimensional geotechnical and hydrological model so that the construction method(s) can be validated and justified by calculation and detailed in terms of specifications; to obtain the full set of design parameters (including their potential range of variation) in order to finalise the dimensioning of all elements of the design;
- to achieve a final, accurate assessment of cost and duration;
- to update the risk register, re-assess the level of residual risk, and confirm mitigation measures in order to reduce the non-acceptable risks to a ALARP level;
- to identify requirements for the collection of additional geological, hydrogeological and geotechnical information during the construction phase, including the necessary full scale field trials, if any.

3.3.4 Investigations during the construction phase

At this phase, investigations should be carried out for the following purposes:

- to validate the 3-dimensional geotechnical and hydrogeological model using face mapping, investigations ahead of the tunnel face (e.g. probe drilling, geophysics), TBM performance data, etc.;
- to monitor the ground, ground support and groundwater behaviour;
- to systematically update the 3D ground model in order to predict ground and groundwater behaviour in the subsequent section to be excavated, and to adjust the design / construction method(s) accordingly;
- to analyse the excavated material and assess its potential re-use, or spoil characteristics taking into account environmental constraints;
- to record the condition of structures/ buildings that may be affected by the excavations, and to monitor ground movement and settlement.

3.3.5 Further use of investigation results

The results from all phases of the investigation should be collected, centralised and maintained during construction and, in some cases, during the operation of the facility. During construction, the data should be reviewed to verify design assumptions and to assist in contractual issues, if needed. At a later stage, the data may be utilised when modifications or upgrades are to be implemented or when problems are realised in the maintenance or operations. This data would be continuously updated with monitoring data from geotechnical and environment instrumentation.

It is recommended that a GIS-based model be established to organise and store the project "geo" data in a geo-referenced system. This is especially important for complex projects where a significant amount of information is generated and validated data have to be quickly available and shared among different stakeholders. It is advisable that the Owner initialises and maintains the system throughout the life cycle of the project.

Project stage	Expected results	Investigation means
Feasibility study	Longitudinal geological profile (1/2000 to 1/1000).	Additional boreholes both at the portals and along the alignment.
	Longitudinal detailed geotechnical and geomechanical profile (1/2000 to 1/1000) with the quantitative characterisation of ground behaviour and support classes, identified hazards, distribution of support sections and controls during construction.	Laboratory and field tests.
	Geological and geotechnical cross-sections at the portals, shafts and along the tunnel (1/200 to 1/100)	In specific cases/locations, geophysics may provide useful information.
	Definition of detailed set of design parameters and their variability.	Excavation of experimental sections along the tunnel alignment, if needed.
	Detailed characterisation of the hydrogeological regime.	Continue the monitoring program of water sources and groundwater
	Update of the Risk Register	
	Specifications for investigations during construction	

Table 3 – Investigations for detailed design.

3.4 REQUIRED SITE INVESTIGATION EFFORT

Tunnels demand for a comprehensive investigation which requires considerable time and expenses. Adequate site investigations play a fundamental role in implementing a global strategy of project risk management (see "Guidelines for Tunnelling Risk Management ", ITA WG2, 2004). In addition, the insurance industry requires that the project to be insured is covered by comprehensive and adapted investigation (ITIG, 2012).

Consequently, the Owner has to be aware that investigation should be planned on the basis of needed information and not on the basis of cost, and that sufficient time and budget have to be devoted. Economies on the investigation phases could apparently save time on the design and/or tender schedule, but would generally not allow to achieve the best and most economic project, to define a proper share of risk when setting the contractual conditions between the Owner and the Contractor, and to improve the control of project cost and schedule during construction.

As already mentioned, site investigations should be executed in various phases, and conceived as an iterative process with specific goals at each stage.

At the beginning of the project, generally the ratio of knowledge gained to effort expended is high. Field mapping and desk studies are relatively inexpensive and yet they yield much information. The "knowledge vs. cost curve", shown schematically in Figure 2, is therefore steeper at this stage. Consequently, this phase is of paramount importance and it should be provided for at the very beginning of the studies.

During the preliminary and detailed investigation phases (e.g. with core drilling, field and laboratory tests, etc.), there is still a lot of very important information obtained for tunnel design and risk management. Although the cost to obtain this information is higher than in the previous stage, it makes a significant contribution to improving the reliability of the knowledge of the ground conditions. This phase is therefore vital to the development of the project. The corresponding cost generally ranges between a few percent of the cost of the project construction. Case histories of site investigations for tunnels in the U.K. indicate that the cost for this phase of the investigation is generally less than 3% of the construction cost and may exceptionally go as low as 0.5%, generally depending on the overall cost of the project.

However, it should be borne in mind that the higher the risks of a project and the more complex the ground conditions, the more money will have to be spent to gain reliable data. Investing less than 1% in site investigations at the preliminary design stage is generally considered to be risky.

On completion of a large or major project, a budget for the site investigations of about 3% (potentially increasing up to 8-10% depending to the complexity and depth of the underground work, the need for exploratory galleries/shafts, or the use – such as nuclear or hazardous waste) of the project construction cost should be considered as normal.

The U.S. National Committee on Tunnelling Technology (USNC/TT) in 1984 recommended that "expenditures for geotechnical site exploration should be increased to an average of 3 % of estimated project cost for better overall results". In addition, in case of urban tunnels "The level of exploratory borings should be increased to a level of 1.5 linear feet of borehole per route feet of tunnel alignment for better overall results".

The data collected in Annex 3 give some references which support the percentages mentioned above.

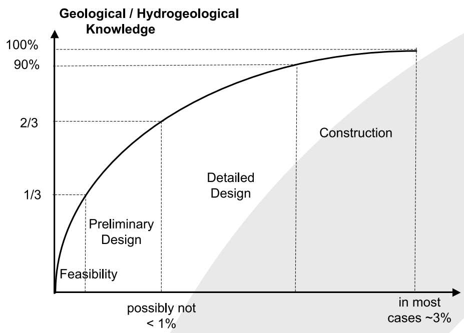

Figure 2
Schematic knowledge vs. cost curve

As requested by the ITIG Guidelines (2012), "the Ground Reference Conditions shall be issued to tenderers as integral and formative information on which tenders shall be based and the Client shall take responsibility for the information so issued […]. Ground Reference Conditions […] shall form part of the Contract and shall provide the basis for comparison with ground conditions encountered in relation to those assumed and allowed for at the tender stage by the Contractor. The Ground Reference Conditions shall provide the baseline against which encountered conditions can be assessed and compared. The Ground Reference Conditions shall also identify hazards appropriate to the site and ground conditions established from the investigations to permit associated risks to be assessed and catered for at time of tender, consistent with the Contract Documentation requirements".

Hence, the results of site investigations have also to be used for allowing the Contractor to bid and for defining contractual conditions. The following sections illustrate the principles on which investigation results are used form a contractual point of view in the international practice.

4.1 INTRODUCTION

The factual and interpreted data collected during the various phases of the site investigation will have varying degrees of significance when utilised in contract documentation. Thus when considering the needs of a site investigation, one not only has to consider the technical content on which the tunnel design and construction will be based, but also how this information will be utilised in contracts and in the procurement process, in particular on how the geotechnical risk is handled.

Past experience gained from major construction projects, especially tunnelling projects, has highlighted some fundamental principles:

- the integrity and reliability of all types of factual information ("data") has to be maintained throughout the life cycle of the project;
- interpreted information from desk studies, or interpretations made from the factual data gained during the project's site investigations must be distinguished from the factual data;
- whatever the method of procurement or the form of contract, geotechnical risk is best managed when the knowledge of the subsurface has been adequately developed before contracting construction services (whether traditional contracting forms, lump-sum, fixed price design or build contracts), and when an agreed model of ground conditions is introduced and made contractual.

An agreed ground condition model provides a sound basis for negotiation in case of changed conditions and this is formalised in different countries in various ways (Geotechnical Baseline Report in the Anglo-Saxon approach; Plan de Management des Risques in the French approach; etc.).

4.2 SITE INVESTIGATION REPORTS

Examination of worldwide practice indicates that four types of reports are generally produced, each having its own specific function. These are namely reports providing:

- factual data (e.g., Factual Report or Geotechnical Data Report, in the Anglo-Saxon approach; Cahier des Données Factuelles, in the French approach);
- interpreted information in terms of geotechnical behaviour (e.g., Geotecnical Interpretative Report or Geotechnical Memoranda for Design, in the Anglo-Saxon approach; Mémoire de Conception, in the French approach);
- the contractual reference for the geological, hydrogeological and geotechnical model (Geotechnical Baseline Report, in the Anglo-Saxon approach; Mémoire de Synthèse, in the French approach; etc.;);
- data collected during construction (Post-Construction Geotechnical Report, in the Anglo-Saxon approach; Dossier de Suivi Géotechnique d'Exécution, in the French approach; etc.;).

It is necessary that the first two documents are completed and/or updated at each phase of the project. The relevance of each report will depend on the contractual framework adopted for the project. This will vary from country to country and examples are given in Annex 4.

Factual data

The report should contain only factual information, data and objective considerations that have been gathered during the different stages of a Project. This report does not include engineering interpretations. The data contained in this report underpins all the other reports. This report often becomes a Contractual Document. Note that factual data include boring logs and soil/rock classifications which are prepared by experienced professionals.

The factual data report includes:

- the list and extracts of all the geological maps used;
- the description of the site exploration programme (dates, localisation, methodology, description of procedures employed, etc.);
- groundwater information;
- the logs of all borings, trenches, and other site investigations;
- the results of all field investigations and laboratory tests (in many cases the data may come from processed laboratory test results, following standard procedures; the final calculated test value is considered a factual information);
- the reported experience of any exploratory gallery/shaft/adit, if existing;
- the references of the bibliography used and the sources of information that provide relevant data (data from similar works, regional geological literature, history of land use, etc.);
- plans and sections indicating summarised borehole information and geological structure.

Interpreted data

These reports include subjective considerations and comments by the Geotechnical Team, in accordance with his understanding, critical evaluation and interpretation of the factual data. The interpretative report presents the geotechnical and engineering interpretation of the data and defines the parameters characterising the geotechnical/geomechanical behaviour of the ground and its variability. This report may be part of the bid package but is not given the status of Contractual Document.

The interpretative report addresses project related issues, it highlights possible impacts on the adjacent facilities and potential problems as well as risks for the various design options and construction methodologies; it indicates the requirement for further site investigations or observations before or during construction.

The interpretative report can also include design analyses, such as rock-mass interaction analyses where ground characterization is used to predict the ground behaviour, response, and support requirements.

Contractual baseline data

Specific contractual documents have to be produced when contracting the project. The geotechnical documents are generally presented in the form of baselines upon which a tender would be prepared and risk sharing would be agreed. As such, in the Anglo-Saxon approach the Baseline Report is a contractual document and is meant to be as objective as possible.

The report states the anticipated (or to be assumed) ground conditions to be encountered during underground construction upon which bidders may rely. Risk associated with conditions consistent with or less adverse than the baselines are allocated to the Contractor, and those materially more adverse than the baselines are be accepted by the Owner.

It establishes the envelope of geological, hydrogeological and geotechnical knowledge relevant to the project, defines the expected geotechnical conditions and highlights all the identified uncertainties. To the maximum extent possible, baseline statements are best described using quantitative terms. Qualitative descriptions, if required, should be clearly defined.

Data collected during construction

The Post-Construction Geotechnical Report (or similar in other national approaches) is intended to form a final record of all "geo" information gathered during the course of the project. It will also constitute a living document into which all future monitoring results are included and any modifications to the project are recorded.

The report should ideally include the following:

- as-observed records of geology and ground conditions;
- monitoring results both during and post-excavation (i.e. groundwater levels, deformation measurements, survey, etc.);
- records of all investigations carried out during construction, including probe drilling and monitoring of performance;
- a record of construction experience, incidents and expedients;
- a full set of site investigation reports, plans, sections, and other records and documents, kept for reference purposes;
- as-built records of the structure, including boreholes and temporary excavations, and of subsequent alterations made in the course of repairs or modifications.

5 CONCLUSIONS

Good geological knowledge and engineering geology are of paramount importance for the successful execution of a tunnelling project. The required information can be obtained from a well executed site investigation program which includes collecting and collating all information and data as well as evaluating design parameters.

Site investigation provides important information that is required for reducing the risks associated with tunnel construction and constitutes an essential component of modern tunnel engineering. As such, site investigation should be viewed as one key component of the global strategy for project risk management in terms of reducing geological, geotechnical and hydrogeological risks.

This document has been prepared by Working Group 2 of the ITA, and aims at consolidating updated information on key aspects of site investigation principles and practices that may assist stakeholders in their approach to tunnelling projects. The document, which is based on international best practices, can be used as a general guide for the site investigation strategy which may be adopted to address the specific needs of each project.

Working Group 2 would welcome comments from users, as to the contribution of this approach to serving Member Nations needs and facilitating the dissemination of site investigation knowledge and general practice at an international level.

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ANNEX 1 >> Elements of a Site Investigation

The following is a list of important elements associated with a tunnel project, and on which Site Investigations should be focused:

1. **Topography**, status of the land usage and accessibility conditions
2. Location and condition of **existing surface** structures, such as buildings, and underground structures such as basements, foundations, utilities, pipelines, etc.
3. **Water** use, water rights and water management requirements
4. **Accessibility** to the investigation site(s)
5. **Geomorphology**
   a. Soft rock fillings of valleys and glacial / glacigenic relicts
   b. Landslides and deep-seated gravitational slope deformations
   c. Rockfall, mudflow, avalanches, flooding etc.
6. **Field geology**
   a. Geological model – a three dimensional model of strata, folding, faults, joint characteristics (ground conditions)
   b. In-situ stress conditions
   c. Geological data relevant to design and construction method(s)
7. **Fault zones** and related characteristics
8. **Seismology**
   a. Neotectonic regime
   b. Active faults
   c. Volcanic zones
9. **Hydrogeology**
   a. Aquifers, aquitards and aquicludes (extension, geometry and properties)
   b. Groundwater levels and related seasonal changes
   c. Discharge of groundwater and flow direction
   d. Drainage network: main receiving water, feeders
   e. Water balance
   f. Chemical and physical water properties
   g. Karst phenomena and sinkholes
10. **Contamination**
    a. Natural, such as hazardous gases
    b. Man-made
11. **Geothermal activity**
12. **Radioactivity**
13. **Geotechnical** and **geomechanical** characteristics
14. **Meteorological** and climatic data
15. **Excavation** material
    a. Reuse (e.g. lithology, grain distribution, etc.)
    b. Disposal (asbestos contents, etc.)
16. In case of **immersed** or **under sea** tunnels, the followings should also be investigated
    a. Depth of water
    b. Tidal conditions including current and wave condition
    c. Navigation and ship traffic condition.

01 KUHTAI HYDROELECTRIC POWER PLANT, AUSTRIA

• PROJECT IDENTIFICATION

Location
Kühtai, Tyrol, Austria, Europe

Construction period
Scheduled: 2014 - 2017

Owner
TIWAG – Tiroler Wasserkraft AG
Eduard-Wallnöfer Platz 2
A- 6020 Innsbruck

Designer(s)
Technical Design: TIWAG
Geological Layout: Geoconsult ZT GmbH

Contractor(s)
Still not defined
Investigation gallery: ALPINE Bau GmbH

• GENERAL PROJECT DESCRIPTION

TIWAG – Tiroler Wasserkraft AG plans to extend the Sellrain-Silz HEPP that has been in operation since 1981.

This extending contains the construction of an additional reservoir with further water supply lines from the central and eastern Ötztal valley and the upper Stubaital valley and will result in a considerable improvement of the present energy production in the project area. Central features of the power plant are:

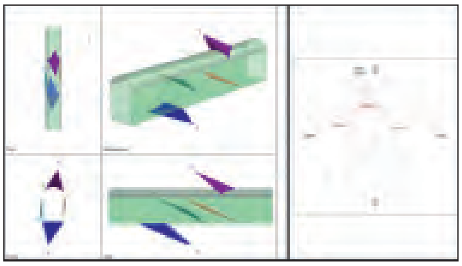
- reservoir in the upper Längental valley with an available storage capacity of about 31.1 million m and a dam height of about 113 m (rockfill dam with a clay core),
- the power plant Kühtai 2 with an output of 140 MW, connecting plant reservoir with the existing Finstertal valley reservoir
- and the 25.5 km water supply line from the upper Stubaital valley to the plant reservoir.

The addition of the Kühtai 2 power plant to the existing Kühtai pumped storage hydro power station is to be achieved by constructing a headrace tunnel between the Kühtai and Finstertal Valley reservoirs.

The turbine building is located entirely in a cavern at a depth of around 175 m in the right-hand side of the valley where the future abutment of the dam will be situated.

The additional water catchment area extends from Fernaubach brook in the upper Stubaital valley to Fischbach brook and Winnebach brook in the central Ötztal valley. The impoundments are situated at approx. 2,090 to 2,410 m above sea level.

• TUNNEL CHARACTERISTICS



Total Tunnel Length
25,5 km (headrace drive),
~ 15 km (all other tunnels like access tunnel, penstock and caverns)

Boring diameter
see below

Overburden(min-max)
30 – 1.063 m

Characterization scheme
NATM

Excavation type
NATM, TBM (see below)

Contract model
B2203-1, B2203-2

Headrace
25.5 km, Ø 4.2 m, 30 to 1063 m, TBM

Penstock
1.225 m, Ø 4.8-5.8 m, NATM (possibly TBM)

Penstock
375 m, Ø 6.1-6.7 m, NATM

Cavern of Power Plant
83.000 m³, NATM

Drainage gallery
700 m, Ø 5.5-6.0 m, NATM

Investigation gallery
735 m, NATM

Cavern
(Length-Whith-Height) 64 x 31.5 x 50 m

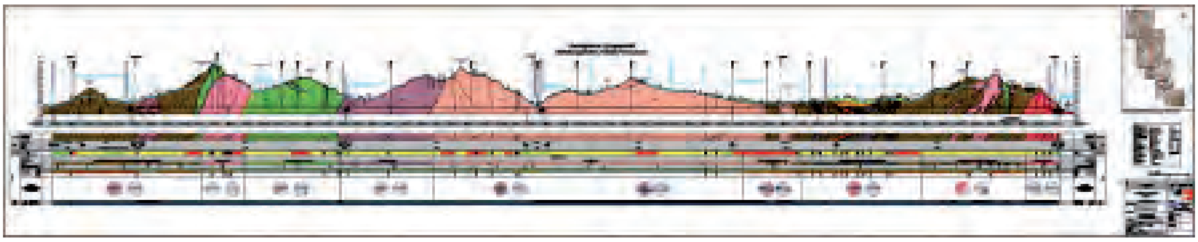
• ENVIRONMENTAL AND GEOLOGICAL CONDITIONS

The project area between Kühtai and the upper Stubaital valley is located in the north-western Stubai Alps and is predominantly high alpine in character. The area under investigation is almost exclusively above 2,000 m above sea level, the highest peaks in this area reaching over 3,000 m, parts of this area are glaciated.

In geological terms, the project area lies in the Ötztaltal-Stubai Crystalline Complex. Orthogneisses and paragneisses predominate in the region of the planned structures, as well as migmatites, amphibolites and mica schists. The Ötztal valley complex is bounded to the East by the Brenner Line and extends northward to the Inntal valley. It forms the border of the Engadine Window in the West and is intersected by faults and fracture zones in the South. In the South, the Ötztal-Stubai Crystalline Complex extends without interruption to the Periadriatic Lineament.

Morphologically, this alpine region is characterised by glacial erosion. The pronounced cirques indicate the previous extent of the glaciers. Massive rock glaciers and moraines also testify to the earlier glaciation. Almost all tributary valleys and cirques have deposits resulting from glaciation recession

• **GEOLOGICAL PROFILE**



Geological longitudinal section of the 25.5 km headrace gallery.

• **SITE INVESTIGATION TARGETS**

Geological Setting
- Bedding
- Quarternary soils covering the hard rock mass
- Permafrost related structures like rock glaciers

Ground Types / Characteristics
- Types of gneiss, migmatites, amphibolites
- Joint spacing

Structural Geology
- Orientation of joints
- Folding
- Fault zones and orientation

Fault Characteristics
- Geometry
- Filling

Alteration / Weathering

Hydrogeology

Geothermal Situation

In-situ Stress
- At the cavern location

Gravitative Mess Movements
- Deep landslide in a near-to-slope situation of the headrace gallery.
- Possible landslide in an abutment situation of the proposed dam (which figured out as stable rock mass with the investigations).

• **MEASURES**

Desk Study
- Feasibility study
- Studies from existing constructions (former HEPP's)
- Studies of regional geological literature
- Orthophotos
- Laserscan Images

Mapping
- Site visits of the headrace galleries of the existing HEPP
- 1:10.000 geological mapping all over the surface (ca. 110 km²)
- 1:5.000 geological mapping at reservoir site
- 1:2.000 geological mapping at water impoundments and dam site
- 1:10.000 hydrogeological mapping all over the surface
- 1.5.000 laserscan image geohazard process mapping

Drillings
- 26 core drillings from surface
- 11 core drillings from exploratory tunnel (at cavern site)

Geophysical Methods
- 21 seismic profiles
- 5 geoelectric profiles
- geophysical borehole tests in all drillings (acoustic / optical borehole image)

Field Tests
- Trial pits
- Lugeon tests in boreholes
- Lefranc tests in boreholes
- Pump tests in boreholes
- SPT tests in boreholes in soils
- Boreholes have been developed as monitoring wells (standpipes)
- One borehole has been developed as inclinometer
- At cavern site (from exploratory tunnel):
  - Radial press (two tests) in 2 m diameter caverns
  - Dilatometer tests in boreholes
  - Hydro fracturing test in borehole
  - Lugeon tests in boreholes

- Hydrogeological field measurements (discharge, temperature, electrical conductivity)

Laboratory Tests
- Soil tests (186 samples): Grain distribution
- Rock tests (74 samples): Modal analysis (thin sections)
- Water analysis (ion balance, stable isotopes, Tritium)

Exploratory Tunnel
- 1 exploratory gallery ~ 735 m at cavern site (realized in 2010/2011)

Monitoring
- Hydrogeological monitoring at springs, gauges and monitoring wells
- Inclinometer
- Geotechnical monitoring in exploratory gallery

02 GOTTHARD BASE TUNNEL, SWITZERLAND

• PROJECT IDENTIFICATION

Location
Switzerland

Construction period
1993 - 2016

Owner
AlpTransit Gotthard Ltd (until 2016)
Swiss Federal Railway (operator)

Designer(s)
Lombardi Engineers Ltd.
Amberg Engineering Ltd.
Pöyry Ltd.
Gaehler & Partner Ltd.
Rothpletz Lienhard Ltd.
Gruner Ltd.
CES

Contractor(s)
Murer / Strabag Implenia / Frutiger / Bilfinger Berger/Pizzarotti Implenia/Hochtief/Alpine/ Impregilo

Engineer's)
See Designers

• GENERAL PROJECT DESCRIPTION

The Swiss New Rail Link through the Alps (NRLA) is creating a fast and efficient railway link. Its core piece is the 57.1 km long Gotthard Base Tunnel, the longest railway tunnel of the world when it will start the commercial operation in 2016. The new railway link crosses the Alps with minimal gradients and wide curves at only 550 metres above sea level creating the first flat railway through the Alps.

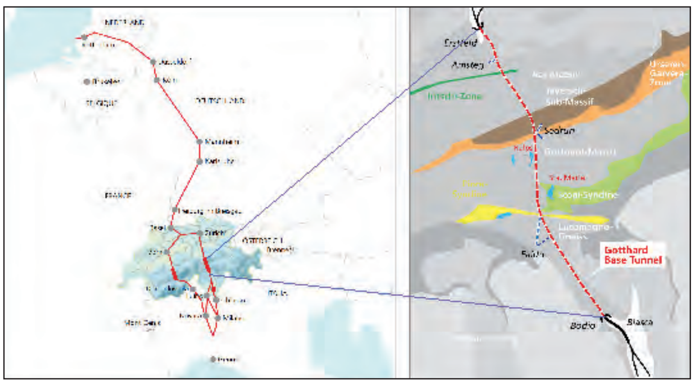
The flat railway allows efficient rail transport of goods as well as shorter journey times in national and international passenger traffic. The new routes cut passenger travelling times substantially. The new Gotthard route is a high-speed rail link. Passenger trains can traverse it at maximum speeds of up to 250 kilometres per hour. Nevertheless the main purpose of the new railway infrastructure is to shift a major part of the heavy transalpine goods traffic through Switzerland from the road to the rail.

The Gotthard axis of the NRLA is Switzerland's largest-ever construction project. With construction of the new Gotthard rail link, the country is implementing one of Europe's largest environmental protect ion projects.

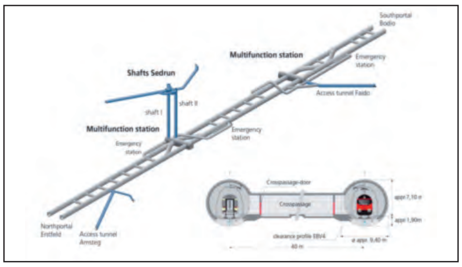
The Gotthard Base Tunnel consists of two 57-kilometres-long single-track tubes. These are connected together every 312.5 metres by cross passages. Including all cross-passages, access tunnels and shafts, the total length of the tunnel system is around 152 km. It joins the north portal at Erstfeld to the south portal at Bodio. With a rock overburden of more than 2300 metres, the Gotthard Base Tunnel is also the world's deepest railway tunnel constructed to date.

Two multifunction stations at Faido and Sedrun divide the two tubes into three approximately equally long sections. The multifunction stations each contain emergency stop stations and two track crossovers. In case of an incident such as a fire in the train or a fault in the Gotthard Base Tunnel, whenever possible the affected train travels out of the tunnel into the open air. If this is not possible, the driver stops the train at an emergency stop.

For construction purposes, the Gotthard Base Tunnel was subdivided into five main sections. Access adits provided access to the underground construction sites for workers, materials and machines. To save time and costs, construction work proceeded on the various sections simultaneously. For construction of the Sedrun section, access from the surface was through a 1-kilometre-long horizontal access tunnel and two 800-metres-deep vertical shafts. From there, the two tubes were blast-driven to the north and south. Because the deep overburden in bad ground conditions (squeezing rock) high stresses threatened to deform the tunnel on a distance of 1. Kilometres. Special supporting means were necessary in this zone. The engineers developed an innovative new concept with flexible steel rings (TH-profiles), which partly closed under the rock pressure. The rock pressure could finally be to a technically manageable degree reduced by allowing large deformations.



• Tunnel characteristics



Total Tunnel Length
Nominal length 57.1 km
System length 151.8 km

Boring diameter
8.8 / 9.4 / 9.5 / 11

Overburden(min-max)
100 – 2'350 m

Characterization scheme
2 single track tubes, connected with cross links every 312.5
2 multifunction stations
3 acceess galleries
2 vertical shafts (800 m)
1 bypass gallery
1 inclined ventilation shaft

Excavation type

TBM	98.1 km
Conventional	53.7 km

Contract model
Unit price contracts for civil work based on design bid build approach

The Gotthard Base Tunnel crosses the Alps in mainly hard crystalline rock masses, with a high uniaxial strength and a brittle failure mode.

Weak ground conditions with a ductile failure mode were expected on less than10 per cents of the total tunnel length.

The main ground related hazards were:

- rock fall, caused by the joint systems
- rock burst, mainly in zones of high overburden
- convergences or high rock pressure
- face instabilities
- combined scenarios.

Nearly two thirds of the total length of the 151.8 kilometres long tunnel system were excavated with TBM's.

One third of the total length was excavated by application of conventional tunnelling methods.

Logistics for was the main challenge for the contractors.

Depending on the general construction schedule railway (inner lining in parallel to the excavation) or conveyer belt systems (inner lining followed the excavation) were used for the transportation of the muck.

One of the biggest stories of success was the use of the spoil for the production of concrete aggregates of the rock support and the final lining. 100% of the concrete gravel for the tunnel construction has been produced from excavated rock material with origin from the TBM-drives and the conventional drives. No quality failure related to the concrete aggregates occurred on he entire inner lining.

The environmental requirements were generally fulfilled on a high level, also in the eyes of the public and the environmental organisations.

The Swiss Federal Office of Transport required in the project specific standards a lifetime of 100 years for the civil work. No major rehabilitation work with significant operational limitations is allowed during this time.

The solution to achieve this high requirements was a double lined tunnel with the provisional rock support as outer lining (first lining) and the permanent, in the minimum 30 cm thick, inner concrete lining (second lining).

• Environmental and Geological Conditions

The Gotthard Base Tunnel crosses the following main tectonic units from north to south:

- **the Aar massif**
- **the Gotthard massif**
- **and the pennine Gneiss Zone**

The Aar massif and the Gotthard massif are the backbone of the Swiss Alps. Both massifs consist mainly of gneisses and granites. These rocktypes showed generally a brittle failure mode. Under special circumstances squeezing was observed in the crystalline rock masses.

Younger sedimentary rocks are wedged in between the three main tectonic units. Some these rock masses are massively fractured, especially in the Tavetsch intermediate massif. In this rock mass types the phenomenon of squeezing was observed on a distance of 1 km.

The main ground related hazards were:

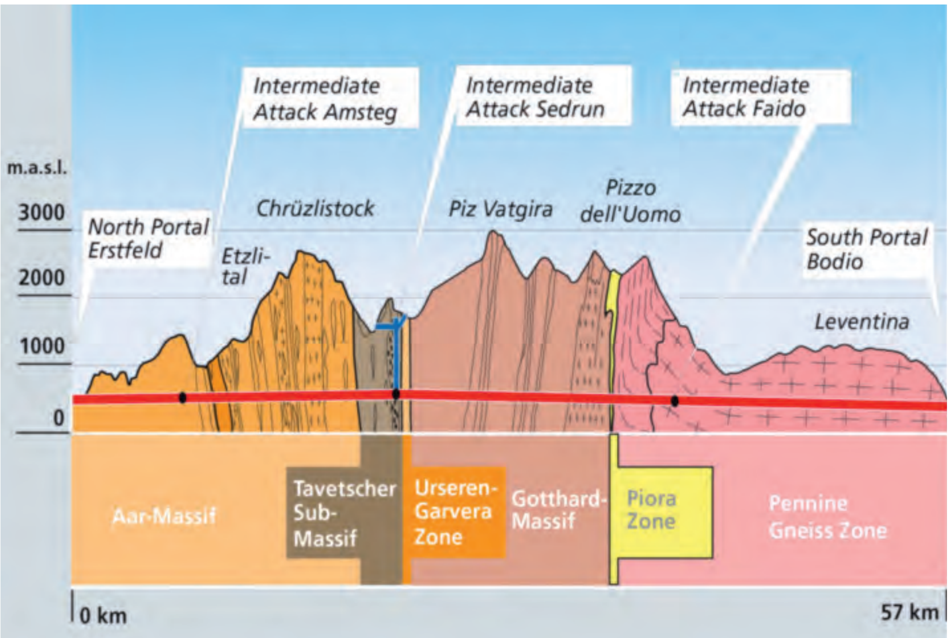
- rock fall, caused by the joint systems
- rock burst, mainly in zones of high overburden
- convergences or high rock pressure
- combined scenarios

The tender design assumed that more than 90% of the excavation could be done in good ground conditions without any bigger difficulties.

High rock or ground water temperatures and high initial ground water pressures caused by the high overburden had to be taken into account.

A maximum groud temperature of around 50°C was expected (highest temperature measured 46°C)

• **GEOLOGICAL PROFILE**



Geological longitudinal section of the 25.5 km headrace gallery.

• **PROJECT STAGE(S)**

Construction period 1993 - 2016

Prefeasibility Study
1. / 1993

Feasibility Study
2. / 1995

Variant Study
3. / 1989

Authoritie's Permissions Project
1. / 1995 – 1999 for 4 of 5 main lots
2. / 1995 – 2006 for 1 lot

Tender Design for Owner
1. / 1997 - 1999

Tender Design for CC
2. / -

Post Contract respectively Construction Design
3. / 2001 - 2014

Other
4. / none

• **SITE INVESTIGATION TARGETS**

Geological Setting
• Tectonic situation

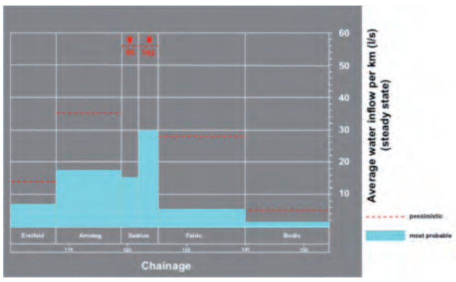
Ground Types / Characteristics
• Highly diverse rock mass types had to be traversed during the construction of the Gotthard Base Tunnel.
• They range from the tough Gotthard granites, through the highly-stressed pennine gneisses of the Leventina, to soft rocks of the Tavetsch intermediate massif.

Structural Geology
See geological profile

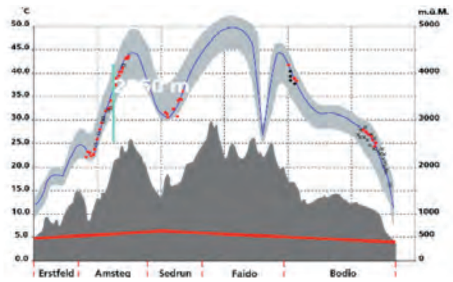
Fault Characteristics
• Kakeritic faults
• Ductile shear zones (mylonites)
• Brittle fault zones

Alteration / Weathering
• No special effects

Hydrogeology
• Forecast of probable water inflows with high pressure and high temperature (steady state)



Geothermal Situation
• Forecast of ground temperatures with numerical model



In-situ Stress
• in direct correlation to the overburden
• horizontal stresses similar normally in the same magnitude as the vertical stresses

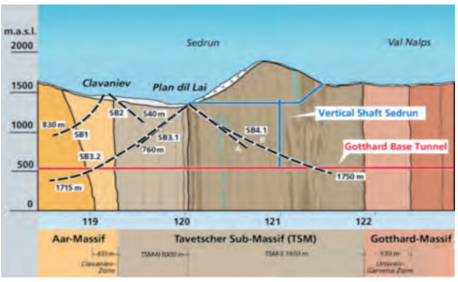
Gravitative Mess Movements
• no

• **MEASURES**

Desk Study
- yes

Mapping
- yes

Drillings
- yes, long inclined core drillings in the Tavetsch Intermediate Massif



- extended core drillings in the Piora Zone (see below)

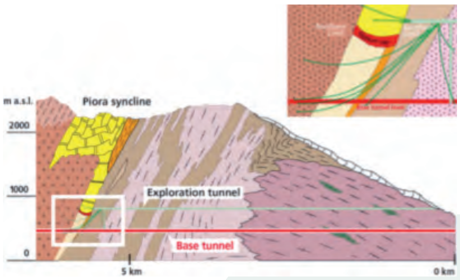
- systematic exploratory drillings (percsussion drillings) during the excavation in the conventional drive and the TBM-drives, mainly in both tubes
- core dillings in special cases (squeezing rock zones and during the excavation close to the Nalps concrete arch dam)

Geophysical Methods
- yes, in few special cases with only limited information for the excavation due to the inhomogeneous ground conditions

Field Tests
- bore hole tests
- various in situ tests in order to classify the muck for its reuse

Laboratory Tests
- yes, mainly triaxial tests, abrasivity tests
- various in situ tests in order to classify the muck for its reuse

Exploratory Tunnel
- yes, Piora exploratory system, tunnel of 5.3 km length



Monitoring
- yes, 3 D deformations (in all drives)
- extensometers in special cases
- monitoring of surface deformations during 15 years throughout the whole year (also winter time!) in order to detect dangerous deformation trends to the nerby concrete arch dams

© figures from AlpTransit Gotthard
text AlpTransit Gotthard and Heinz Ehrbar

03 CITYRINGEN, DENMARK

• Project Identification

Location
Copenhagen, Denmark

Construction period
2011 – 2018

Owner
Metroselskabet I/S

Contractor(s)
Copenhagen Metro Team I/S

• General Project Description

The Cityringen project is a new fully underground metro system with 17 stations, 2 crossovers and three construction and ventilation shaft structures interconnected by 2 single track tunnels of 16.5 km in length.

The geology in the Cityringen project area is characterised overall by a typically 1-5m thick fill layer. This is underlain by typically 10-25m thick quaternary layers. These deposits are highly variable and comprise a recognised sequence sand/gravel and clay till layers. At a number of locations extensive meltwater sands and gravels are deposited directly on the limestone, in particularly in the northern part of the alignment, whereas in the south-western part of the alignment the quaternary layers mainly consist of clay till.

The quaternary layers are underlain by Copenhagen limestone. The limestone is uniformly bedded, with extensive flint beds and bioturbated zones. The upper 0-4 metres of the limestone is locally glacially disturbed and heavily fractured.

• Tunnel characteristics

Total Tunnel Length
33 km

Boring diameter
5.8 m

Overburden(min-max)
35 m

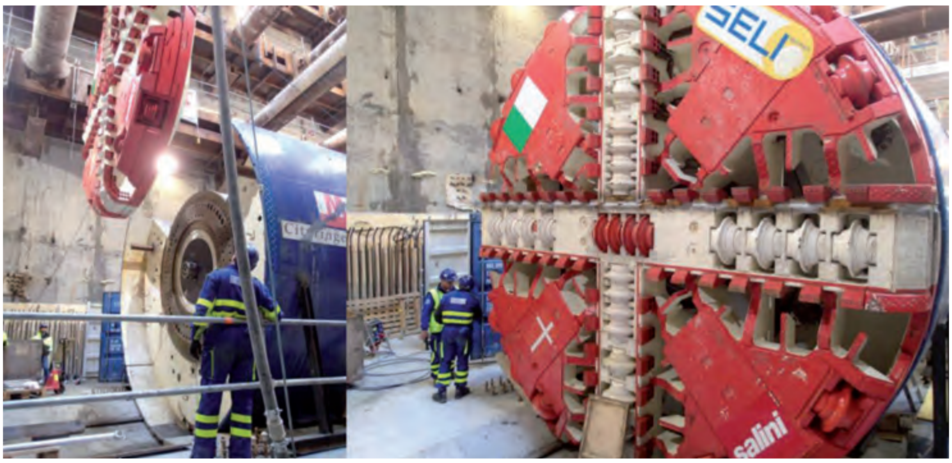
Excavation type
Earth Pressure Balance TBM

• Environmental and Geological Conditions

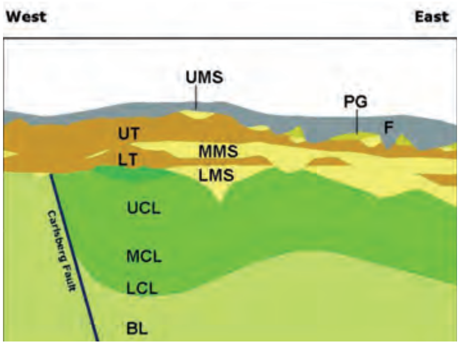
Geologically the project area is featuring 2-5m of fill layers. This is underlain by 10-25m of quaternary layers, mainly consisting of glacial till and meltwater sand and gravel. The meltwater deposits are highly variable, consisting of fine-grained sand and coarser-grained sand and gravel, often with larger boulders. The coarse sediments usually occur in the lower part of the meltwater units and may possess very high permeabilities.

The quaternary layers are underlain by Copenhagen Limestone from the Danian period. The limestone is fractured to a varying degree, however, it is mostly severely fractured in the uppermost few meters. The induration and fissuring in the limestone is generally highly variable.

The eastern part of the Cityringen alignment passes the inner city of Copenhagen where many buildings are old and sensitive to variations in groundwater levels. For this reason the municipality of Copenhagen has in this area prohibited any groundwater lowering outside the construction zones unless appropriate measures are taken to keep the groundwater level within natural limits. The western part of the alignment passes through a catchment for domestic water supply at Frederiksberg where a key issue is protection of the groundwater resource in terms of quantity and quality, with chemical parameters of interest being salinity/chloride, nickel and sulphate. Numerous contaminated sites - typically originating from former dry-cleaning shops, petrol-filling stations and mechanical workshops - are located close to the planned construction sites.



• **GEOLOGICAL PROFILE**



Geology is comprised fill (F) and post-glacial deposits (PG), glacial till (UT - Upper till, LT - Lower till), meltwater sand/gravel (UMS - Upper, MMS - Middle, LMS - Lower) and limestone (UCL - Upper, MCL - Middle, LCL - Lower, BL - Bryozoan)

• **PROJECT STAGE(s)**

Prefeasibility Study

Feasibility Study

Variant Study

Authoritie's Permissions Project

Tender Design for Owner

Tender Design for CC

Post Contract respectively Construction Design

Other

• **SITE INVESTIGATION TARGETS**

Geological Setting
•

Ground Types / Characteristics
• Geology in site investigation boreholes, with the aim of establishing af full geological model along the alignment

Structural Geology
•

Fault Characteristics
•

Alteration / Weathering
•

Hydrogeology
• Identification of depth to flow zones/ waterbearing zones. Estimation of total transmissivity. Groundwater quality

Geothermal Situation
•

In-situ Stress
•

Gravitative Mess Movements
•

• **MEASURES**

Desk Study
• Collection of existing non-project data, e.g. earlier boreholes and pumping tests

Mapping
•

Drillings
• Project investigations before tender:
• 130 geotechnical boreholes (shell&auger in quaternary layers, core drilling in limestone) at stations, shafts and along the alignment.
• 80 hydrogeological boreholes (DTH drilling throughout the entire drilling depth) at stations and shafts, with several screens, for use in pumping tests and continuous groundwater monitoring.

Site investigation method	Approximate number
Borehole	500
Geophysical log, including flow log	250
Short duration pumping test	600
Long duration pumping test	33
Groundwater chemical sampling	350
Seismic survey	13 km
Groundwater level monitoring	250 wells

• In total 500 borings
• Average spacing 40 m
• 13 km seismic survey
• Total length of site investigation borings 17 km
• Cost of site investigations are 2,5% of construction cost

Location of borings along aligenment.

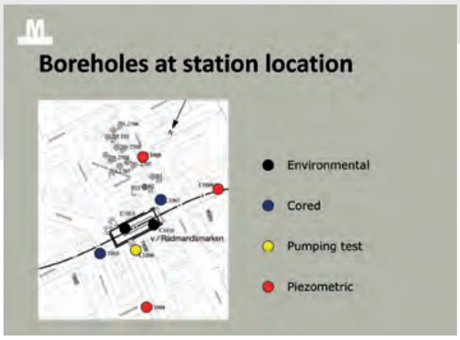
Geophysical Methods
• Geophysical logging in selected deep boreholes, including gamma, density, porosity and flow logs. In some boreholes OATV logs have been undertaken

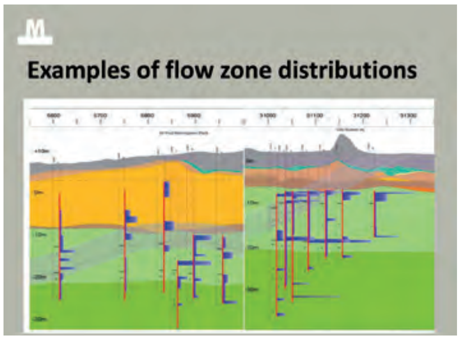
Field Tests
•

Laboratory Tests
•

Exploratory Tunnel
• N.A.

Monitoring
• N.A.





04 PORCE III HYDROPOWER PROJECT, COLOMBIA

• Project Identification

Location

Amalfi, Department of Antioquia, Colombia.

Construction period
2006 – 2012

Owner
Medellin Public Utilities Company (EPM)

Designer(s)
Ingetec, Bogota, Colombia

Contractor(s)
CCC Porce III Consortium:
Construçoes Camargo Correa
Conconcreto S.A.
Coninsa – Ramon Hache S.A.

Engineer(s)
Ingetec, Bogota, Colombia

• General Project Description

Porce III Hydropower Project features a 151 m high, CFRD dam, which impounds 3,756 km² catchment area of the Porce River and tributaries into a 170 hm³ reservoir; a 730 m long open channel spillway with a discharge capacity of 11,350 m³/sec, controlled by four radial gates; a headrace conveyance system, composed of a 12,452 m long upper tunnel, a 159 m long vertical shaft and a 274 m long lower tunnel; an underground power station houses four vertical-shaft Francis turbines coupled to four synchronous three-phase generators, yielding a total 660 MW, or 4,254 GWh/year.

• Tunnel Characteristics

Total Tunnel Length
12,726 m (upper and lower headrace)

Boring diameter
8.5 m

Overburden(min-max)
30 - 550 m

Characterization scheme
Geotechnical characterization was performed using Barton´s Q System, Bienawski's RMR System and

Excavation type
Drill-and-blast

Contract model
Design + Construction

• Environmental and Geological Conditions

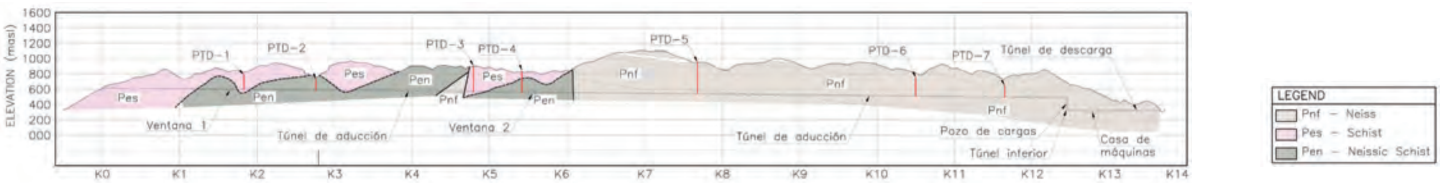
The surface of the area where the project´s headrace tunnel was excavated is covered 80% by residual soils and colluvium deposits. The lithologic units present correspond to Paleozoic rocks constituted by schist of varied composition and quarz-feldspar and aluminic gneiss. The rock is folded in a northerly direction along the tunnel alignment and is affected by faults, joints and shear zones.

The predominant geomorphologic units along the tunnel alignment correspond to High Mountain Schist and High Mountain Gneiss. Schist is composed of quartz, mica (sericite, muscovite, biotite and hornblend) and graphite, whereas neiss is composed of both, quarzic feldspar gneiss and aluminic gneiss. The main geologic structures defined correspond to a series of tight folds with a general N-S direction: the Castillo and Primavera Faults, and El Roposo shear zones.

The headrace tunnel was excavated along the left bank of the Porce River, through metamorphic rocks composed 11% by schist, 36% by gneissic schist and 53% by gneiss.

The project was developed within the deep canyon environment of the Porce River, which runs north through a tropical rain forest, were wildlife is abundant, including a wide variety of birds, reptiles and mammals. The owner was therefore quite demanding regarding the preservation of the pre-existing natural environment and stringent regarding the restotation of the environment affected by project construction.

• Geological Profile



Geological longitudinal section of the 25.5 km headrace gallery.

• SITE INVESTIGATION TARGETS

Geological Setting

The general geological setting of the project corresponds to metamorphic rocks (basically neiss and schist), of paleozoic age, highly weathered at the surface, mostly fresh and competent at depth (at tunnel level).

Ground Types / Characteristics

Ground types were defined in the technical specifications for bid as well as for construction and contractual purposes. The headrace tunnel involved basically five types of ground:

- Type I: competent, hard, massive, slightly fractured, stable rock, where excavation may advance without the need to install support, and only localisez shotcrete and/or bolts could be required;
- Type II: moderately hard to hard, moderately folded, fractured to moderately fractured rock, in which spalling over time may occur and therefore support is required;
- Type IIIA: medium to low strength, folded, fractured to highly fractured rock, which discontinuity planes are altered, and spalling may occur at the excavation face, therefore immediate installation of support is required;
- Type IIIB: friable and/or crumbly material, fault or shear zones composed of gouge or highly fractured material, including residual soil in the portal area;
- Type IIIC: higly fractured rock, cohesionless, where excavation shall be performed in three stages or sections: upper, mid and lower. Squeezing phenomena expected.

Structural Geology

Along the headrace tunnel alignment, the main geologic structures correspond to a series of tight folds, with a general N-S direction, El Castillo and Primavera Faults, and La Primavera and El Reposo shear zones.

A series of faults, with orientation N20° - 25°E are El Roble anticline, El Roble sincline, the Hondoná anticline, and El Totuno sincline. These folds affect the quartzic schists of variable composition.

La Primavera shear zone has direction N15 – 20°E/40°SE, and was defined base don Drillholes PCP-1 AND PCP-2, as well as on seismic refraction conducted during previous studies.

El Reposo shear zone was defined based on drillholes performed during previous studies.

Fault Characteristics

The headrace tunnel is affected by two geologic faults: El Castillo and El Salado Faults. The first is a reverse fault, oriented N40°W, dipping SW, affecting metamorphic rocks. It is located around K4+600 in the tunnel, and at the surface, it is covered by loose rock fragments, along a 100 m wide alignment. The fault material is 10 to 20 m thick and is composed of greenish grey milonite, gneiss and schist fragments embedded in a silty clay and brown sand matrix. RQD varies between 0% and 17%.

El Salado Fault is located 1.3 km east of El Castillo Fault, and crosses the tunnel at about K6+100. The fault´s direction is N20° - N30°W, dipping vertically. It is composed of highly fractured to crushed rock fragments 10 – 20 m wide, and an influence zone of some 100m that narrows with depth.

Alteration / Weathering

The alteration/weathering phenomena was observed in exploratrion galleries excavated in the dam´s left abutment and along the main and access roads. A significant thickness (some 30 m) of moderately to highly weathered rock had to be entirely removed to construct the CFRD dam.

As for the headrace tunnel, such weathered material was evident at the tunnel's and tunnel adits' portals, as well as along the first 30 or 40 meters of tunnel, excavated in poor ground and supported with streel sets.

In-situ Stress

In-situ stress measurements were carried out for the headrace tunnel, prior to commencemenet and during construction. The target of such tests was to investigate the magnitude of the minor principle stress (σ_3) in certain key locations of the tunnel, and compare such values to the internal pressure of the tunnel, in order to determine whether modification in the tunnel alignment or the installation of a steel liner would be necessary, in order to prevent hydraulic fracture phenomena that could generate leakage from the tunnel to the ground surface or into the powerhouse.

Four such locations were selected and hence, four corresponding sets of hydraulic fracture tests were conducted: the first two sets, prior powerhouse and headrace excavation, were performed, respectively, from an exploration gallery that ended near the future powerhouse, and from the surface, at a location of relatively low overburden (110m) due to a topographic depression; the third set, from within the tunnel, near the intersection with the vertical shaft, to check for effective overburden at such elbow; and a fourth set, performed near the intersection of the bottom of the shaft and the lower headrace tunnel, in order to check for effective minor principle stress values as the pressure tunnel approached the underground powerhouse surface.

The results of such tests allowed to optimise the design in the following ways: a) at the location of the low overburden due to the topographic depression generated by the running creek, the original tunnel alignment was displaced some 90m further into de mountain, to gain vertical overburden; b) as regards the elbow's optimum location, hydrofracture tests indicated the need to displace the elbow and the shaft 60 m further upstream, to gain lateral overburden; c) the test results indicated the need to steel-line the full length of the lower headrace tunnel that splices into the powerhouse.

Gravitative Mass Movements

Based on the results of the investigations, the dam´s left abutment required extensive tieback installation, and the tunnel´s adit portals required stabilization measures, including shotcrete, bolts, revegetation with native grass species and adequate drainage.

• MEASURES

Desk Study

Desk studies went through a step-by-step process, according to each stage of the project:

- a conceptual and prefeseability stage, in which different scenarios or alternatives for the project´s optimum location and layout were proposed, during which preliminary geological and geotechnical investigations were conducted;

- a feasibility stage, in which the selected alternative was studied and developed in much further detail, along with a significant number of drillings and other geotechnical investigations were performed as well as corresponding costs;
- a third stage, in which the detailed studies, drawings, technical specifications and contractual documents were prepared for bidding purposes.

All field investigations performes prior to, and during such stage, constituted the necessary parameters for such detailed office design of the project.

Mapping

The project involved topographical mapping and geological mapping. Topographical mapping was used to optimize the location and design of the project along its various stages. Three scales of maps were used: the first two, 1:25000 scale, which was the general scale of the project encompassing the basin, and the 1:10000 scale, covering the reservoir are, were both aerial photograph-based restitution scales; the third scale, 1:2000, was used for all detailed design of project's components, including the dam and appurtenant works, the access roads and the headrace tunnel's portals as well as its three adit's portals.

On the other hand, geological maps corresponding to the above-mentioned topographic maps were prepared, to the same scales, that is, general geological and geomorphological maps of the basin and reservoir areas, and detailed geological maps of the surface and underground works, including the headrace tunnel and powerhouse, for bidding purposes. In addition, a 630m long exploratory gallery, referred to ahead, was excavated, the end point of which was close to the future underground powerhouse which allowed detailed geological mapping of this project component.

During excavation of the headrace tunnel and underground powerhouse, detailed geological maps were drawn of the actual geology encountered, following each blast of the tunnel face, drawn at a 1:200 scale, not only to provide as-built records, but to assist in the design of the tunne's permanent lining: shotcrete, concrete or steel.

Drillings and pits

There were seven drillings performed from the surface along and over the headrace tunnel alignment, spaced between 1.0 km and 3.0 km, (1.5 km on average), depending on the degree of difficulty of the access to each drilling site, plus drillings at the inlet portal, and in two adit tunnels to the main tunnel, thus covering the full length of the tunnel. The following table summarizes the drillings and corresponding lengths

Headrace Tunnel Drillholes	Length (m)	Projected Station In Tunnel
PTD-1	110	K1+575
PTD-2	145	K2+700
PTD-3	340	K4+675
PTD-4	275	K5+400
PTD-5	420	K8+350
PTD-6	250	K11+325
PTD-7	160	K12+375
Adit Tunnel 1 (L=558 m) Drillholes	**Length (m)**	**Projected Station In Adit**
P-V1-1	55	K0+170
P-V1-2	34	K0+080
P-V1-3	15	K0+020
Adit Tunnel 2 (L= 649 m) Drillholes	**Length (m)**	**Projected Station In Adit**
P-V2-1	40	K0+165
P-V2-2	35	K0+070
P-V3-3	23	K0+015
Adit Tunnel 3 (L=705 m) Drillholes	**Length (m)**	**Projected Station In Adit**
P-V3-1	50	K0+240
P-V3-2	40	K0+130
P-V3-3	10	K0+010
Inlet Portal Pits	**Depth (m)**	**Location**
AP-V2-1	3.4	Portal area
AP-V2-2	3.0	Portal area
AP-V2-3	3.0	Portal area

Geophysical Methods

Geophysical methods included seismic refraction lines along the tunnel alignment,

Field Tests

A numer of field tests were conducted in order to establish basic design parameters. The most outstanding tests were in the area of hydraulic fracture, in order to determine the magnitude of the minor principle stress at key locations alongthe headrace tunnel alignment (see above).

Laboratory Tests

In order to establish the statigrapgy and the physical properties of the rock for the headrace tunnel, and as a complement to the field

work performed, a core drilling program was conducted, from which samples were retrieved for a variety of tests in the laboratoty. Lab tests were performed on soil and rock. Soil tests were performed almost entirely on surface samples and on samples obtained form pits excavated in the vicinity of the inlet portal.

Soil tests were basically: Atterberg Limits, specific gravity, hydrometric analyses, water content, Proctor compaction, following ASTM and AASHTO Standards. The table below summarizes such tests.

Rock tests were executed both pon surface as well as on cores retrieved form drillholes. Tests included: Grading of granular materials, compaction, direct shear, tensile strength, wave propagation velocity, triaxial, slake durability and petrography. The table below summarizes the type and number of tests perfomed.

Exploratory Tunnel

In 2004, a 635m long exploratory tunnel, 2.5m x 2.5m, was excavated between the Porce River left bank and the future underground power station, in order to investigate detailed golgical and geotechnical conditions for the powerhouse, regarding its orientation and temporary as well as permanent support requirements.

Monitoring

The design included the installation of a series of instruments for monitoring the behaviour of diferent project components once placed in operation, for long-term monitoring. A set of instruments, among which are inclinometers, piezometers and accelerographs were placed in the dam.

In the headrace tunnel, monitoring during construction included the installation of tape extensometer rings and pressure cells for longterm monitoring.

DATA COLLECTED FROM FRENCH ROAD TUNNELS

The five case histories presented below regards roads tunnels which where design and constructed between 2003 and 2014. Data were collected and actualized in 2010 by CETU, the Centre of Tunnel Studies of the French Ministry of Public Works.

The tunnel construction costs include the civil works only and they are those announced at the bidding stage. No major claims where observed after the completion of the works.

The quantity of site investigations were obtained from the tender documents (factual data reports). To assess the cost of site investigations, boreholes and in-situ test only were considered. The costs of geophysics and exploratory galleries (when present) were initially excluded from the analysis. For the case of Bois de Peu Tunnel, the cost of site investigation is given with and without the exploratory gallery.

Tunnel	Start of the works	Type	Total Length	Cumulated length of boreholes	Cost of explorations / cost of tunnel	Invest. Cost [M€]	Constr. method
Saint Vallier	2002	Road	178 m	225 m	2,6 %	1,26	D&B
Schirmeck	2003	Road	550 + 150 m	704 m	3,7%	1,01	D&B
Bois du Peu	2004	Road	2*600 + 90 m	885 m	2,2% Excluding costs for exploratory gallery	1,09	D&B
Peute Combe	2009	Road	2*600 + 120 m	1219 m	3,85%	0,95	D&B
Saint Béat	2010	Road	110 + 310 m	1586 m	2,1%	1,12	D&B

DATA COLLECTED FROM FRENCH TRAMWAY AND METRO TUNNELS

Tunnel	Start of the works	Type	Total Length	Cumulated length of boreholes	Cost of explorations / cost of tunnel	Invest. Cost [M€]	Constr. method
Tramway C2V	2010	Tramway	1500+90m	1902	N/A	1,20	EPB TBM
Paris Line 4 extension	2007	Metro	460m	380 m	N/A	0,83	D&B
Lyon line B extension	2010	Metro	1470m	2078 m	N/A	1,42	Slurry TBM
Paris Line 14 extension Lot1	2014	Metro	3620m	3475 m	N/A	0,96	TBM
Rennes Line B	2014	Metro	8100m	7679 m	N/A	0,95	EPB TBM

Assumptions on costs [€/m]:
- *Pressiometric borehole: 300;*
- *Subhorizontal and/or inclined core recovery boreholes: 1000;*
- *Vertical core recovery boreholes: 800;*
- *Destructive boreholes: 150*

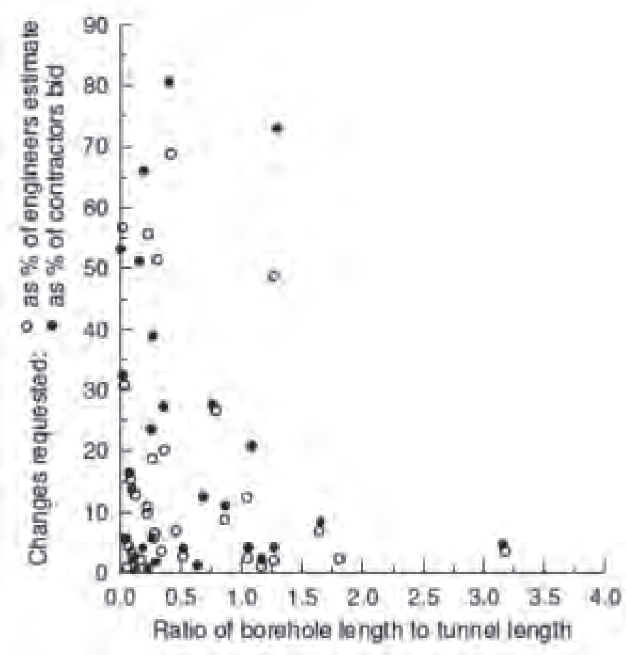
D&B = drill and blast

DATA COLLECTED FROM EUROPEAN LONG AND DEEP TUNNELS

Tunnel	Start of the works	Type	Total Length	Cumulated length of boreholes	Cost of explorations / cost of tunnel	Invest. Cost [M€]	Constr. method
Lötschberg	1994	Railway	34,6 km	N/A	2,8 %	N/A	Gripper TBM / DB
Gothard	1998	Railway	53,9 km	N/A	1,4 %	N/A	TBM / DB
Brenner	2011	Railway	57,0 km	~ 36 km	8,7 % I ncluding exploratory galleries	0,63	
LTF	Detailed design phase	Railway	57,1 km	~ 62 km	8,9 % ncluding exploratory galleries	1,08	
Koralm Base Tunnel	In construction		33 km	~ 21 km	1,9 %	0,64	N/A
Semmering Base Tunnel	In construction		27 km	~ 38,5 km	1,7 %	1,43	N/A

DATA COLLECTED FROM U.S. NATIONAL COMMITTEE ON TUNNEL TECHNOLOGY

Data were collected by the U.S. National Committee on Tunnel Technology (USNCTT 1984) by interviewing the Owners, Engineers and Contractors of 84 different tunnel projects.



Country	Factual Data	Interpreted Data	Contractual Data	Data collected during construction	Ref. document(s)
FRANCE	Cahier des Données Factuelles	Mémoire de Conception	Mémoire de Synthèse	Dossier de Suivi Géotechnique d'Exécution	AFTES (2012), Characterisation of geological, hydrogeological and geotechnical uncertainties and risks, GT32R2A1 NFP 94500 (2013), Missions d'ingénierie géotechniques – Classification et spécifications
KOREA	Geotechnical Data Report				KTA(2015) Standard Specifications for Tunnel
SWITZERLAND					SIA 198 (2004), Construction d'ouvrages souterrains SIA 199 (1998), Etude des massifs rocheaux pour les travaux souterrains
USA / UK (Anglo-Saxon approach)	Factual Report or Geotechnical Data Report	Geotechnical Interpretative Report or Geotechnical Memoranda for Design	Geotechnical Baseline Report	Post-Construction Geotechnical Report	ASCE (2007), Geotechnical Baseline Reports for Underground Construction

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Advancement in Geophysical Investigations for Tunnels

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ABSTRACT:

Modern major construction is inconceivable without high-level engineering explorations, which play a major role in increasing the economic efficiency of capital investments. For the design of structures it is indispensable to procure comprehensive high-quality information about the subsurface, within very short periods. The study of diverse natural conditions predetermines a variety of methods and technical means which can be used for carrying out exploratory work.

Most of the time, while working on tunnels, caverns and other underground projects, decision makers are working with limited and imperfect information. Engineering geophysics is an efficient means of subsurface investigation to fill in the information gaps and provide a complimentary source of information to enhance our understanding of subsurface conditions. The merit of application of this low cost aid lies in its ease of deployment and rapidity in providing a reliable knowledge of the underground over a large area, substantiating the requisite geotechnical evaluation studies thereby. Technological advancements and development of portable digital data acquisition instrument systems have increased the versatility in evaluating underground conditions and site characterization.

The state-of-the-art subsurface geophysical investigations are helpful towards minimizing & optimizing involvement of the conventional direct exploration methods, aiding in accelerated and economical development of the underground construction projects. The investigations also play a key role in quality checks of construction and non-destructive health checks during entire life cycle of tunnels, caverns and other underground projects.

The present paper aims at presenting various possible applications for geophysical techniques for investigations in planning, pre-construction, construction and maintenance stages of underground projects. The paper also presents various advancements made in geophysical investigations.

INTRODUCTION

Geophysical tests are indirect methods of exploration in which changes in certain physical characteristics such as magnetism, density, electrical resistivity, elasticity, or a combination of these are used as an aid in developing subsurface information. Geophysical methods provide an expeditious and economical means of supplementing information obtained by direct exploratory methods, such as borings, test pits and in situ testing; identifying local anomalies that might not be identified by other methods of exploration; and defining strata boundaries between widely spaced borings for more realistic prediction of subsurface profiles. Typical uses of geophysical tests include determination of the top of bedrock, the rippability of rock, the depth to groundwater, the limits of organic deposits, the presence of voids, the location and depth of utilities, the location and depth of existing foundations, and the location and depth of other obstruction, to note just a few. In addition, geophysical testing can also obtain stiffness and dynamic properties which are required for numerical analysis.

Geophysical testing can be performed on the surface, in boreholes (down or cross hole), or in front of the TBM during construction.

Sub-surface imaging by means of geophysical survey is a powerful tool for site assessment and mapping which historically has been under-utilized world-over. Continuing improvements in survey equipment performance and automation have made large area surveys with a high data sample density possible. Advances in processing and imaging software have made it possible to detect, display, and interpret small geological features with great accuracy.

Some of the unique advantages of geophysical survey:

- Geophysical methods are quick to apply, saving in terms of time and money.
- Light and portable equipment allows access to remotest of sites.
- Provides information on critical geological features like faults/ fractures/ weak zones/ shear zones, not visible from surface information
- Large areas mapped quickly and inexpensively
- Researchers can assess site conditions, and target specific locations for detailed investigations by drilling, while avoiding others.
- Geophysical methods can quickly produce subsurface geology avoiding delays during execution due to meeting the unexpected.
- Shear wave profiles can be quickly obtained for ascertaining liquefaction potential and earthquake response.
- Buried utilities, pipes and cables, can be detected before drilling/ excavation, avoiding damage to utilities and costly accidents.
- Concrete structures can be quickly scanned to ascertain integrity and detect defects like voids, honeycombing etc.

BENEFITS AND LIMITATIONS OF GEOPHYSICS

Geophysical surveys can offer considerable time and financial savings compared with borehole investigations. At an early stage of site investigation it may be beneficial to undertake a reconnaissance geophysical survey to identify areas of the site which should be further investigated using invasive techniques i.e. those where anomalies have been identified. Geophysics has a unique advantage of providing continuous profile of subsurface rather than discreet information as provided by boreholes. This is critical in areas with complex geology and in projects like tunnels, where a small shear zone can lead to major challenges during execution. Geophysical surveys can be used effectively to determine the geological, hydrogeological and geotechnical properties of the ground mass in which the engineering construction is taking place.

Using geophysical techniques to solve engineering problems has sometimes produced disappointing results, particularly when a method, which lacked the precision required in a particular site investigation has been used, or when a method has been specified that is inappropriate for the problem under consideration. In most of the cases these problems can be avoided by taking services of an experienced geophysicists and access to various techniques available. In other cases the geological conditions at the site have been found to be more complex than anticipated at the planning stage of the geophysical survey and hence interpretation of the geophysical data by the geophysicists has not yielded the information expected by the engineer. It is often advisable to undertake a feasibility study at the field site to assess the suitability of the proposed geophysical techniques for the investigation of the geological problem.

Once the geophysical data has been obtained, it is possible to produce a model of the geological structure, which gives a realistic correlation with the data. The best overall model is obtained by using all the available geological information from boreholes and field mapping. Without this input of precise information, which includes knowledge of the fundamental physical properties of the geological material at the site, the model cannot be constrained in practical terms. There needs to be close collaboration between site geologists, engineers and geophysicists in the interpretation of the geophysical data.

PLANNING AND PRE-CONSTRUCTION STAGE

Carefully planned and executed geophysical program can considerably reduce uncertainties associated with geological surprises encountered while executing an underground project. Geological mapping and conventional borehole programs can provide only limited and discrete information along proposed tunnel route. A geophysical program not only provides a much detailed and continuous information of subsurface, but also can be used effectively to plan boreholes at anomalous locations, thus enhancing the accuracy of subsurface investigation while at the same time reducing cost and time involved in obtaining such information.

The range of geophysics that can be used in the domain of underground engineering is very broad:

- Gravity method
- Magnetic method
- Seismic refraction method
- Seismic reflection method
- Hybrid seismic method
- Spectral analysis of surface waves
- Multi-channel analysis of surface waves
- Continuous surface wave system
- Refraction micro-tremor
- Borehole seismic method
- Vertical seismic profiling
- Seismic tomography
- Electrical resistivity method
- Spontaneous potential method
- Induced polarization method
- Electrokinetic probing
- Ground penetrating radar
- Transient electromagnetic method
- VLF method
- Magnetotelluric method
- Radiometric method
- Airborne/ heliborne methods

Few of the above listed methods, proven to be extremely effective for tunnel investigations, are discussed in hereunder:

Heliborne Time Domain Electro-Magnetic Method (TDEM)

The Electro-Magnetic (EM) method is based on the physical effect of electromagnetic induction where an electrical current is induced in the ground and thus a secondary magnetic field is created. This secondary magnetic field is governed by the electrical resistivity of the ground. EM systems measure the EM time decay or frequency response and the related resistivity distribution is subsequently obtained by inverse modelling. Time-domain systems (TEM) measure an EM step response decaying with time. They are generally well suited for deeper investigations due to the higher transmitter moment. Some TEM systems can provide highly accurate and well calibrated data.

AEM (airborne electro-magnetic) data provides a powerful tool for geotechnical projects due to coverage and survey speed. Significant cost reductions can be achieved by planning geotechnical drillings based on the preliminary geological model derived from EM. Integrated with EM, limited drilling sites can be linked and combined to a model covering the complete area of interest.

Airborne or heliborne EM should be the first ground investigation step. Drilling locations can then be planned efficiently based upon AEM results. Subsequently drilling results should be incorporated in EM data interpretation and visualization leading to a combined geological model (e.g. bedrock topography). AEM is better suited for regional-scale projects rather than isolated projects because costs are relatively high for small surveys.

NGI's Geophysical subcontractor, Danish airborne electromagnetic (AEM) provider SkyTEM, carried out an AEM survey covering a tunnel corridor between Thimpu and Wangdue in Bhutan.

The survey target is the resistivity contrast between the weathered layer, the underlying intact bedrock, and possible weakness zones in the bedrock. High resistivity areas (competent bedrock) can be distinguished from low resistivity areas (incompetent and/or weathered rock). The regional resistivity is quite high (mostly above 1000 Ù.m), which is typical for gneisses. Due to this resistive background, the AEM depth of investigation was higher than anticipated, mostly between 300 and 800 m.

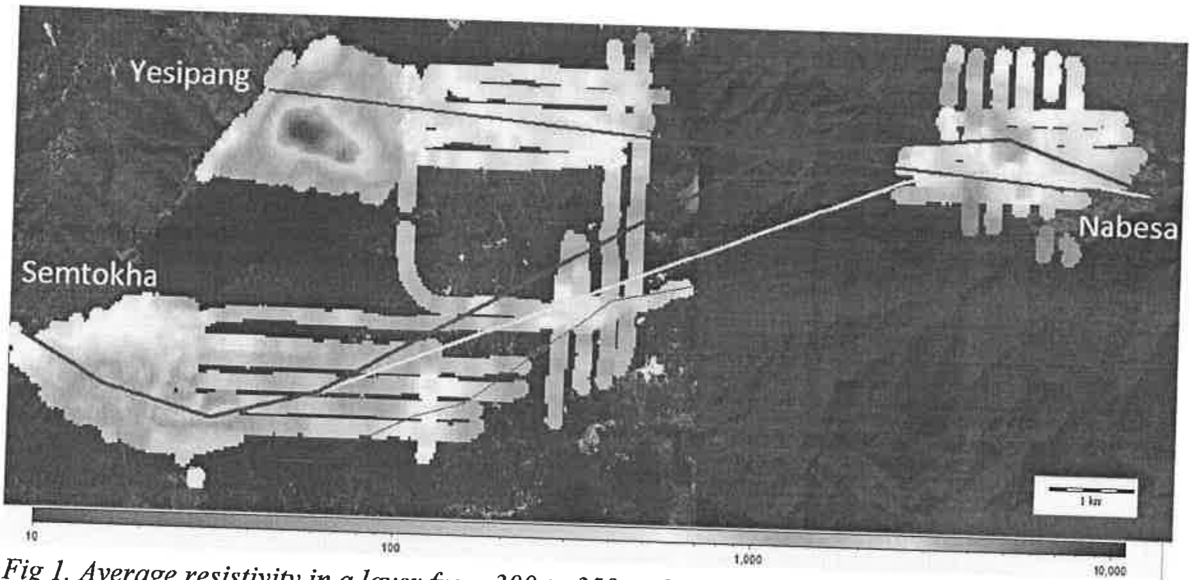


Fig 1. Average resistivity in a layer from 300 to 350 m depth below surface, for the entire survey area. The scale is in Ωm: red is conductive while blue is resistive

It was evident from the investigations that a deep conductive zone, south west of Nabesa portal, may intersect the initial alignment for the tunnel.

Deep seismic reflection surveying is the most advanced technique in geophysics today, thanks to its application on a huge scale for oil and gas exploration. This technique does, however, have other applications on a smaller scale, such as for civil engineering project site investigation. The methodology is identical, but the equipment and parameters are adjusted to provide a higher resolution at shallow depths.

In tunnel projects seismic reflection method can detect geologic structures in fault zones, find shallow, soft layers of underground earth materials, reduce mapping uncertainties and can greatly reduce the investigation costs of engineering projects.

Examples below show interpreted seismic reflection sections detailing geology of the area:

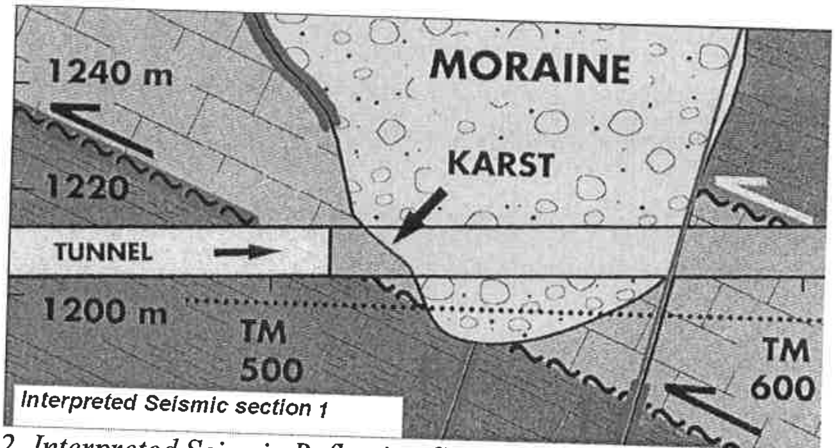


Fig 2 Interpreted Seismic Reflection Section Detailing Geology of Area

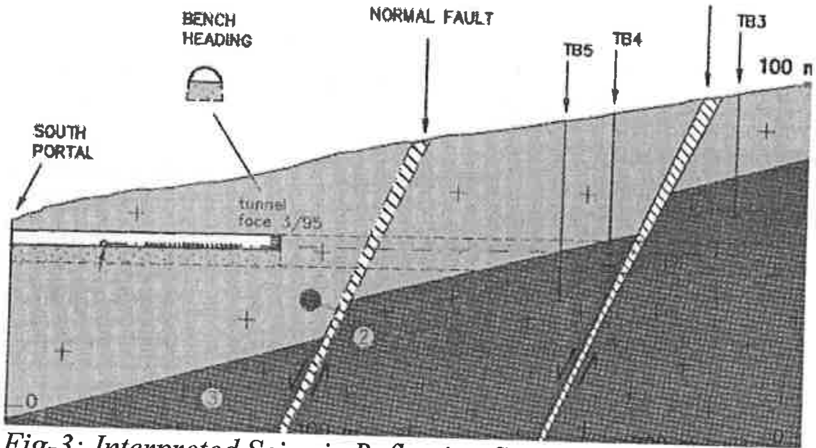


Fig-3: Interpreted Seismic Reflection Section Showing Faults

Seismic Refraction survey is an indispensable tool to determine bedrock profile, rock quality and depth, thickness of overburden, fractures and weak zones, topography of ground water etc. The method, like most other geophysical methods, provides continuous profile of subsurface, critical for engineering projects. Coupled with shear wave measurements, it also allows estimation of dynamic elastic moduli like Poisson's Ration, Young's Modulus, Shear Modulus. Example below shows a 03 layer model obtained from seismic refraction, with last interface of 3000 m/s depicting topography of rock.

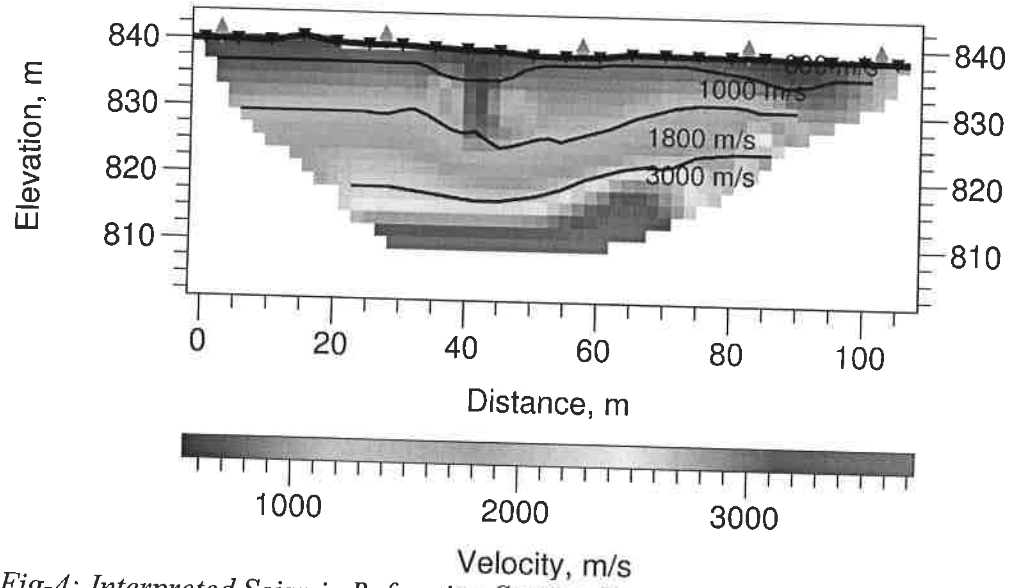


Fig-4: Interpreted Seismic Refraction Section Depicting Topography of Rock

Electrical Resistivity Imaging uses an array of electrodes (typically 64) connected by multicore cable to provide a linear depth profile, or pseudosection, of the variation in resistivity both along the survey line and with depth. The technique is extremely useful for investigations of important sites to get information on weak zones or buried channels, under the rock interface, which goes undetected in seismic refraction, which terminated at rock interface. Resistivity imaging can also be effectively used to determine rock profile along dam axis across high current shallow rivers where deployment of hydrophones is not possible restricting use of seismic refraction.

The example below shows use of ERI to detect a soft zone under rock cover, which would otherwise go undetected even by closely spaced boreholes:

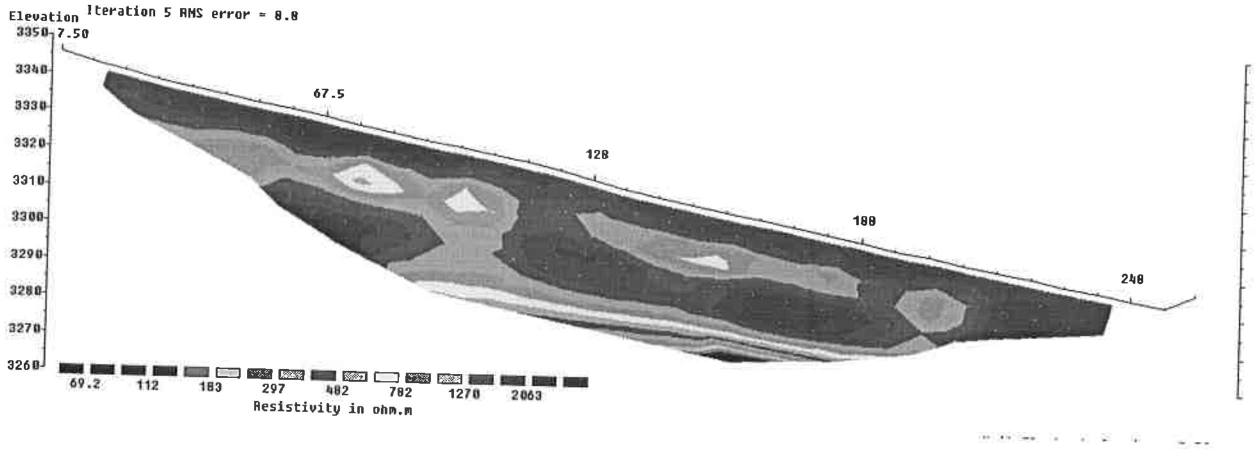


Fig-5: Interpreted Electrical Resistivity Imaging Section Showing Soft Zone Under Rock

ReMi (Refraction Microtremor) can be performed under the same layout as used for seismic refraction, to obtain excellent shear wave velocity profiles of subsurface. ReMi is a new, proven seismic method for measuring in-situ shear-wave (S-wave) velocity profiles. It is economic both in terms of cost and time. Testing is performed at the surface using the same conventional seismograph and vertical P-wave geophones used for refraction studies. The seismic source consists of ambient seismic "noise", or micro-tremors, which are constantly being generated by cultural and natural noise. Because conventional seismic equipment is used to record data, and ambient noise is used as a seismic source, the ReMi method is less costly, faster and more convenient than borehole methods and other surface seismic methods, such as SASW and MASW used to determine shear-wave profiles. Depending on the material properties of the subsurface, ReMi can determine shear wave velocities down to a minimum of 40 meters (130 feet) and a maximum of 100 meters (300 feet) depth.

Typical 2D ReMi profile constructed using various 1D profiles is shown hereunder showing shear wave velocities up to a depth of 100m:

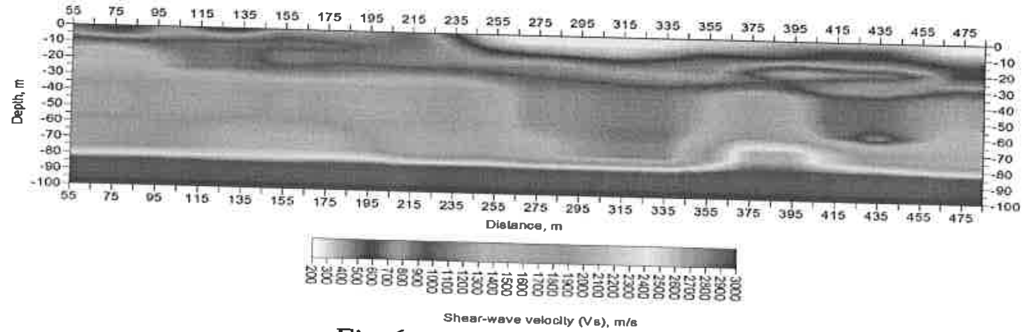


Fig-6: Example of ReMi Result

Cross hole Seismic Tomography

The latest technique of seismic refraction tomography provides much more realistic and accurate subsurface velocity model compared to typical layered models obtained through conventional seismic refraction surveys. It is based on generation of elastic energy using various sources at predetermined depths in one bore hole and detecting it in another borehole through a chain of hydrophones. Velocity analysis involves estimation of time required to cross the distance between source and receiver depending on variations in elastic properties of material crossed. Deviation survey is carried out prior to tomography for determining alignment of bore holes. The set-up includes a source hole & a receiver hole as shown hereunder:

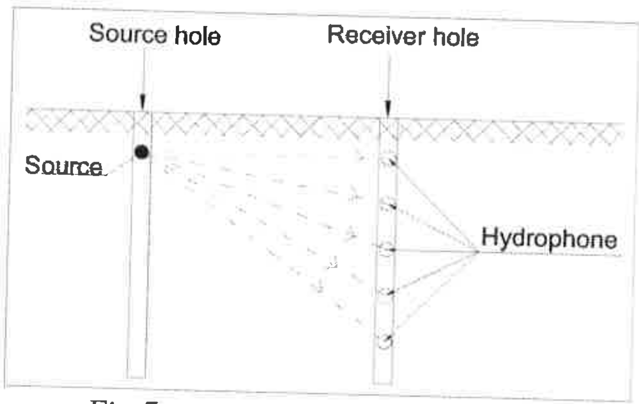


Fig-7: Typical seismic tomography Setup

A tomographic section generated from survey between two non planar holes for a hydro project for detection of a cavity.

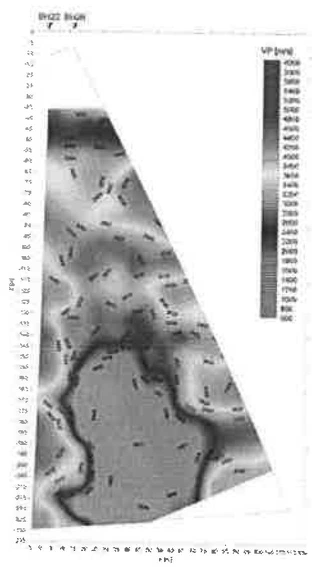


Fig-8: Tomographic section showing cavity

Various other geophysical methods like cross-hole seismic surveys, gravity, magnetic etc., can be applied to obtain critical subsurface information.

CONSTRUCTION STAGE

During construction stage of tunnels and underground projects, geophysical methods can be effectively used to predict unfavorable geological conditions (e.g. Tunnel Seismic Prediction ahead of tunneling) and to check and inspect quality of construction. As an example a quick GPR run on concrete surface can effectively detect distribution of reinforcement bars, presence of honey combing and other similar defects.

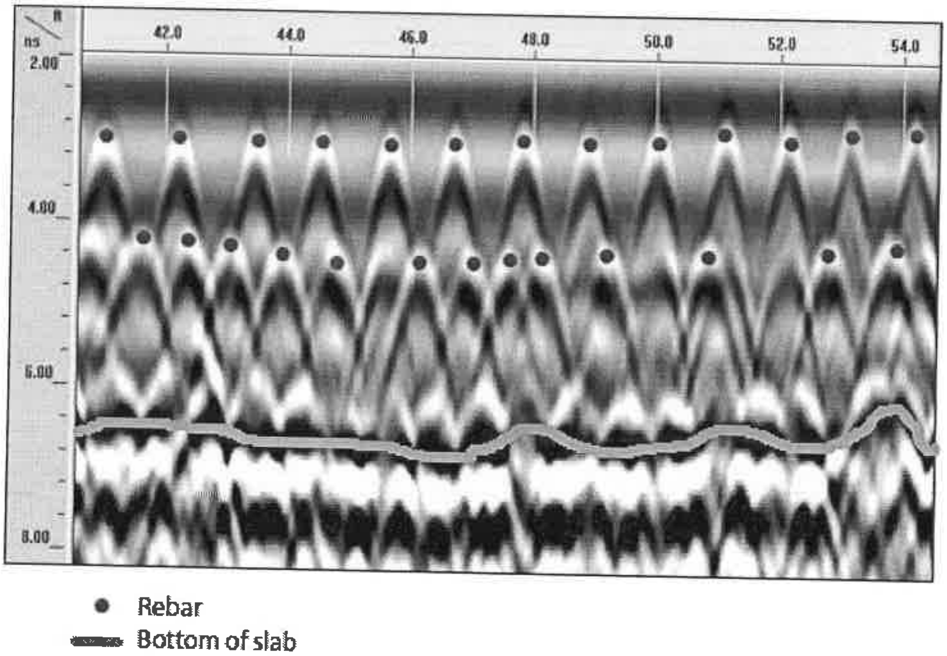


Fig-9: Interpreted GPR Section Showing Rebars & Concrete Slab Bottom

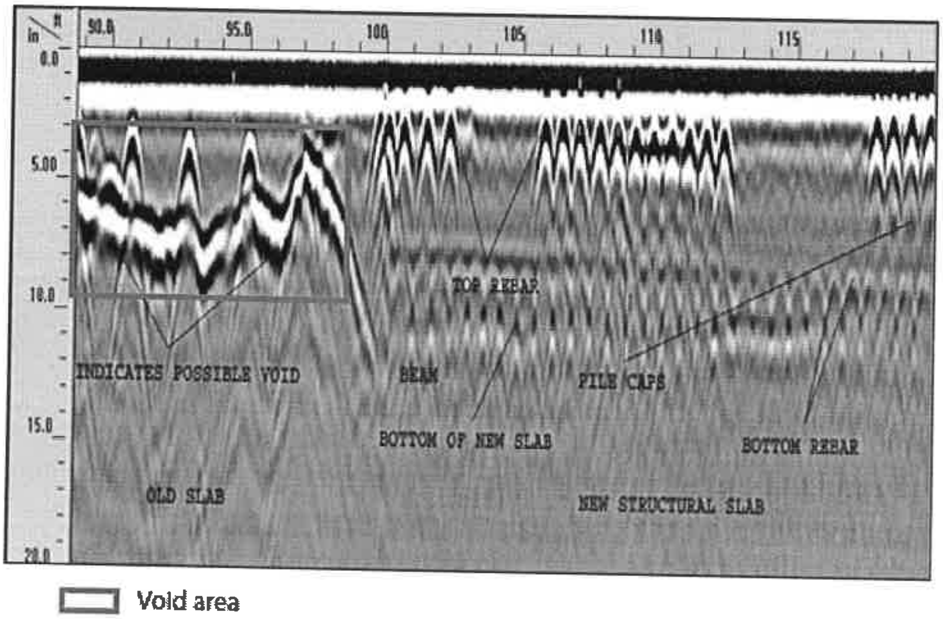


Fig-10: Interpreted GPR Section Showing Voids

MAINTENANCE STAGE

Ground Penetrating Radar is routinely used for health checks of tunnels in various ways:

- Provides Information on Construction & Condition
- Masonry Tunnels- mapping delamination in masonry arches, voids in and behind the brick lining, moisture variation due to leaking pipes, construction arrangement & unexpected changes in masonry and overburden thickness, mapping hidden blind construction shafts.
- Concrete Tunnels- Determine the thickness and arrangement of spayed, in-situ or pre-cast concrete. Voids within or behind the lining. Map variation along a tunnel, such as changes in moisture levels or geology.
- Unlined Tunnels- Maps voids, fractures and manmade features such as rock anchors within rock tunnels.

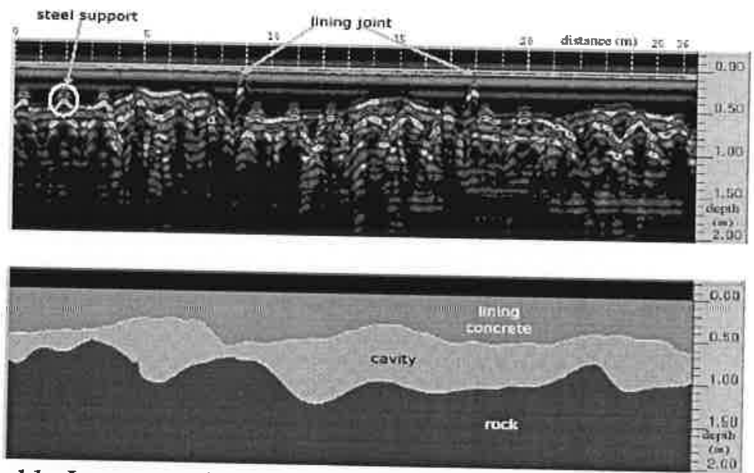


Fig-11: Interpreted GPR Section Showing Cavity Behind Lining

Electrical Resistivity Imaging can also be conducted along any line on the surface of the tunnel to detect features like cavities behind tunnel walls.

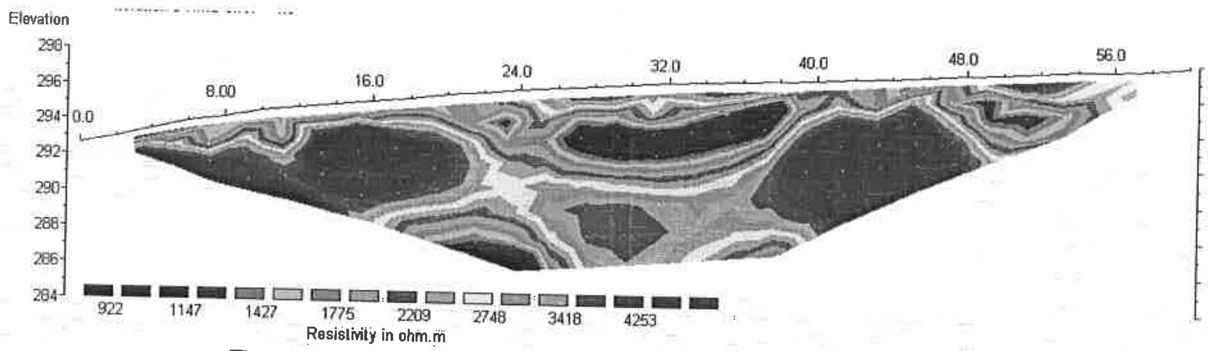
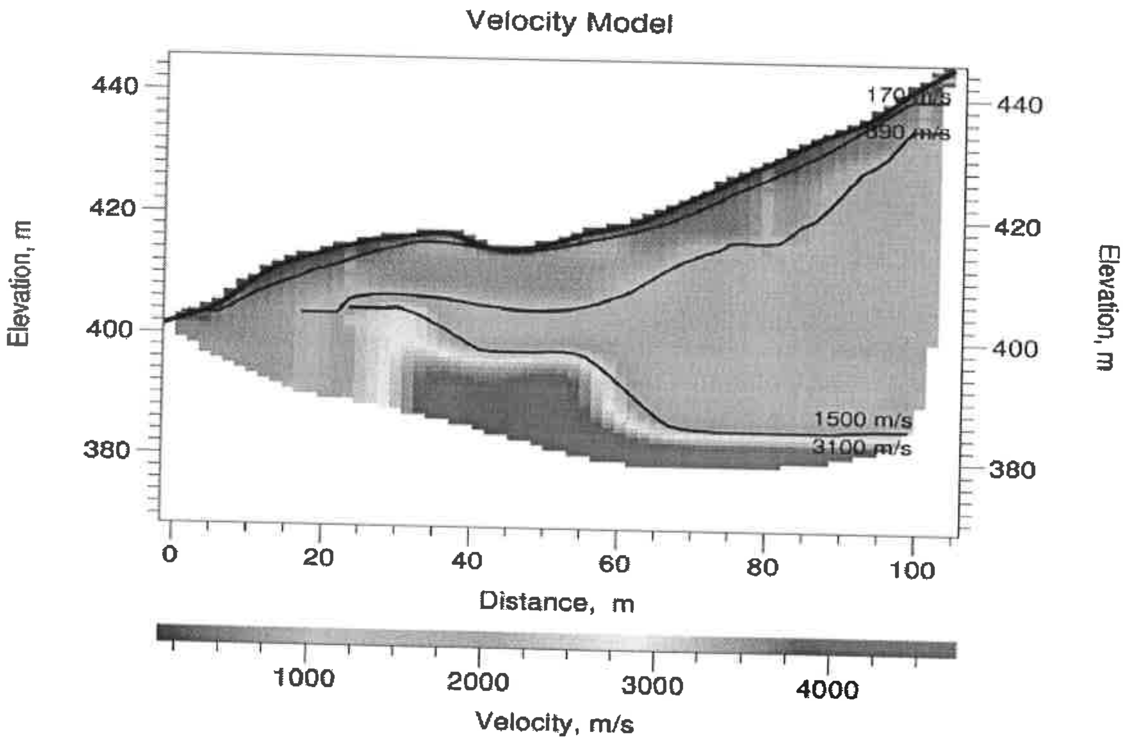


Fig-12 Interpreted Electrical Resistivity Section Showing Cavity

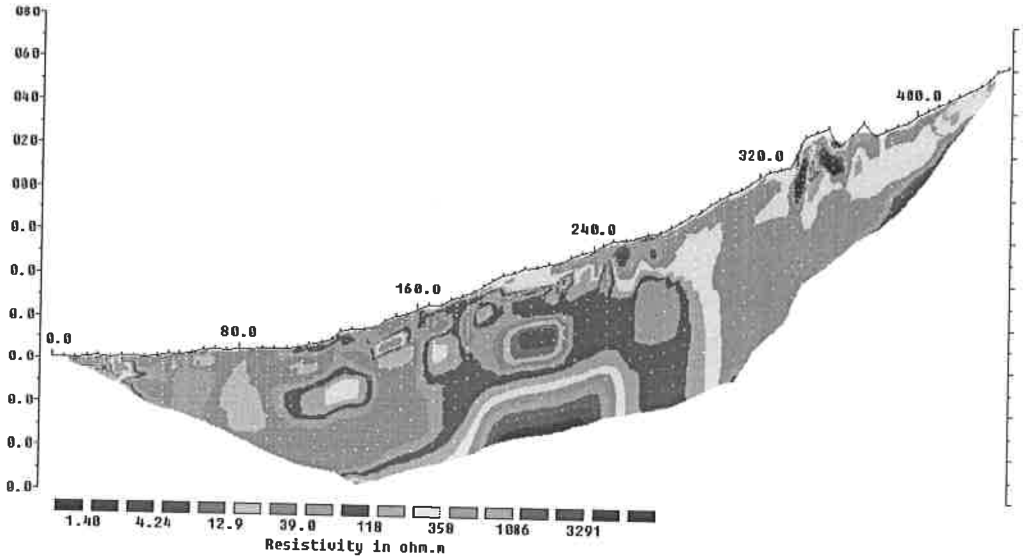
CASE STUDY-1

This case study is related to a railway line project. The proposed Railway line passes through foot hills of the Himalayas and dense reserve forest of Darjeeling district at West Bengal and East Sikkim district of Sikkim state. Geophysical seismic refraction survey was carried out at three tunnel portal locations to determine the stratigraphy of the proposed area. The study successfully revealed the highly undulating topography of rock, and abrupt thickening of overburden/ weathered zone.



CASE STUDY-2

This case study related to a tunnel project in the state of J&K where a shear zone's presence was suspected based on geological signatures. The main objective of investigations was to detect features like thrust, shear zones etc., upto a depth of 70m from the surface. Investigations lines were selected carefully to reveal information along the tunnel route. Site specific details and elevation details have been removed from images to maintain confidentiality of the project as per confidentiality requirement of the client.

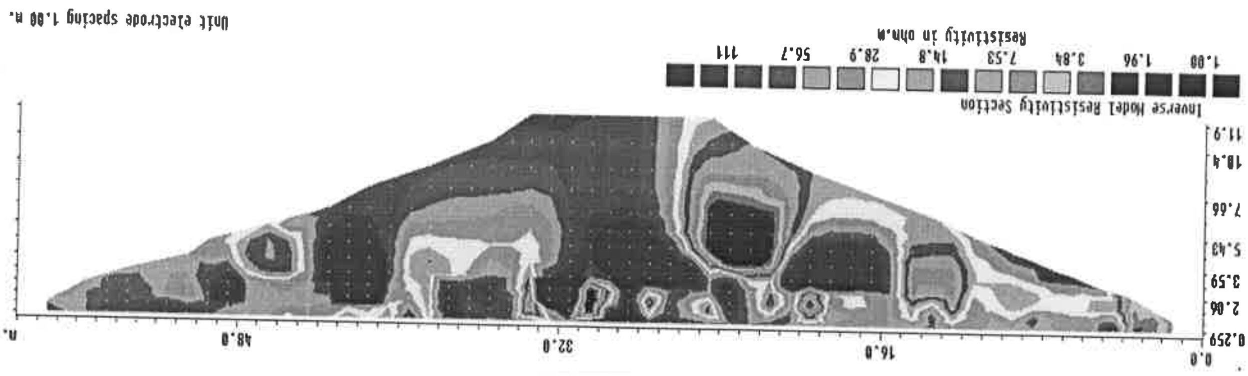


The subsurface resistivity section clearly demonstrates the geoelectrical and lithological layer sequence beneath the ground. The different colour contours represent the different lithological layers system within a depth of 78 m bgl. Following are the notable features of this profile (Elevation values changed).

- The rock (high resistivity- corresponding to quartzite as per local geology) seems to be lower than investigates depth upto RL 1860m, then coming up to RL 1900-1920 and abruptly terminating at Ch 250m. The rocky strata again appears Ch 290 onwards, and post this there is a distinct change in stratigraphy, with presence of high resistivity in all parts of profile.
- There is an extremely low resistivity zone (deep blue) between Ch 200- Ch 220m, with a thickness of around 7-10m. Such low resistivity zones can be typically associated with shear zones.

CASE STUDY-3

This case study related to investigations from inside a tunnel, to detect saturated anomalous zones responsible for seepage in the tunnel. Electrodes were planted along the length of tunnel, along various co-parallel lines, and zone responsible for seepage were clearly identified. One such section us presented here, with Blue zone being the source of seepage.



PLANNING FOR GEOPHYSICAL INVESTIGATIONS

The sooner that geophysical information is obtained and evaluated, the greater is the potential for optimization of the alignment and profile. There are many cases where tunnel projects benefited because the horizontal or vertical alignment was dramatically changed as a result of sufficient subsurface information.

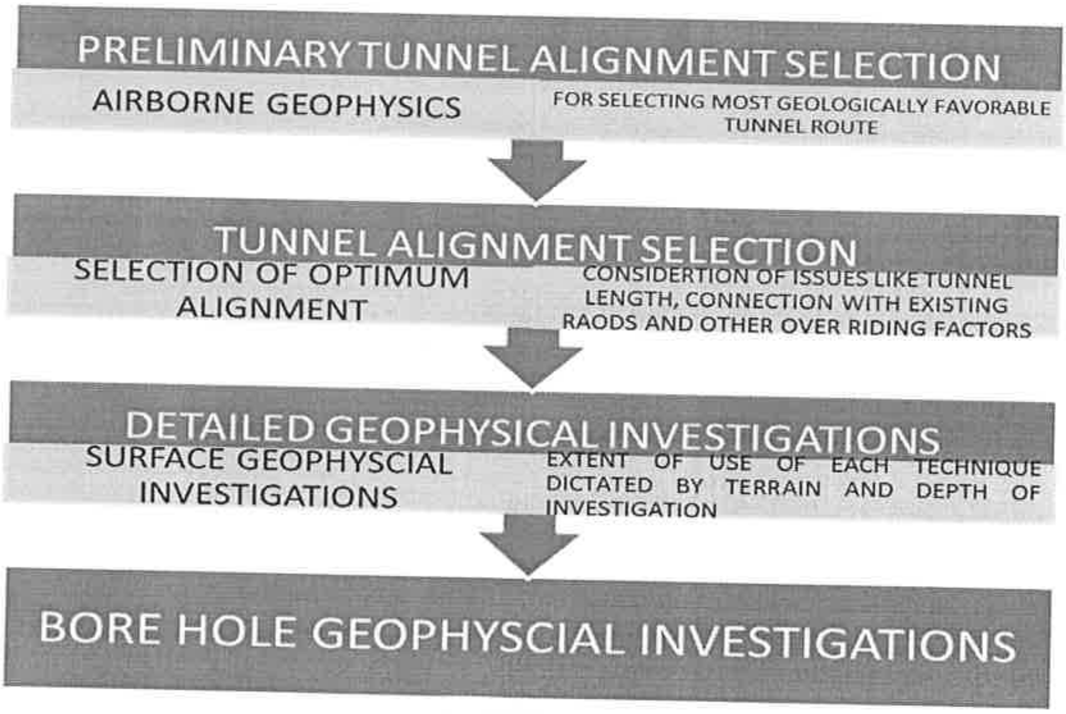
Many project owners have a mistaken impression that using a Design-Build contracting method will allow them to limit, and possibly even avoid, investigations merely by shifting the responsibility to the contractor. Any tunnel project requires that that a hopefully-realistic fixed cost be agreed upon by the two parties at a very early stage of the project and at a time when most projects have too little definitive geological information. Thus, there could be greater risk to the owner and greater chance for claims if the "assumptions" about geology that are made during the negotiation period are not correct. It is therefore essential that geophysical and limited geotechnical investigation was conducted early in the project so that maximum factual information is provided to the bidders.

SCOPE OF GEOPHYSCIAL INVESTIGATIONS

One of the most difficult and controversial aspects of any geophysical investigation is deciding how much investigation to do. Among other things, the difficulty results from the fact that despite advances in site investigation techniques, tunneling is mostly an art, or, at best, an inexact science. There is no guarantee that any given amount of investigations will provide sufficient information for tunnel design, even if properly planned and executed. In fact, one of the purposes of exploration is to determine whether any conditions exist that may warrant further investigation, and the phased exploration concept is based on this premise.

The nature of the project also plays a major role in determining the scope and cost of the investigation. Conventional projects in uniform geology might require less investigation but complex projects in adverse geology like what is encountered in Himalyas might require much more investigation. These more complex projects can benefit by use of the newer, more promising geophysical techniques being developed.

GEOPHYSICAL TECHNIQUES TO BE USED FOR NEW TUNNEL PROJECTS



AIRBORNE GEOPHYSICAL TECHNIQUES

Figure 13 below shows various possible options available at the preliminary stage of the proposal. Airborne geophysical techniques are to be used at this stage. With airborne geophysical techniques, a large area can be covered very quickly. Specifically, high resistivity areas i.e. competent bedrock can be distinguished from low resistivity areas i.e. incompetent or weathered rock.

In complex geological areas like Himalaya, which is generally characterized by steep slopes, lofty hills, and complex geological and tectonic settings, AEM surveys are most essential for investigating a large area. After subsurface information about various alignment options is available, other factors like tunnel length, connectivity with existing roads, areas to be served by linking various en route locations need to be considered before selecting the most favorable tunnel alignment.

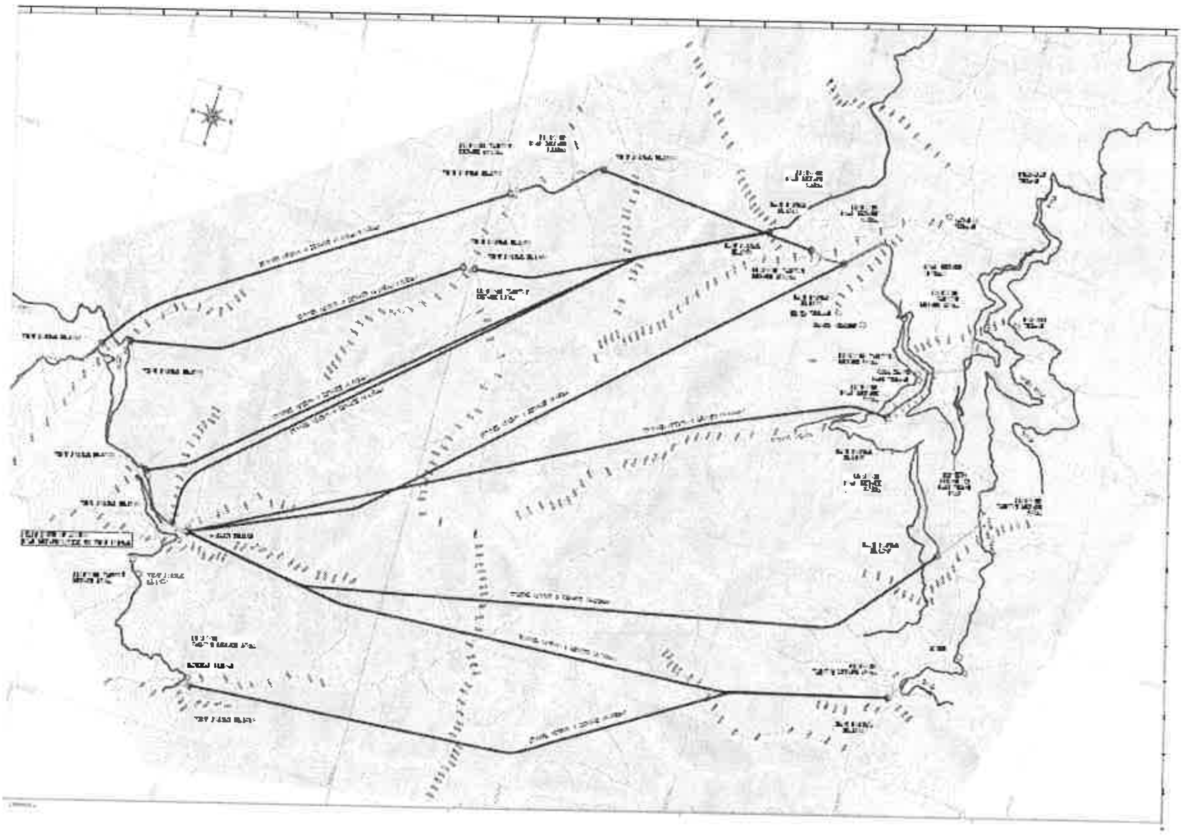


Fig-13: Tunnel route alignment options

SURFACE GEOPHYSICAL INVESTIGATIONS

The aim of investigations at this stage is to cover as much as possible area/volume of the selected tunnel alignment so that maximum subsurface information is made available. A combination of seismic reflection, seismic refraction and electrical resistivity imaging should be used at this stage.

5.3.2.1 SEISMIC REFLECTION

Seismic reflection is a very well developed technique due to its extensive use in oil and gas exploration projects where depth of investigations is in excess of 4000m. Seismic reflection should be carried out along the entire tunnel alignment. The planned depth of investigation should extend a few tunnel diameters beneath the anticipated invert. A suitably designed seismic reflection profile can cover entire rock mass of interest along tunnel alignment. Figure 14 below shows seismic reflection profile layout along a tunnel cross section. Fig 15 shows rock mass investigated by undertaking seismic reflection along tunnel alignment.

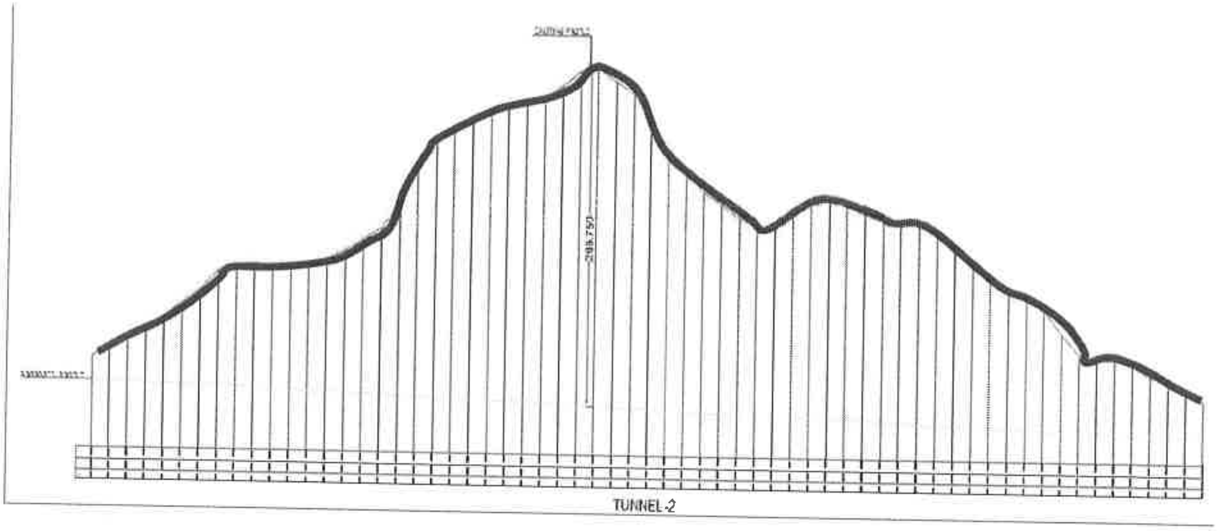


Fig. 14 Seismic reflection profile along planned alignment

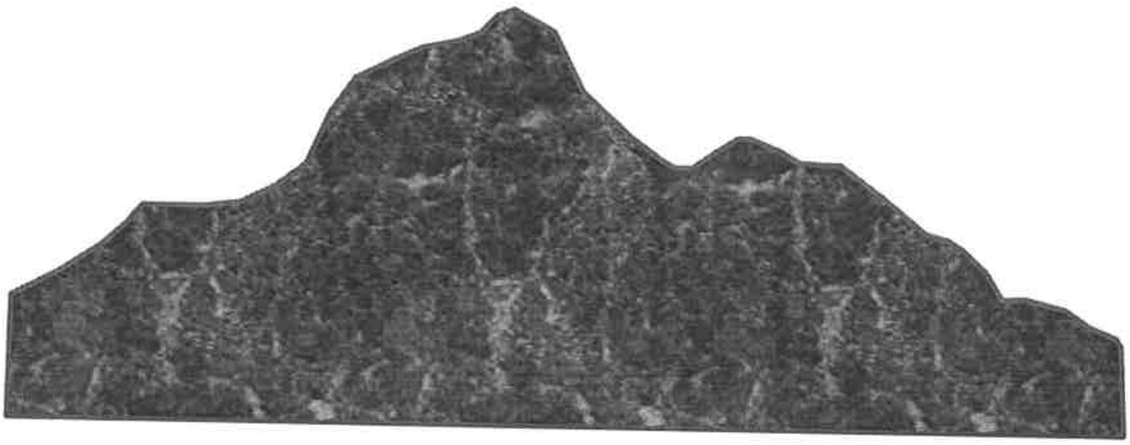


Fig. 15 Rock mass/volume investigated by seismic reflection along tunnel alignment

ELECTRICAL RESISTIVITY IMAGING (ERI)

Electrical resistivity imaging should be should be carried out along the entire tunnel alignment. Commercially available resistivity systems with 60-120 electrodes with profile spread of 1200m can provide depth of investigations up to 200-250m. In cases where the rock overburden is in excess of the possible depth investigated by ERI, the information obtained from first 200-250m of subsurface can be extrapolated to the anticipated invert. The geological model can be further refined by using bore hole data from subsequent geotechnical investigations. Fig 16 illustrates the rock mass/volume that can be investigated by using ERI.

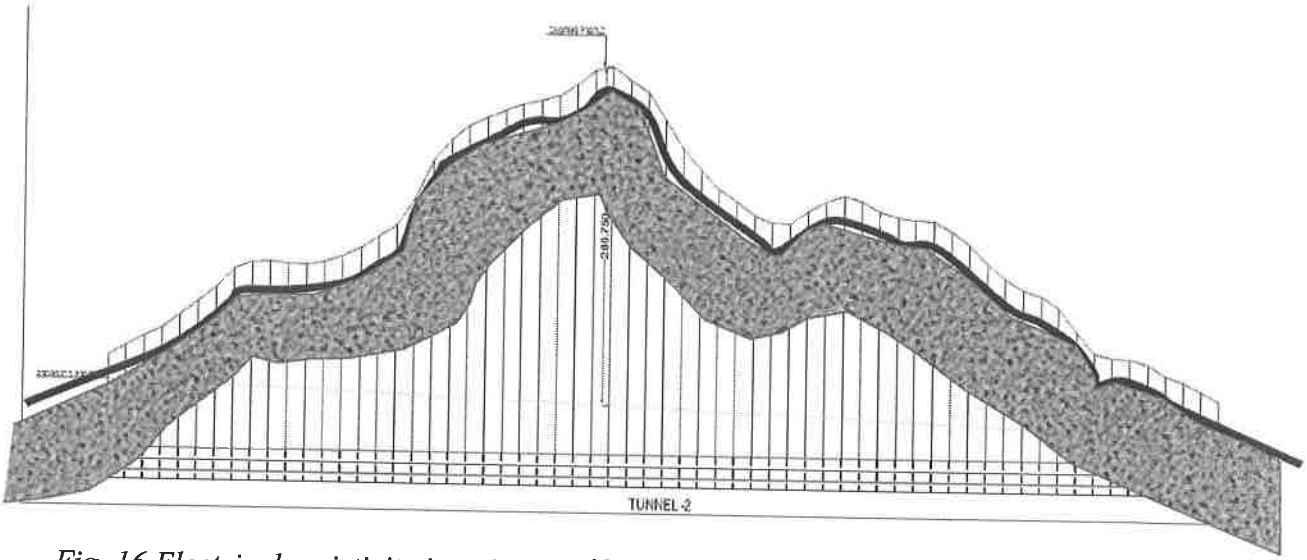


Fig. 16 Electrical resistivity imaging profile along planned alignment and rock mass/volume investigated

SEISMIC REFRACTION TOMOGRAPHY (SRT)

Seismic refraction tomography (SRT) should be used at tunnel portals and areas where required depth of investigation is up to 70m. SRT is relatively easier and cheaper as compared to seismic reflection and should be used. Combination of ERI and SRT is to be used to investigate for portal stability assessment and design.

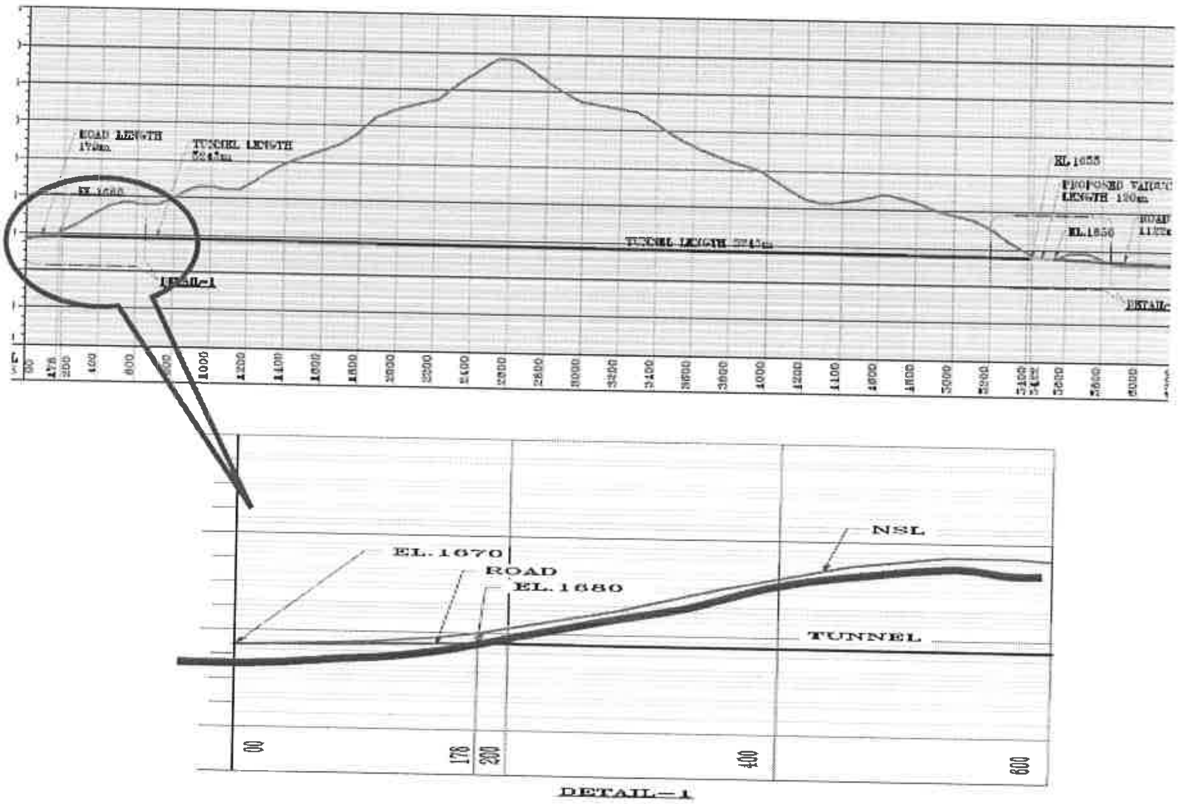


Fig. 17 Seismic refraction tomography profile at portal

SEISMIC TOMOGRAHY

Seismic tomography is conducted between a pair of bore holes. It requires preparation of plastic casing and water tight bore holes which is expensive and time consuming. Hence, this technique should be used selectively only in cases where the costs and time involved are justified.

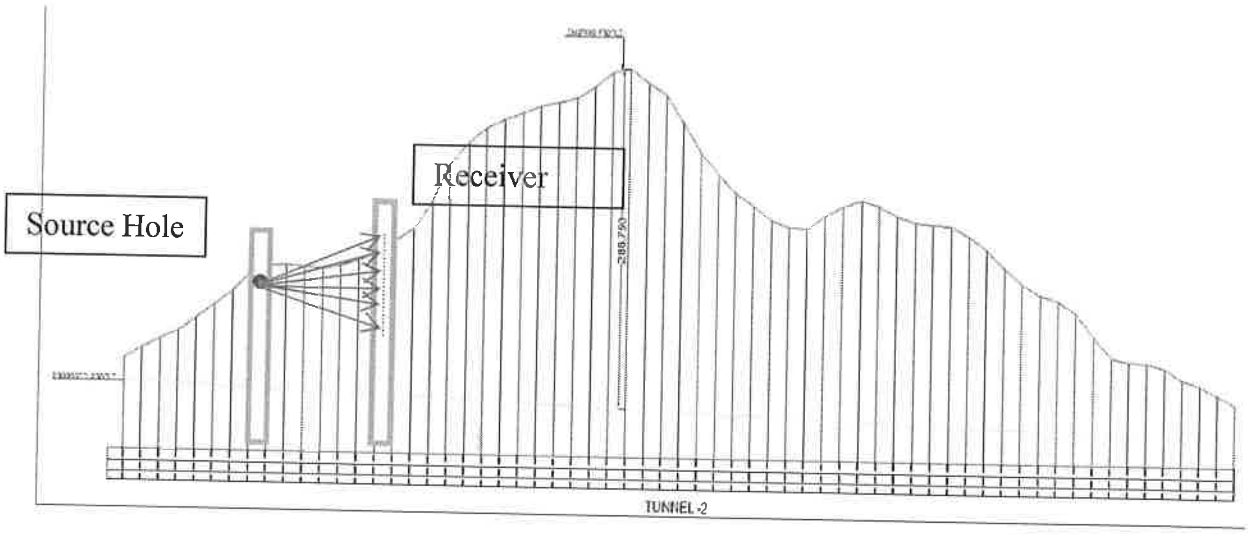


Fig 18 Source and receiver bore hole arrangements for seismic tomography

CONCLUSIONS:

Owners, planners, and designers frequently do not appreciate the vital importance of geophysical services to underground projects. It is well documented that insufficient investigation can result in misleading information and can substantially increase the risk of not finding hazards and unknown conditions that can seriously delay or stop construction, with costly consequences.

There exist various geophysical techniques to investigate the subsurface before finalizing the design to determine presence of anomalies like faults, shear zone, water table, bedrock profile, etc. The selection of techniques is governed by the objectives of study and geology. The information can be acquired quickly at a very small cost, and can save huge amount of money and time otherwise wasted when problems are not anticipated. For any further details author can be contacted at sanjay@parsan.biz.

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2. Planning & Site Investigation in Tunneling- Harvey W. Parker
3. Seismic Reflection Models, Courtesy Mr Claude Robillard, Geophysics GPR International, Canada
4. Airborne Electromagnetic Surveys for Carrying Out Feasibility Studies for Constructing Road and Rail Tunnels in Himalaya by Rajendra Bhasin, Thomas Pabst, Sara Bazin and Arnstein Aarset

Guidelines for the Design of Tunnels

ITA Working Group on General Approaches to the Design of Tunnels

Abstract—*This second report by the ITA Working Group on General Approaches to the Design of Tunnels presents international design procedures for tunnels. In most tunnelling projects, the ground actively participates in providing stability to the opening. Therefore, the general approach to the design of tunnels includes site investigations, ground probings and* in-situ *monitoring, as well as the analysis of stresses and deformations. For the latter, the different structural design models applied at present—including the observational method—are presented. Guidelines for the structural detailing of the tunnel lining and national recommendations on tunnel design are also given. It is hoped that the information herein, based on experiences from a wide range of tunnelling projects, will be disseminated to tunnel designers throughout the world.*

Résumé—*Le groupe de travail AITES sur le dimensionnement des tunnels présente ici son deuxième rapport. En rassemblant toutes les informations, qui étaient accessibles entre les pays sur le dimensionnement des tunnels, nous espérons, que les expériences gagnées sur beaucoup de projets des travaux souterrains seront propagées dans tout le monde. Parce que le sol participe d'une grande partie à fournir des moyens de stabilité pour des ouvertures souterraines, des méthodes de dimensionnement comprennent aussi bien les investigations sur le chantier, les essais laboratoires et la surveillance pendant le progrès du travail que l'analyse des contraintes et des déformations. Concernant ce dernier point, des modèles de dimensionnement différents et actuellement appliqués sont présentés, y compris aussi la méthode d'observation. Recommendations pour les détails de revêtement et quelques recommandations nationales sur le dimensionnement des tunnels achèvent ce rapport.*

1. Scope of the Guidelines

The International Tunnelling Association (ITA) Working Group on General Approaches to the Design of Tunnels was established in 1978. As its first project, the group developed a questionnaire aimed at compiling information about structural design models used in different countries for tunnels constructed prior to 1980. A synopsis of the answers to the questionnaire was published by the International Tunnelling Association in 1982 (ITA 1982).

As a continuation of that first report, the working group herein presents guidelines that attempt to condense the various answers from the first report and include additional experiences in the general approaches to the design of tunnel structures. These guidelines fulfill one of the main objectives of the International Tunnelling Association, namely, to disperse information on underground use and underground structures throughout the world by crossing national borders and language barriers.

Those interested in the subject of tunnel design should also consult published reports of other ITA working groups, e.g. the recent ITA report on contractual sharing of risk (see *T&UST* 3:2) and ITA recommendations on maintenance of tunnels (see *T&UST* 2:3). Furthermore, a number of national and international organizations, such as the International Society on Rock Mechanics, have published recommendations on related subjects, such as field measurements and laboratory testings for rock and ground. Some of these publications and reports are listed in the Appendix.

In tunnelling, most often the ground actively participates in providing stability to the opening. Therefore, the design procedure for tunnels, as compared to aboveground structures, is much more dependent on such factors as the site situation, the ground characteristics, and the excavation and support methods used. Recommendations on tunnel design naturally are limited with regard to their consistency and applicability because each tunnelling project is affected by special features that must be considered in the design. Nevertheless, it is hoped that the general outline provided in these guidelines, based on the experience gained from many tunnelling projects, may be of some help for those starting a project.

This report is edited by Heinz Duddeck, Animateur of the ITA Working Group on General Approaches in the Design of Tunnels. Present address: Prof. Heinz Duddeck, Technical University of Braunschweig, Beethovenstrasse 51, 3300 Braunschweig, Federal Republic of Germany.

2. Outline of General Approaches

2.1. *General Procedure in Designing a Tunnel*

Planning a tunnelling project requires the interdependent participation of the following disciplines, at a minimum:

- Geology.
- Geotechnical engineering.
- Excavation technology. e.g. machine tunnelling.
- Design of the supporting structural elements, including long-term behavior of materials.
- Contract principles and law.

Although the experts in each of these disciplines may be responsible only for their specific area of knowledge, the decision on the main design features should be the outcome of the cooperative integration of all the disciplines. Only thus can it be ensured that the project, in all its details, has been developed in unity, and not as the consecutive addition of the separate work of each of the experts.

The basics documents for tunnel design should include or cover:

- The geological report presenting the results of the geological and geophysical survey.
- The hydrogeological report.
- The geotechnical report on site investigations, including the interpretation of the results of site and laboratory tests with respect to the tunnelling process, soil and rock classification, etc.,
- Information on line, cross-section, drainage, and structural elements affecting later use of the tunnel.

- Plans for and a description of the projected excavation or driving procedure, including the different cross-sections related to different ground conditions.
- Design documents for the types of excavation methods and tunnel supports likely to be applied, considering, e.g. excavation advance and face support (types and number of anchors, shotcrete strength, closure length, etc.).
- The program for the *in-situ* monitoring of the tunnel by field measurements.
- The analysis of stresses and deformations (for unlined tunnels as well as for single-or double-lined tunnels), and the dimensioning of the tunnel support for intermediate phases and final linings.
- The design for waterproofing or drainage.
- Structural documents for the final design of the tunnel project, including the detailing.
- During and after the excavation, reports on the field measurements and interpretation of their results with respect to the response of the ground and the structural safety of the tunnel.
- Documentation of the problems encountered during the excavation and measures applied, e.g. strengthening the ground or changing the projected type of support, based on monitoring results.

The above sequence of these basic documents also provides the general outline of the design procedure.

2.2. *Elements of the Structural Design Model for Tunnels*

In planning, designing, analysing and detailing a structure, engineers promise that the structure will neither suffer structurally nor collapse during its projected lifetime. Thus, models of the reality are necessary for analysis in order to predict the behaviour of a tunnel during the excavation and during its lifetime. Models are also needed for bidding on projects.

The following main elements involved in the design procedure are shown as a flow-chart in Fig. 1:

(1) *Geology and site investigations* must confirm the line, orientation, depth, etc., of the opening, e.g. a cavern.

(2) *Ground probing and soil or rock mechanics must be applied* to determine the ground characteristics, e.g. primary stresses, soil or rock strength, faults, water conditions.

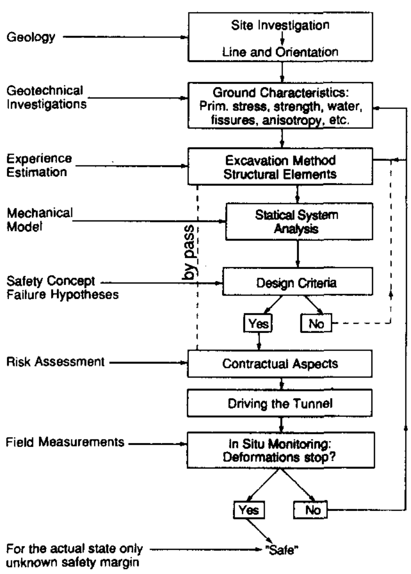


Figure 1. Design process for tunnelling.

(3) *Experience and preliminary estimates or calculations* are used to determine the cross-section required and the choice of the excavation method or the tunnel driving machine to be used, as well as the methods of dewatering the ground and the selection of the supporting structural elements.

(4) After steps (1)-(3) are completed, the tunnelling engineer must derive, or even invent, a structural model. By applying equilibrium and compatibility conditions to the model, the engineer has to arrive at those criteria that are factors in deciding whether or not the design is safe. Different models may be used for each excavation phase, for the preliminary and the final tunnel lining, or for different ground behaviour, e.g. in discontinuous rock or homogeneous soft soil. Modelling of the geometric features may vary greatly, depending on the desired intensity of the analysis.

(5) A safety concept drawn from failure hypotheses may be based on criteria such as strains, stresses, deformation, or failure modes.

The bypass in Fig. 1 indicates that for many underground structures, as in mining or in self-supporting hard rock, no design models at all are applied. In such cases, past experiences alone may be sufficient.

Risk assessment by the contractor as well as by the owner is needed at the time of contract negotiations. Risks involve possible structural failures of the tunnel support and lining, functional failures after completion of work, and financial risks. The contractual aspects also include risk sharing and risk responsibilities.

In-situ monitoring can be applied only after the tunnelling has begun. If the displacements stop increasing over time, it generally may be assumed that the structure is designed safely. Yet monitoring provides only part of the answer to the question of safety, for it does not tell how close the structure may be to sudden collapse or nonlinear failure modes. The results of field measurements and experiences during excavation may compel the engineer to change the design model by adjusting it to real behaviour.

An iterative, step-by-step approach is characteristic of the design of structures in the ground that employ the participating strength of the ground (see loops in Fig. 1). The designer may begin by applying estimated and simple behavioural models. Adjustments based on actual experiences during the tunnelling excavation (such as excavating the initial section in the same ground conditions or driving a pilot tunnel) will bring the model closer to reality and refine it (if refinement is consistent with the overall accuracy attainable). The interpretations of *in-situ* measurements (and some back analyses) also may assist designers in making these adjustments.

All of the elements of the structural design model in Fig. 1 should be considered an interacting unity. Scattering of parameters or inaccuracy in one part of the model will affect the accuracy of the model as a whole. Therefore, the same degree of simplicity or refinement should be provided consistently through all the elements of the design model. For example, it is inconsistent to apply very refined mathematical tools simultaneously with rough guesses of important ground characteristics.

2.3. *Different Approaches Based on Ground Conditions and Tunnelling Methods*

The response of the ground to excavation of an opening can vary widely. Based on the type of ground in which tunnelling takes place, four principal types of tunnelling may be defined:

(1) for cut-and-cover tunnelling, in most cases the ground acts only passively as a dead load on a tunnel structure erected like any aboveground engineering structure.

(2) In soft ground, immediate support must be provided by a stiff lining (as, for example, in the case of shield-driven tunnels with tubbings for ring support and pressurized slurry

for face support). In such a case, the ground usually participates actively by providing resistance to outward deformation of the lining.

(3) In medium-hard rock or in more cohesive soil, the ground may be strong enough to allow a certain open section at the tunnel face. Here, a certain amount of stress release may permanently be valid before the supporting elements and the lining begin acting effectively. In this situation only a fraction of the primary ground pressure is acting on the lining.

(4) When tunnelling in hard rock, the ground alone may preserve the stability of the opening so that only a thin lining, if any, will be necessary for surface protection. The design model must take into account the rock around the tunnel in order to predict and verify safety considerations and deformations.

Especially in ground conditions that change along the tunnel axis, the ground may be strengthened by injections, anchoring, draining, freezing, etc. Under these circumstances, case (2) may be improved, at least temporarily, to case (3).

The characteristic stress release at the tunnel face (Erdmann 1983) is shown in Figs 2 and 3. The relative crown displacement w is plotted along the tunnel axis, where $w/w_0 = 1.0$ represents the case of an unsupported tunnel. In medium-stiff ground nearly 80% of the deformations have already taken place before the lining (shown here as shotcrete) is stiff enough to participate.

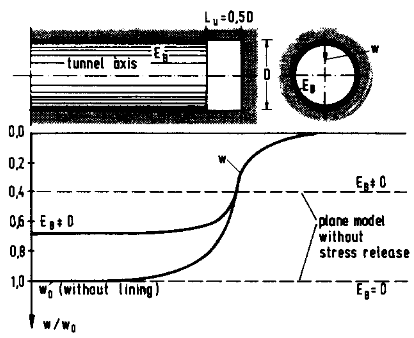


Figure 2. Crown displacement w *along the axis, ahead and beyond the tunnel face.*

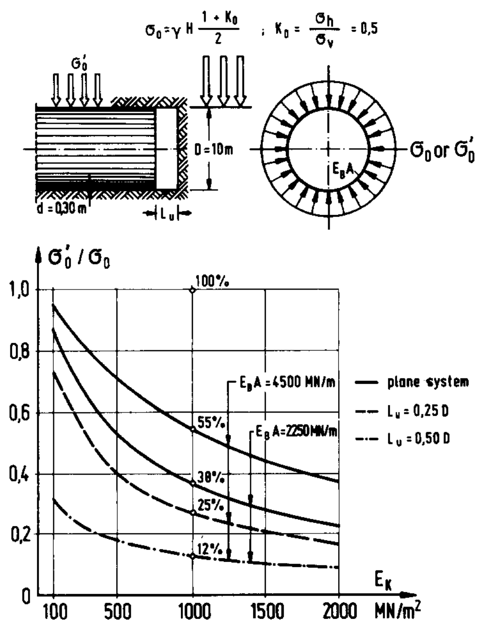


Figure 3. Ground stresses acting on the lining as fractions of the primary stress (Erdmann 1983).

For a simplified plane model with no stress release, where the full primary stresses are assumed to act on a lined opening, the displacement may be only 0.4 of that occurring in the unsupported case. The corresponding stress release is shown in Fig. 3. The simplified example, considering only the constant part of radial pressure, yields the values shown for a ring stiffness of $E_BA = 15{,}000 \times 0.3 = 4500$ MN/m and a ground deformation modulus of $E_K = 1000$ MN/m².

Even in the unrealistic case when the full primary stress acts simultaneously on the ground opening and the lining, only 55% of the stress is taken by the lining; in the case of E_BA = 2250 MN/m, only 38% is taken by the lining. If an open section of 0.25 of the tunnel diameter is left without lining support, the lining takes only 25% of the primary stresses; for $L_U = 0.5\,D$, it takes only 12% of the primary stresses.

For very soft ground requiring immediate support (as in the case of very shallow tunnels), almost 100% of the primary stresses are acting on the lining. The values change, of course, with other stiffness relationships and other stress distributions than those shown in Fig. 3, with other cross-sections, and other tunnelling methods.

2.4. *Site Investigations, Structural Analysis and* In-Situ *Monitoring*

An adequate intensity of site exploration, from which geological and hydrological mappings and ground profiles are derived, is most important for choosing the appropriate tunnel design and excavation method. A well-documented geological report should provide as much information as is obtainable about the physical features along the tunnel axis and in the adjacent ground. The amount of information should be much greater than the information required for entering directly into a structural analysis.

The results of an analysis depend very much on the assumed model and the values of the significant parameters. The main purposes of the structural analysis are to provide the design engineer with: (1) a better understanding of the ground-structure interaction induced by the tunnelling process; (2) knowledge of what kinds of principal risks are involved and where they are located; and (3) a tool for interpretating the site observations and the *in-situ* measurements.

The available mathematical methods of analysis are much more refined than are the properties that constitute the structural model. Hence, in most cases it is more appropriate to investigate alternative possible properties of the model, or even different models, than to aim for a more refined model. For most cases, it is preferable that the structural model employed and the parameters chosen for the analyses be lower-limit cases that may prove that even for unfavourable assumptions, the tunnelling process and the final tunnel are sufficiently safe. In general, the structural design model does not try to represent exactly the very actual conditions in the tunnel, although it covers these conditions.

In-situ monitoring is important and should be an integral part of the design procedure, especially in cases where stability of the tunnel depends on the ground properties. Deformations and displacements generally can be measured with much more accuracy than stresses. The geometry of the deformations and their development over time are most significant for the interpretation of the actual events. However, *in-situ* monitoring evaluates only the very local and actual situation in the tunnel. Therefore, in general the conditions taken into account by the design calculations do not coincide with the conditions that are monitored. Only by relating measurement results and possible failure modes by extrapolating can the engineer arrive at considerations of safety margins.

In many cases, exploratory tunnelling may be rewarding because of the information it yields on the actual response of the ground to the proposed methods for drainage, excavation, TBM driving, support, etc. In important cases a pilot tunnel may be driven; such a tunnel may even be enlarged to the full final tunnel cross-section in the most representative ground along the tunnel axis. For larger projects, it may be useful to excavate a trial tunnel prior to commencing the actual work. More intensive *in-situ* monitoring of the exploratory tunnel sections should check the design approach by numerical analysis.

2.5. *Design Criteria and Evaluating Structural Safety*

An underground structure may lose its serviceability or its structural safety in the following cases:

— The structure loses its watertightness.
— The deformations are intolerably large.
— The tunnel is insufficiently durable for its projected life and use.
— The material strength of the structural elements is exhausted locally, necessitating repair.
— The support technique (for example, in erecting segmental linings) fails or causes damage.
— Exhaustion of the material strength of the system causes structural failure, although the corresponding deformations develop in a restrained manner over time.
— The tunnel collapses suddenly because of instability.

The structural design model should yield criteria related to failure cases, against which the tunnel should be designed safely. These criteria may be:

- Deformations and strains.
- Stresses and utilization of plasticity.
- Cross-sectional lining failure.
- Failure of ground or rock strength.
- Limit-analysis failure modes.

In principle, the safety margins may be chosen differently for each of the failure cases listed above. However, in reality the evaluation of the actual safety margins is most complex and very much affected by the scattering of the involved properties of the ground and the structure and, furthermore, by the interacting probabilistic characteristics of these properties. Therefore, the results of any calculation should be subject to critical reflection on their relevance to the actual conditions.[1]

National codes for concrete or steel structures may not always be appropriate for the design of tunnels and the supporting elements. Computational safety evaluations should always be complemented by overall safety considerations and risk assessments employing critical engineering judgment, which may include the following aspects:

- The ground characteristics should be considered in light of their possible deviations from average values.
- The design model itself and the values of parameters should be discussed by the design team, which includes all of the experts involved (see Section 2.1, "General Procedure in Designing a Tunnel," above).
- Several and more simple calculation runs with parametric variations may uncover the scattering of the results. In general, this approach is much more informative than a single over-refined investigation.
- The *in-situ* measurements should be used for successive adjustment of design models.
- Long-term measurement of deformations via extrapolation may reveal to a large extent the final stability of the structure, although sudden collapse may not be announced in advance.

3. Site Investigations and Ground Probings

3.1. *Geological Data and Ground Parameters*

The appropriate amount of ground investigations on site and in laboratories may vary considerably from project to project. Because the types of ground explorations and probings depend on the special features of the tunnelling project, its purpose, excavation method, etc., they should be chosen by the expert team, especially in consultation with the design engineer. The intensity of the ground explorations will depend on the homogeneity of the ground, the purpose of the tunnelling, the cost of boring, e.g. for shallow or deep cover, and other factors.

The geological investigations should include the following basic geotechnical information (see also ISRM Commission on Classification of Rocks and Rock Masses 1981).

3.1.1. Tunnels in rock

Zoning. The ground should be divided in geotechnical units for which the design characteristics may be considered uniform. However, relevant characteristics may display considerable variations within a geotechnical unit. The following aspects should be considered for the geological description of each zone:

- Name of the geological formation in accordance with a genetic classification.
- Geologic structure and fracturing of the rock mass with strike and dip orientations.
- Colour, texture and mineral composition.
- Degree of weathering.

Parameters of the rock mass e.g. in five classes of intervals, including:

- Thickness of the layers.
- Fracture intercept.
- Rock classification.
- Core recovery.
- Uniaxial compressive strength of the rock, derived from laboratory tests.
- Angle of friction of the fractures (derived from laboratory direct shear tests).
- Strength of the ground in on-site situations.
- Deformation properties (modulus).
- Effect of water on the rock quality.
- Seismic velocity.

Primary stress field of the ground. For larger tunnel projects, tests evaluating the natural stresses in the rock mass may be recommended. For usual tunnel projects one should least estimate the stress ratio σ_h/σ_v at tunnel level, where σ_h is the lateral ground pressure and σ_v the major principal stress (usually in the vertical direction), for which the weight of the overlying rock generally may be taken. Tectonic stresses should be indicated.

Water conditions. Two types of information about water conditions are required:
(1) Permeability, as determined by:
Coefficient k (m/s) (from field tests).
Lugeon unit (from tests in boreholes).
(2) Water pressure:
At the tunnel level (hydraulic head).
At piezometric levels in boreholes.

Deformability of the rock mass. In-situ tests are required to derive the two different deformation moduli, which can be determined either from static methods (dilatometer tests in boreholes, plate tests in adits, or radial jacking tests in chambers) or from dynamic methods (wave velocity by seismic-refraction or by geophysical logging in boreholes). Engineering judgment should be exercised in choosing the value of the modulus most appropriate for the design—for instance, by the relevant tangent of the pressure-deformation curve at the primary stress level in the static method.

Properties for which information is needed when tunnel boring machines are to be employed include:

— Abrasiveness and hardness.
— Mineral composites, as, e.g. quartzite contents.
— Homogeneity.

Swelling potential of the rock. The presence of sulfates, hydroxydes, or clay minerals should be investigated by mineralogical testing. A special odeometer test may be used to determine the swell test-curve of a specimen subjected first to a load-unload-reload cycle in a dry state, and then unloaded with water.

The following *ground water conditions* should be given:

- Water levels, piezometric levels, variations over time, pore pressure measurements in confined aquifers.
- Water chemistry.
- Water temperatures.
- Expected amount of water inflow.

3.1.2. Tunnels in soil

The geotechnical description should primarily follow the recommendations given above for rock. Additional special features for soil include:

1. *Soil identification* (laboratory testing):

- Particle size distribution.
- Atterberg limits w_l, w_p.
- Unit weights, γ, γd, γz.
- Water content w.
- Permeability k.
- Core recovery.

2. *Mechanical properties* determined by laboratory testing:

- Friction angle ϕu, ϕ.
- Cohesion c_u, c.
- Compressibility m_v, c_v.

3. *Mechanical properties* detemined by field testing:

- Shear strength τ_v (Vane-test).
- Penetration N (Standard Penetration Test).
- Deformability E (Plate bearing, Dilatometer).

4. *Ground water condition* (in addition to those listed in 3.1.1.): permeability, as determined by pumping tests.

3.2. *Evaluation of Parameters by Ground Probing and Laboratory Tests*

The properties of the ground that are relevant for the tunnel design should be evaluated as carefully as possible. *In-situ* tests, which cover larger ground masses, generally are more significant than are laboratory tests on small specimens, which often are the better preserved parts of the coring. The natural scattering of ground properties requires an appropriate number of parallel tests—at least three tests for each property (see also the corresponding ISRM recommendations).

Results of laboratory tests must be adjusted to site conditions. The size of specimen, the effects of ground water, the inhomogeneity of the ground on site, and the effects of scattering must be considered. The conclusions drawn from tests also should take into consideration whether the specimens were taken from disturbed or undisturbed ground.

In many cases, the first part of the tunnelling may be interpreted as a large-scale test, the experiences from which may be drawn upon not only for the subsequent excavations but also for predicting ground behaviour. In certain cases, long horizontal boreholes may facilitate ground probing ahead of the face, or a pilot tunnel may serve as a test tunnel that at the same time provides drainage. The on-site investigations provide valuable results for checking the correlation of large-scale *in-situ* tests with laboratory tests.

Special tests that correspond directly to the proposed tunnelling method may be required, e.g. for the sufficient preservation of a membrane at the face of a bentonite shield. The evaluation of the parameters should indicate the expected scattering. From probabilistic consideration of normally distributed quantities it can be deduced that a mean value or a value corresponding to a moderately conservative fractile of a Gaussian distribution is more appropriate than the worst case value.

A set of all the parameters describing the ground behaviour of one tunnel section with regard to tunnelling should be seen as a comprehensive unit and should be well-balanced in relation to each of the parameters. For example, a small value of ground deformation modulus indicates a tendency to plastic behaviour, to which corresponds a ratio of lateral to vertical primary stress that is closer to 1.0. Hence, for alternative investigations some complete, balanced sets of parameters should be chosen instead of considering each parameter alone, unrelated to the others.

The available methods for ground probing and laboratory tests, their applicability and accuracy are given in the Appendix.

3.3. *Interpretation of Test Results and Documentation*

The field and laboratory tests should be given in well-documented reports, in the form of actual results. Based on these reports, an interpretation of the tests that is relevant to the actual tunnelling process and the requirements of the design models for the structural analysis is necessary. At the time the tests are planned, the team of experts referred to in Section 2.1 should decide which ground properties and ground characteristics are necessary for the general geotechnical description of the ground and for the projected design model. Thus, a closer relationship may be achieved between ground investigations and tunnelling design, and between the amount and refinement of tests and the tunnelling risks.

The documents should lay open the rational interpretational way in which design values are derived from test results. This method has proven to be especially useful in the tendering process, because it condenses the relevant data for the description of the ground and for the design of the tunnel on a band along the tunnel axis beneath a graphical representation of the tunnel profile (see the examples in Figs 9-13).

Such condensed tables may be prepared first for tendering and the preliminary design, and then improved through experience gained and incoming monitoring results. However, it should be clearly stated, especially in the contract papers, that much relevant information is lost or oversimplified in such tables, and that therefore the geotechnical reports and other complete documents should be considered the primary documents.

4. On Structural Design Models for Tunnelling

4.1. *Alternative Design Models*

The excavation of a tunnel changes the primary stress field into a three-dimensional pattern at the tunnelling face. Farther from the face, the stress field eventually will return to an essentially two-dimensional system. Therefore, the tunnel design may consider only two-dimensional stress-strain fields as first approximations.

The design of a tunnel should take into account the interaction between ground and lining. In order to do so, the lining must be placed in closest possible bond with the ground. To preserve its natural strength, the ground should be kept as undisturbed as possible. The deformations resulting from the tunnelling process (see Fig. 2) reduce the primary ground pressure and create stresses in the lining corresponding to that fractional part of the primary stresses in the ground which act on the sustaining lining. The stresses depend on the stiffness relationship of the ground to the lining, as well as on the shape of the tunnel cross-section. The latter should be selected such that an arching action in the ground and the lining may develop.

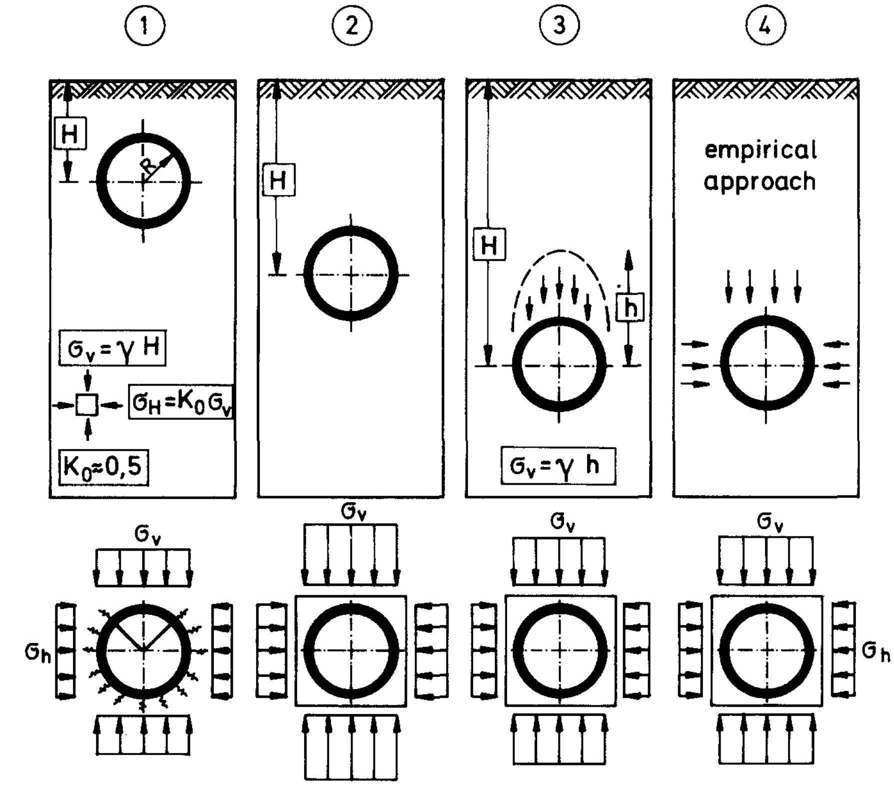


Figure 4. Alternative plane-strain design models for different depths and ground stiffnesses.

Figure 4 presents four different structural models for a plane-strain design analysis. The cross-sections need not be circular. These four models are explained more explicitly below.

In soft ground, immediate support is provided by a relatively stiff lining. For tunnels at shallow depth (as for underground railways in cities), it is agreed that a two-dimensional cross-section may be considered, neglecting the three-dimensional stress release at the face of the tunnel during excavation. In cases (1) and (2) in Fig. 4, the ground pressures acting on the cross-section are assumed to be equal to the primary stresses in the undisturbed ground. Hence, it is assumed that in the final state (some years after the construction of the tunnel), the ground eventually will return to nearly the same condition as before the tunnelling. Changes in ground water levels, traffic vibrations, etc., may provoke this "readjustment."

In case (1), for shallow tunnels and soft ground, the full overburden is taken as load. Hence, no tension bedding is allowed at the crown of the tunnel. The ground reaction is simplified by radial and tangential springs, arriving at a bedded-beam model.

In case (2), for moderately stiff ground, the soil stiffness is employed by assuming a two-dimensional continuum model and a complete bond between lining and ground. As in case (1), stress release due to predeformations of the ground is neglected. Inward displacements result in a reduction of the pressure on the lining.

Case (3) assumes that some stress release is caused by deformations that occur before the lining participates. In medium-hard rock or in highly cohesive soil, the ground may be strong enough to allow a certain unsupported section at the tunnel face (see Fig. 2). Also, for tunnels having a high overburden, a reduction of the acting crown pressure (represented in Fig. 4 by $h < H$) is taken into account.

In case (4), the ground stresses acting on the lining are determined by an empirical approach, which may be based on previous experiences with the same ground and the same tunnelling method, on *in-situ* observations and monitoring of initial tunnel sections, on interpretation of the observed data, and on continuous improvements of the design model.

If a plane model is not justified—as is the case for caverns, for more complicated geometries of underground structures, or for an investigation directly at the tunnelling face—a three-dimensional model may be necessary (see Fig. 5). The three-dimensional model also may be conceived as consisting of discontinuous masses (block theory) or a continuum with discrete discontinuous fissures or faults.

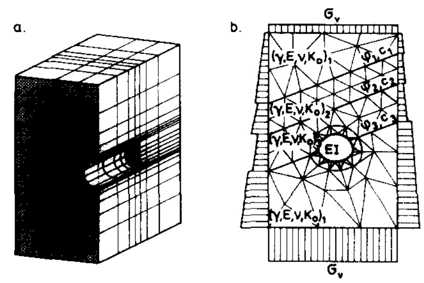


Figure 5a. Three-dimensional continuum model.
Figure 5b. Example of two-dimensional finite-element model.

4.2. Continuum or Discontinuum Model

For structural design models such as those in Figs 5a and b, the ground may be modelled as homogeneous or heterogeneous, isotropic or anisotropic; as a two-dimensional, i.e. allowing some stress release before the lining is acting, or a three-dimensional stiff medium. The lining may be modelled either as a beam element with bending stiffness or as a continuum. Plasticity, viscosity, fracture of the rock, non-linear stress-strain and deformation behaviour, etc., may be covered by special assumptions for material laws.

The design criteria are computed by numerical solutions. From their origins, the finite-element method and the boundary-element method are basically continuum methods. Thus, homogeneous media and stress-strain fields are evaluated best. In general, discontinua such as rock with fissures and faults, and failure modes, which are initiated by local rupture, shear failure, or full collapse, cannot be covered by continuum methods.

A continuum or discontinuum model is appropriate for tunnel structures where the ground provides the principal stability of the opening (as in hard rock) or where the geometrical properties of the underground opening can be modelled only by numerical analysis, e.g. in the case of closely spaced twin tunnels.

4.3. *Bedded-Beam Model (Action-Reaction Model)*

If the stiffness of the ground is small compared to the stiffness of the lining, a design model such as that shown in Fig. 6 may be employed. In such a case, the active ground pressures are represented by given loads and the passive reaction of the ground against deformations is simulated by constant bedding moduli. The model may be particularly well-suited to the design of linings of shield-driven tunnels. As to applicability, the stiffness ratio β may be smaller than 200:

$$\beta = E_s R^3 / EJ < 200,$$

where: E_s is the representative deformation stiffness modulus of the ground,
R is the radius of the tunnel cross-section or its equivalent for non-circular tunnels,
EJ is the bending stiffness of the lining.

A more correct solution for the bedding is given by a non-zero stiffness matrix for all elements with regard to radial and tangential displacements.

However, in most cases and in view of the unavoidable approximations based on the other assumptions, a simpler approach may be sufficient. Such an approach considers only radial (and, eventually, tangential) bedding, neglecting the interdependence of radial and tangential displacements and beddings. For non-circular cross-sections, the continuum solution reveals that bedding may be increased at corner sections of the lining, with smaller radius of the curvature.

The bedded-beam model may be adjusted to more complex cases, e.g. by reducing the crown load in accordance with stress release at the tunnel face (see Fig. 3) or, for deep tunnels, by assuming bedding also at the crown.

For articulated effective hinges in linings the bending moments are smaller; the deformations may be larger, depending on the ground stiffness. For hinged linings the limit of β given above is not valid.

The analysis of the bedded beam yields ring forces, bending moments, and deformations as design criteria for the lining. If the lining ring is completely closed, the bending moments may be considered less important than the ring forces for providing equilibrium (a smaller safety factor may be justified for the bending moments). Allowances also may be made for a plastic rotation capacity of the lining segments.

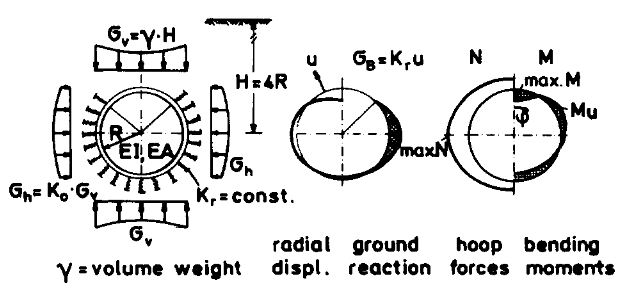


Figure 6. Example of a bedded-beam model for shallow tunnels.

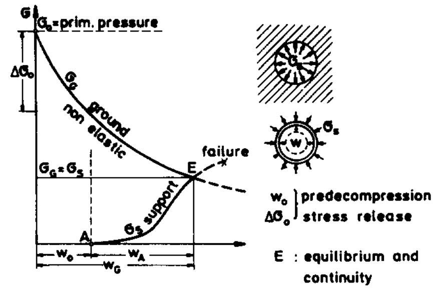


Figure 7. Characteristic curves for the ground and the support for convergence-confinement models (Fenner-Pacher curves).

For tunnels with very pronounced stress release due to inward deformations, e.g. for deep tunnels in rock, a simple approach to design considerations is given by the convergence-confinement model, which is based only on the interaction of the radial inward displacement and the support reaction to these deformations by resisting ring forces and the corresponding outward pressure (see Fig. 7).

The primary stresses σ_0 in the ground are released with progressive inward displacements. The acting pressure may even increase when rock joints are opening with larger displacements. In self-supporting rock, the ground characteristic in Fig. 7 meets the w-axis; because the primary stresses are released completely, a supporting lining is not necessary. Before the supporting members are installed, it is unavoidable—even desirable—that decompression associated with the predeformation w_0 will occur. The stiffness of the lining determines where both curves (characteristic lines) will intersect. At this point, equilibrium as well as compatibility conditions are fulfilled. If the ground characteristic is known, e.g., by *in-situ* monitoring, the predeformation w_0 and the stiffness of the lining (including its development over time and as tunnelling advances), and even its plastic properties are very decisive for the actual stresses in the lining. Both curves in Fig. 7 may vary considerably.

In its usual analytical form, the convergence-confinement model assumes constant ground pressure along a circular tunnel lining. Consequently, it yields only ring forces and no bending moments at all. However, it may be extended to cover ground pressures that vary along the tunnel lining (Gesta 1986).

The model may also be applied as a first approximation for non-circular tunnel cross-sections, although the support reaction curve is distinctly different, e.g. for horseshoe-type cross sections. Therefore, it may be helpful to use the convergence-confinement model in combination with a continuum model and *in-situ* measurements.

Although the convergence-confinement approach is primarily a tool for the interpretation of field measurements, it also may be applied in support of the empirical approach.

4.4. *Empirical Approach*

The structural elements and the excavation procedure, especially for the preliminary support of the tunnel, may be selected mainly based on experience and empirical considerations that rely more on direct observations than on numerical calculations. This procedure may be especially reasonable if experiences from a successful tunnelling project can be applied to a similar, new one yet to be designed. Such a transfer of information is justified only when:

- The ground conditions, including those of the ground water, are comparable.
- The dimensions of the tunnel and its cross-sectional shape are similar.
- The depths of overburden are approximately the same.

- The tunnelling methods to be employed are the same.
- *In-situ* monitoring yields results comparable to those for the preceding tunnelling project.

One disadvantage of prolonged application of the empirical approach is that, lacking an incentive to apply a more appropriate tunnelling design via a consistent safety assessment, the structure may be designed overconservatively, resulting in higher construction costs. The simple empirical approach contributes little to the advancement of the state of the art in tunnelling.

The empirical approach to tunnel design may also be applied to larger projects in only slightly changing ground if provision is made (especially in the tender) for initial experiences to be extrapolated to the subsequent sections along the tunnel axis. Such a situation justifies a measurement programme that is more intensive for the first sections, in order to gain experience.

4.5. *Observational Method*

By combining analytical methods with the empirical approach and the immediate interpretations of *in-situ* measurements, a tunnelling design procedure that is adjustable as the tunnel excavation proceeds may be applied. In this approach, the field measurements of ground movements, displacements and stresses in the lining are used on an ongoing basis to verify or modify the design of the tunnel. More intensively instrumented sections at the early stages of the tunnelling provide the data for these procedures. The interpretation of the measured data yields insight into the ground behaviour as a reaction to the tunnelling procedure.

In applying the observational method, the following conditions must be met:

- The chosen tunnelling process must be adjustable along the tunnel line.
- Owner and contractor must agree in advance on contractual arrangements that allow for modifications of the design on an ongoing basis during the project.
- The field measurements should be interpreted on the basis of a suitable analytical concept relating measurement data to design criteria.
- The interpretation of a particular instrumented section must be used to draw conclusions about the other sections of the tunnel. Hence, the experiences are restricted to those tunnel sections that are comparable with respect to ground conditions, ground cover, etc. (see Section 4.4 "Empirical Approach").
- Field measurement should be provided throughout the entire length of the tunnel in order to check its assumed behaviour.

4.6. *Special Design Features*

Special considerations may be necessary if unusual ground behaviour is expected or is caused by ground improvements. Some special design features and considerations are discussed below.

4.6.1. Ground improvement techniques

Grouting and injections. Intensive grouting or injections of the ground may improve the ground characteristics considered in the design model. Although in most cases grouting is applied only for closing discontinuities in rock or for strengthening soft ground, in both cases the goal is to achieve better homogeneity.

Drainage and compressed air. Usually the ground is stabilized by dewatering it and by avoiding inflows of water. Ground failure may be avoided if the pore water pressure is minimized. The assumed ground characteristics may be valid only if successful drainage is possible or if water inflow is prevented, as in tunnelling under compressed air.

Ground freezing. Improving the ground by freezing changes the ground properties. The time-dependent stress-strain behaviour of frozen ground can be significant. Freezing draws water toward the lining, causing an increase in water volume and heave at the surface. Concreting on frozen ground delays the strength development of the concrete.

4.6.2. Unusual ground behaviour

Swelling ground. Stress release due to tunnelling and/or ground water influx may cause swelling and a corresponding increase in pressure on the lining. In these cases, a circular cross-section or at least an invert arch is recommended. The swelling resulting from a chemical reaction, as in anhydrid, generally is much more pronounced than that due to the physical absorption of water, as in clay.

Underground erosion, mining subsidence, and sinkholes. Tunnelling in ground that is subject to settlements, as in the case of gypsum erosion or mining subsidence, requires special design considerations. A flexible lining that follows the ground movements by utilizing its plastic deformation capacity is more suitable in these cases than is a too-rigid or brittle, failure-prone lining. If the ground has sinkhole potentials, a tunnel structure that can be repaired easily may be more economical than a structure designed to allow the bridging of the sinkholes.

5. *In-Situ* Monitoring

5.1. *Purpose of* In-Situ *Measurements*

In-situ monitoring during the excavation and at longer intervals after the tunnel is completed should be regarded as an integral part of the design not only for checking the structural safety and the applied design model but also for verifying the basic conception of the response of the ground to tunnelling and the effectiveness of the structural support.

The main objectives of *in-situ* monitoring are:

(1) To control the deformations of the tunnel, including securing the open tunnel profile. The time-history development of displacements and convergences may be considered one safety criterion, although field measurements do not yield the margins the structure can endure before failing.

(2) To verify that the appropriate tunnelling method was selected.

(3) To control the settlements at the surface, e.g. in order to obtain information on the deformation pattern in the ground and on that part of settlements caused by lowering the water level.

(4) To measure the development of stresses in the structural members, indicating sufficient strength or the possibility of strength failure.

(5) To indicate progressive deformations, which require immediate action for ground and support strengthening.

(6) To furnish evidence for insurance claims, e.g. by providing results of levelling the settlements at the surface in town areas.

5.2. *Monitoring Methods*

A programme for monitoring the deformations and stresses during the excavation may comprise the following measurements (see Fig. 8):

(1) Levelling the crown (at the least) inside the tunnel as soon as possible. With regard to interpretation of the data, Fig. 2 reveals that often only a small fraction of the entire crown movement can be monitored because a larger part occurs before the bolt can be set. For difficult tunnelling, the distance between two crown readings may be as close as 10-15 m. Levelling of the invert is recommended for rock having swelling potentials.

(2) Convergence readings (in triangular settings; K in Fig.

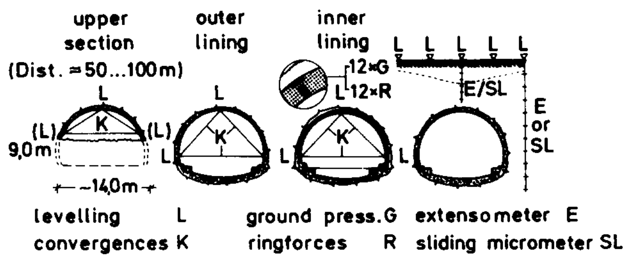


Figure 8. Example of in-situ *monitoring of the tunnel excavation, the preliminary lining, and the surface settlements.*

8) should be the standard method for early information. They are easily applied and are accurate to within 1 mm.

(3) In a few cross-sections, the linings may be equipped with stress cells for reading the ground pressures and ring forces in the lining (*G* and *R* in Fig. 8).

(4) Stress cells also should be installed in a few sections of the final second lining if long-term readings are desired after the tunnel has been completed.

(5) Surface levelling along the tunnel axis and perpendicular to it yield settlements and the correlation to measurements inside the tunnel (see Fig. 2).

(6) Extensometers, inclinometers, gliding micrometers may be installed from the surface well ahead of the tunnelling face, yielding deformation measurements within the ground (see Fig. 8). Monitoring of the ground deformations is especially appropriate for checking and interpreting the design model. Therefore, the installation should be combined with convergence readings and stress cells in the same cross-section.

The frequency of the readings depends on how far from the tunnelling face the measurements are taken, and on the results. For example, readings may be performed initially two times a day; then be reduced to one reading per week four diameters behind the face; and end with one reading per month if the time-data curves justify this reduction in measurement readings.

5.3. *Interpreting Results of* In-Situ *Monitoring*

The results of *in-situ* monitoring should be interpreted with regard to the excavation steps, the structural support work, and the structural design model in conjunction with safety considerations.

The actual readings normally show a broad scatter of values. Expectations of reliability may not be met, especially for pressure cells, because stresses and strains are very local characteristics. Deformation and convergence readings are more reliably obtainable because displacements register integrals along a larger section of the ground.

The *in-situ* measurements should be interpreted in consideration of the following:

- The results should verify whether the tunnelling method is appropriate.
- Graphed time-history charts may reveal a decreasing rate of deformation, or uncover danger of collapse.
- Large discrepancies between the theoretically predicted and actually observed deformations may force revision of the design model. However, measurements are valid only for the actual state at the time and the place where they are taken. Long-term influences such as rising water level, traffic vibrations, and long-term creep are not registered during excavation.
- The readings may promote visual understanding of the structural behavior of ground and support interaction.
- The readings may cover only a fraction of the actual phenomena if bolts and stress cells are installed too late (see Fig. 2).
- The tunnel may be considered stable when all the readings cease to increase. However, a safety margin against failure—especially sudden collapse—cannot be deduced from measurement, except by extrapolation.

6. Guidelines for the Structural Detailing of the Lining

On design aspects with regard to maintenance the reader is referred to other recommendations of the ITA (see *T&UST* 2:3). For concrete linings, the following structural design specifications are suggested.

(1) The thickness of a second lining of cast-in-place concrete may have a lower limit of 25-30 cm to avoid concrete placing problems such as undercompaction or honeycombing of concrete. The following lower limits may be recommended:

—20 cm, if lining is unreinforced;
—25 cm, if lining is reinforced;
—30 cm for watertight concrete.

(2) Reinforcement may be desirable for crack control, even when it is not required for covering inner stresses. On the other hand, reinforcement may cause concrete-placing problems or long-term durability problems due to steel corrosion. If reinforcement in the second lining is provided for crack control, a closely-spaced steel mesh reinforcement may have the following cross-sections in both directions:

- At the outer surface, at least 1.5 cm^3/m of steel;
- At the inner surface, at least 3.0 cm^3/m of steel.

(3) The recommended minimum cover of reinforcement is:

3.0 cm	At the outer surface if a waterproof membrane is provided.
5.0 cm–6.0 cm	At the outer surface if it is directly in contact with the ground and ground water.
4.0 cm–5.0 cm	At the inner tunnel surface.
5.0 cm	For the tunnel invert and where water is aggressive.

(4) For lining segments, specifications (1), (2) and (3) above are not valid, especially if the segmented tunnel ring is the outer preliminary lining. For detailing the tunnel segments, special attention should be given to avoiding damage during transport and erection.

(5) Sealing against water (waterproofing sheets) may be necessary under the following conditions:

- When aggressive water action threatens to damage concrete and steel.
- When the water pressure level is more than 15 m above the crown.
- When there is a possibility of freezing of ingressing water along the tunnel section close to the portals.
- When the inner installations of the tunnel must be protected.

(6) In achieving watertightness of concrete, special specifications of the concrete mixture, avoidance of shrinkage stresses and temperature gradients during setting, and the final quality of the concrete are much more important than theoretical computations of crack widths.

(7) Temperature effects (tension stresses) may be somewhat controlled by working joints (as close as 5 m at the portals) and by additional surface reinforcement in concrete exposed to low temperatures.

(8) An initial lining of shotcrete may be considered to participate in providing stability of the tunnel only when the long-term durability of the shotcrete is preserved. Requirements for achieving long-term durability include the absence of aggressive water, the limitation of concrete additives for accelerating the setting (liquid accelerators), and avoiding shotcrete shadows behind steel arches and reinforcements.

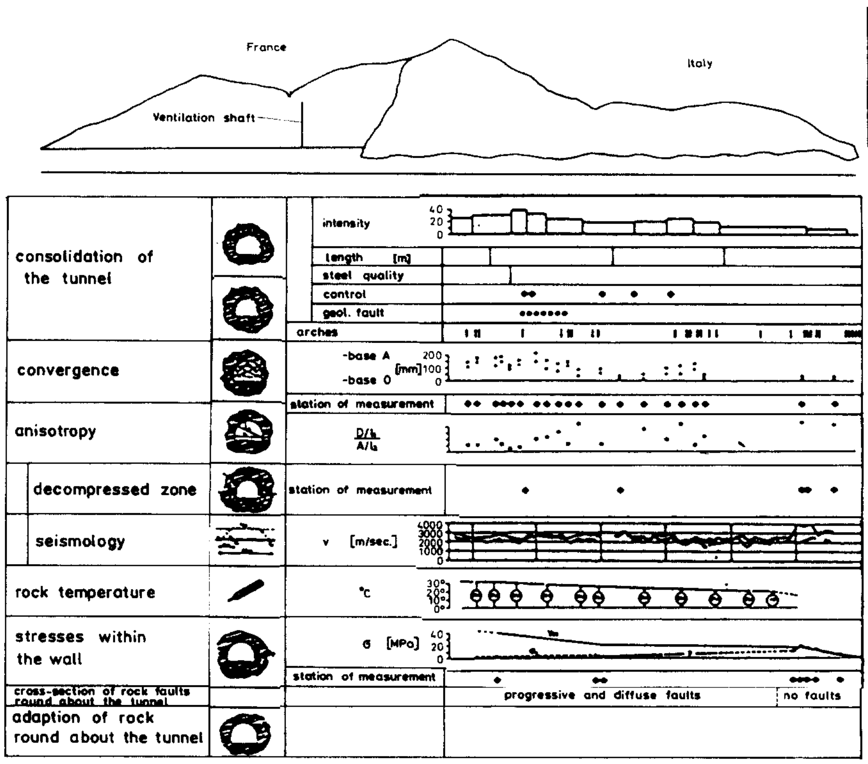


Figure 9. Table of measured data and encountered conditions along a tunnel in France.

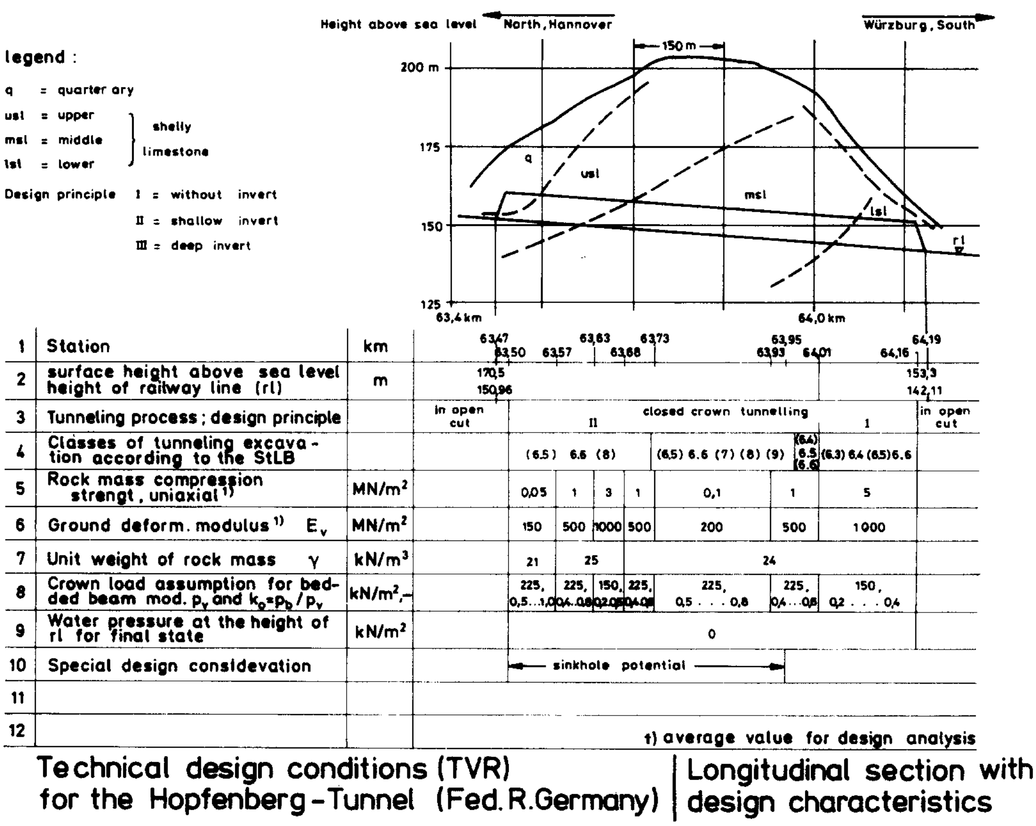
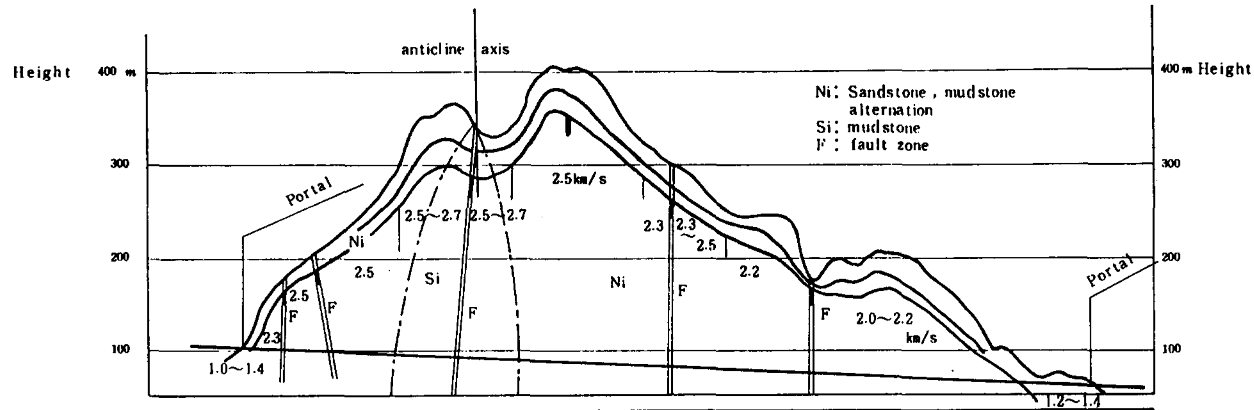


Figure 10. Predicted ground conditions, tunnelling classes and design characteristics along a tunnel of the rapid railway line in Germany.



Kilo meter	41km 500 600 800; 41km 200 400 600 800; 42km 200 400 600 800; 43km 200 400 600 800; 44km 200
Geological Formation	Ni: Nishiyama Formation; Si: Shiiya Formation; Ni; 泥岩優勢
Rock name	alt. of s.s. and m.s.; s.s.and m.s.(s.p.); s.s.and m.s. (s.p.); s.s.and m.s.; s.s.and m.s. (s.p.); m.s.; s.s.,m.s. (s p); s s
Seismic Velocity	1.4~1.0; 2.3; 1.7 2.5; 2.0 2.5; 2.5~2.7; 1.7 2.5~2.7; 1.5 2.5; 2.5; 1.6 2.3; 2.3~2.5; 2.2; 1.5; 2.0~2.2; 1.2~1.4
Unconfined Compressive Strength (Competence Factor)	σ_c=44~58 kg/cm² (7.0~9.7); σ_c=54~64 kg/cm² (1.0~1.2); σ_c=61~108 kg/cm² (1.7~3.0); σ_c=52~56 kg/cm² (4.5~4.8)
Water Inflow	a little
Rock Class	IL; IN; IS; IN; IIN; IL
Note	Squeezing Property

s.s.: sandstone m.s.: mudstone s.p.: sandstone predominates

Figure 11. Predicted ground conditions along a tunnel line (example submitted by Japan).

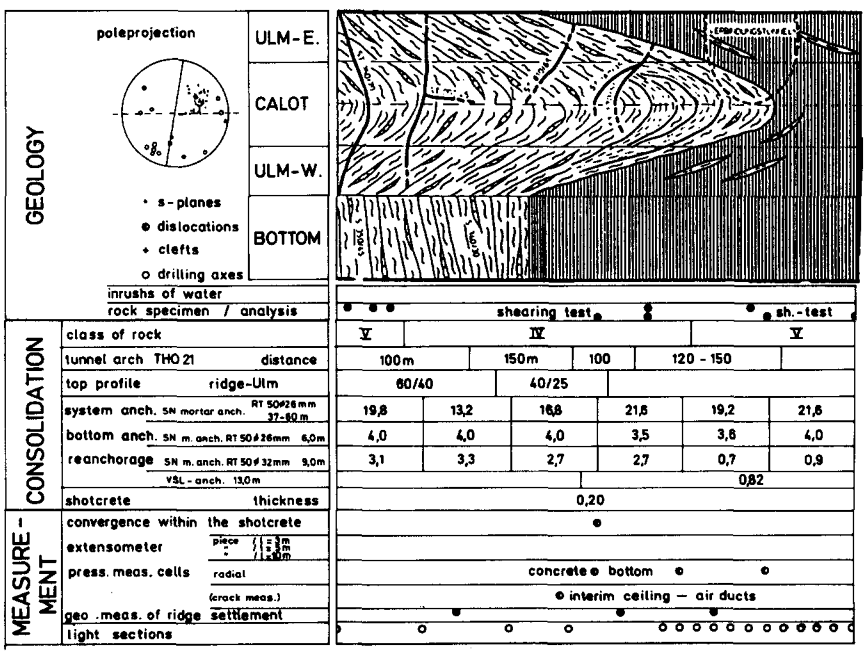


Figure 12. Documentation of geology, ground classes, support, geotechnical field measurements gathered during a tunnel project in Austria.

7. Examples of Presentation of Tunnel Design Data

Figures 9-12 are national examples of tabulated information on geotechnical conditions and design characteristics given in condensed form along a longitudinal tunnel section. This information may be part of the tendering documents and should be amended with ongoing tunnelling. By gathering the data actually encountered along the tunnel line in a similar table, a comparison can be made between predicted and actual tunnelling conditions. □

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Note

[1]See, for example, the Swiss SIA Dokument 260 or the corresponding U.S.-ASCE Code.

Appendix. International and National Recommendations on Structural Design of Tunnels.

Although the following selected list of recommendations by national and international organizations is not complete, it nevertheless should provide the reader with sources of additional information regarding the design of tunnels.

Organization/Country	**Publication**
International Tunnelling Association (ITA).	Views on structural design models for tunnelling. *Advances in Tunnelling Technology and Subsurface Use* 2:3 (1982).
International Society for Rock Mechanics (ISRM)	ISRM recommendations on site investigation techniques, July 1975.
	ISRM Committee on Field Tests: Document No. 1—Suggested Method for Determining Shear Strength
	Document No. 2—Suggested Methods for Rock Bolt Testing
	ISRM Committee on Laboratory Tests; ISRM Committee on Swelling Rocks: Document No. 1—Suggested Methods for Determining the Uniaxial Compressive Strength of Rock Materials and Point Load Strength Index.
	Document No. 2—Suggested Methods for Determining Water Content, Porosity, Density, Absorption and Related Properties, Swelling and Slake Durability Index Properties.
Australia	Australian Standard 1726 - S.A.A. Site Investigation Code.
	Australian Standard 1289 - Methods of Testing Soils for Engineering Purposes.
Austria	ÖNORM B 2203 Untertagebaunorm, Richtlinien und Vertragsbestimmungen, Werkvertragsnorm.
	Projektierungsrichtlinien für Geotechnische Arbeiten. RVS 9.240 u. 9.241, Forschungsges. Srassenwesen. Nov. 1977.
Federal Republic of Germany (in German)	Recommendations for the design of underground openings in rock. *Tunnelbau-Taschenbuch 1980*, Gluckauf-Verlag, Essen (1980), pp. 157-239.
	Recommendations for the analysis of Tunnels in soft ground (1980), *Bautechnik 10* (1980), Berlin, pp. 349-356.
	Recommendations for the Concrete Lining of Tunnels in soft ground (1986). *Bautechnik 10* (1986), Berlin, pp. 331-338.
France	*Tunnels et Ouvrages Souterrains*, Special Issue July 1982, pp. 32-123:
	Réflexions sur les méthodes usuelles de calcul du revêtement des souterrains (Usual calculation methods for the design of tunnel linings).

	Présentation de la méthode de construction des tunnel avec soutènement immediat par béton projeté et boulonnage (Presentation of the tunnel construction method with immediate support by shotcrete and bolting).
	Recommandations sur les conditions d'emploi du boulonnage (Recommendations for conditions of the use of bolting).
	Tunnels et Ouvrages Souterrains 73 (Jan./Feb. 1986), pp. 18–38: Recommendations for use of the convergence-confinement method.
	Tunnels et Ouvrages Souterrains 67 (Jan./Feb. 1985), pp. 32–43: Recommandations relatives au choix d'un type de soutènement en galerie (Recommendations for the selection of tunnel support).
	Tunnels et Ouvrages (1984), pp. 80–97: Recommandations relatives à l'emploi des citres dans la construction des ouvrages souterrains (Recommendations on the use of steel arches as temporary support in tunnel structures).
Japan Tunnel Engineering Committee, Japan Society of Civil Engineering, Japan Tunnelling Association	Standard Specifications for Tunnels: Mountain Tunnelling Method. Nov. 1986. Shield Tunnelling Method. June 1986. Cut-and-cover Method. June 1986.
Switzerland	Recommandation SIA No. 199: Etude du massif rocheux pour les travaux souterrains. 1975. (Also in German) Norme SIA No. 198: Travaux souterrains (avancement à l'explosif). 1975. (Also in German) Recommandation SIA No. 198/1: Construction de tunnels et de galeries en rocher au moyen de tunneliers. 1985. (Also in German)
United Kingdom	British Standard 1377. Methods of test for soils for civil engineering purposes, British Standards Institution, 1975. British Standard 5930, Code of Practice for site investigations, British Standards Institution, 1981. Craig, R. N. and Muir Wood, A. M. A review of tunnel lining practice in the United Kingdom. TRRL Supplementary Report 335, 1978. Tunnelling Waterproofing. CIRIA Report 81, 1979. Dumbleton, M. J. and West, G. A guide to site investigation procedures for tunnels. TRRL Laboratory Report 740, 1976.
United States of America American Society of Civil Engineers (ASCE)	*Guidelines for Tunnel Lining Design.* Ed. by T. O'Rourke. ASCE Technical Committee on Tunnel Lining Design, Technical Council on Research.

General report on Conventional Tunnelling Method

ITA Working Group
Conventional Tunnelling

>> SUMMARY

ITA created the Working Group 19 in 2001, on the occasion of the 27th General Assembly at Milan. After collecting various national reports about the particular experiences made with Conventional Tunnelling in the member countries, after the 30th General Assembly at Singapore the working group started on the elaboration of the inter-national report and completed the Work during the 32nd General Assembly at Seoul.

The aim of the working group is to create a report as a guideline for clients, contractors and tunnelling engineers to promote international understanding by unifying the terminology and by presenting an overview of the current state of the art. The contents are valid for most parts of the world. Therefore the report highlights only the most important principles, but does not deal with the details.

The report starts with a definition of Conventional Tunnelling and illustrates some principles of Conventional Tunnelling. The high flexibility and the wide field of application of Conventional Tunnelling are highlighted.

The following main subjects will be dealt with this report:

Design, Construction Methods, Monitoring During Construction, Construction Contract and Site Organisation.

The report is applicable basically to all types of underground structures, such as traffic tunnels, caverns, hydro tunnels, pipe tunnels and shafts. However, specific questions (e.g. concerning shaft and cavern construction) are not covered.

Not all items are specific to Conventional Tunnelling but some generally applicable information has to be given to gain a better understanding.

This report is not a recipe book that would enable inexperienced readers to make decisions.

Especially in Conventional Tunnelling the knowledge of experienced engineers is essential for successful construction.

>> CONTENTS

1 >> Introduction

1.1. DEFINITION OF CONVENTIONAL TUNNELLING

The definition of what is "Conventional Tunnelling" is rather arbitrary, and subject to variations, depending on the concept adopted.
If the concept is based on excavation equipment, the term conventional tunnel could apply to any tunnel that is not excavated by a Tunnel Boring Machine (TBM). But today in tunnelling the TBM method has become very common and thus could also be regarded as "conventional".

> Conventional Tunnelling in the context of this report means the construction of underground openings of any shape with a cyclic construction process of
>
> - excavation, by using the drill and blast methods or mechanical excavators except any full face TBM
> - mucking
> - placement of the primary support elements such as
>   - steel ribs or lattice girders
>   - soil or rock bolts
>   - sprayed or cast in situ concrete, not reinforced or reinforced with wire mesh or fibres

1.2. PRINCIPLES OF CONVENTIONAL TUNNELLING

Conventional Tunnelling is carried out in a cyclic execution process of repeated steps of excavation followed by the application of relevant primary support, both of which depend on existing ground conditions and ground behaviour. An experienced team of tunnel workers (miners), assisted by standard and/or special plant and equipment shall execute each individual cycle of tunnel construction.
The Conventional Tunnelling Method mainly using standard equipment and allowing access to the tunnel excavation face at almost any time is very flexible in situations or areas that require a change in the structural analysis or in the design and as a result of this also require changes in the support measures.

A standard set of equipment for execution of Conventional Tunnelling may consist of the following items:

- Drilling jumbo to drill holes for blasting, rock bolting, water and pressure release, grouting etc.
- Road header or excavator in cases where blasting is not possible or not economic
- Lifting platform allowing the miners to reach each part of the tunnel crown and of the tunnel face
- Lifting equipment for steel sets
- Loader or excavator for loading excavated ground onto dump trucks
- Dump trucks for hauling excavated ground
- Set of shotcrete manipulators for application of wet or dry shotcrete.

Using this standard set of equipment the following changes can easily be applied during construction if ground conditions change or if monitoring results require action:

- Increase or decrease of support, e.g. the thickness of shotcrete, number and/or lengths of rock bolts per linear meter of tunnel, spacing and dimensions of steel arches, number and lengths of spiles, application of shotcrete at the tunnel face, bolting the face etc.
- Variation of ring closure time - which is the time between the excavation of a section of the tunnel and the application of partial or full support - or variation of ring closure distance from excavation face
- Introduction of primary support ring closure
- Variation of explosives charge per blasting round and variation of detonator sequences.

Other variations in the design enable one to react to changes in the stand-up time of the ground encountered:

- Increased or decreased length of excavation round (common round lengths vary from 0,5 m to 4,0 m)
- Partial excavation by splitting the excavation face into the crown, bench, and in-vert excavation steps or even further in pilot and sidewall galleries and in stag-gered bench/ invert excavations

In case exceptional ground conditions are encountered - regardless of whether predicted or not - the Conventional Tunnelling Method can react with a variety of auxiliary construction technologies like

- Grouting: consolidation grouting, fissure grouting, pressure grouting, compensation grouting
- Technologies to stabilize and improve the ground ahead of the actual tunnel face like forepoling, pipe umbrella, horizontal jet grouting, ground freezing etc.

1 >> Introduction

Conventional Tunnelling in connection with the wide variety of auxiliary construction methods enables experienced project managers to make the most appropriate choice to achieve safe and economic tunnel construction even in situations with changing or unforeseen ground conditions. It allows reacting in both directions - depending on the ground - either changing to the less favourable or towards the more favourable side. This flexibility makes Conventional Tunnelling the most advantageous tunnelling method in many projects.

Underground works, constructed by Conventional Tunnelling, include linear tunnels such as railway tunnels, motorway tunnels or hydro tunnels, but also hydroelectric caverns, underground storage caverns, metro and railway stations. They can be located at a shallow depth or under high overburden, in stable or loading ground, under genuine rock pressure, below the phreatic surface or in dry conditions.
The Conventional Tunnelling Method (CTM) is the best for projects with highly variable ground conditions or for projects with variable shapes.

Conventional Tunnelling enables:

- A greater variability of the shapes
- Better knowledge of the ground by using systematic exploratory drillings at tunnel level ahead of the face
- Greater variability in the choice of excavation methods according to the ground conditions
- Greater variability in the choice of excavation sequences according to the ground conditions
- Easier optimisation of the primary support using the observational method in special cases
- A greater variability in the choice of auxiliary construction methods according to the ground conditions

Conventional Tunnelling is especially convenient for:

- Difficult ground with highly variable ground conditions
- Projects with highly variable shapes of cross section
- Projects with a higher risk of water inflow under high pressure
- Projects with difficult access
- Short tunnels

It is the responsibility of experienced engineers to make the most appropriate choice according to the science of engineering and their personal experience for a safe and economic tunnel construction.

2.1. INTRODUCTION

The design work includes in general:
- The determination of the geometric layout of the underground structures, i.e. the horizontal and vertical alignment of the tunnels, the location and axis direction of the caverns and the choice of the tunnel system
- The determination of the shape and size of the profile (tunnel cross section)
- The determination of the type of excavation (full-face or partial-face excavation, sequence of excavation phases along the tunnel axis), of the temporary and final support measures as well as of auxiliary construction measures such as drainage or ground improvement.

The scope and degree of refinement of design depends on the design phase (Section 2.2) and on the type of contract.
The project is the result of an optimisation process involving the evaluation of variant de-signs. The aim is to determine the most economic solution for the construction, use, operation and maintenance of the underground works, taking into account:
- The planned use of the structure
- The functional requirements for the equipment
- The requirements for user safety
- The design working life
- The requirements for waterproofing
- The safety, serviceability and environmental requirements in the execution and operation phases

An adequate geological-geotechnical exploration and a thorough description of the ground in the early planning stages are important, as the ground conditions may be decisive not only for the shape of the cross section and the method of construction but also for the tunnel system and the alignment.

2.2. DESIGN PHASES

The design of a tunnel project is often subdivided into different phases according to the project stages:
- Conceptual design
- Preliminary design
- Tender design
- Final design

2.2.1. Conceptual Design
The scope of the conceptual design is to select or confirm the alignment of the tunnel and to provide the client with information for the decision-making process. Aspects of tunnelling related to a particular alignment are highlighted and investigated in detail.

2.2.2. Preliminary Design
Based on the selected alignment, the conceptual design of the project is refined and an Environmental Impact Study is carried out. The priority of the preliminary design stage is focused on the legal aspects of water resources, forestry and environmental protection.
Different clients and authorities require individual substages for railway or road tunnels. The common target however is to receive the approval for construction of the project from the authorities.

2.2.3. Tender Design
The scope of the tender design is to detail the works in such a way that the exact pricing of each work item is feasible. Also contractual documents are elaborated.

2.2.4. Final Design
The scope of the final design is the detailing of the work described in the tender stages in such a way that they can be constructed in an economical way, to be structurally safe, dimension-ally accurate and functional.

2.3. INVESTIGATION AND DESCRIPTION OF THE GROUND CONDITIONS

2.3.1.General
The geological investigations form the basis for the description of the ground. The description of the ground is required for the elaboration of a geological model that is adequate for the preparation of the geotechnical model, for the assessment of the ground, its subdivision into different geological units or homogeneous zones and the recognition and assessment of potential hazard scenarios. The characteristic properties of the ground must be reported in the geotechnical model.
Geological, hydrogeological and geotechnical investigations shall be carried out beforehand as well as supplementing during the design and construction phases and shall be geared to the construction and use of the underground structure.
The geological investigations are the owner's responsibility. The investigations should be planned and supervised by experienced engineering geologists in close cooperation with the design engineer and the owner. The elaboration of the geotechnical model is the responsibility of the engineering geologist.

2.3.2. Common exploration methods
The following well-known exploration methods are mainly employed to investigate the site conditions:
- Analysis of existing geological records (e. g. for structures already built in the same or similar geological formations)
- Field mapping
- Remote sensing
- Exploratory boreholes
- Field tests
- Laboratory tests
- Exploratory adits and galleries (pilot tunnels)
- Geophysical measurements

2.3.3. Evaluation and presentation of the results of the geological investigation

The origin of all data shall be documented in a clear and comprehensive way. It has to be stated whether the information derives from:

- Field or laboratory tests
- References to the technical literature
- Information in existing geological reports
- Empirical values
- Estimates or assumptions

Known gaps in the results presented shall be pointed out. The investigation and measurement methods shall be described. In the determination of the geotechnical properties of soil and rock using laboratory and field tests, standardised methods shall be employed if possible.

2.3.4. Extent of site investigations

The extent of the site investigations in the design phase shall be project-specific, executed in suitable steps, corresponding to the planning stage and to the complexity of the geology and taking into account economic criteria. In zones of predicted hazards (such as faults, discontinuities, cavities, etc.), in portal areas and in zones with small overburden the ground may have to be studied in greater detail.

The extent of the site investigations carried out during construction depends on the actual conditions encountered and the predicted hazards (discontinuities, cavities, occurrence of gas and water). The investigations serve to specify measures to reduce and manage the risks in accordance with the hazard scenarios.

2.3.5. Description of the geological conditions

The geological description shall be provided for each geological unit or homogeneous zone. The basis for this is given by the geological investigation. The qualitative description shall, as far as possible, be accompanied by quantitative information.

Geological units in soil are normally described as geological formations of the same origin (e.g. moraines, river gravels, weathered marl, and clay deposits). The description of the soil is based on standard classifications, combining information concerning the petrography of the components and their properties (shape, degree of roundness, degree of weathering, strength, swelling capacity etc.). The soil structure (layering, anisotropy) as well as any special features has to be described (e.g. the presence of blocks or organic constituents). The description shall be supplemented by further information, e.g. grain size distribution, permeability, density de-gree of saturation, behaviour when exposed to free water, etc.

In the case of rock, one must distinguish between rock description based on an intact specimen and the description of the rock mass as a whole. The rock specimen description in-cludes mineral content, structure and texture as well as the petrographic identification.

The description of the rock mass includes the following elements:

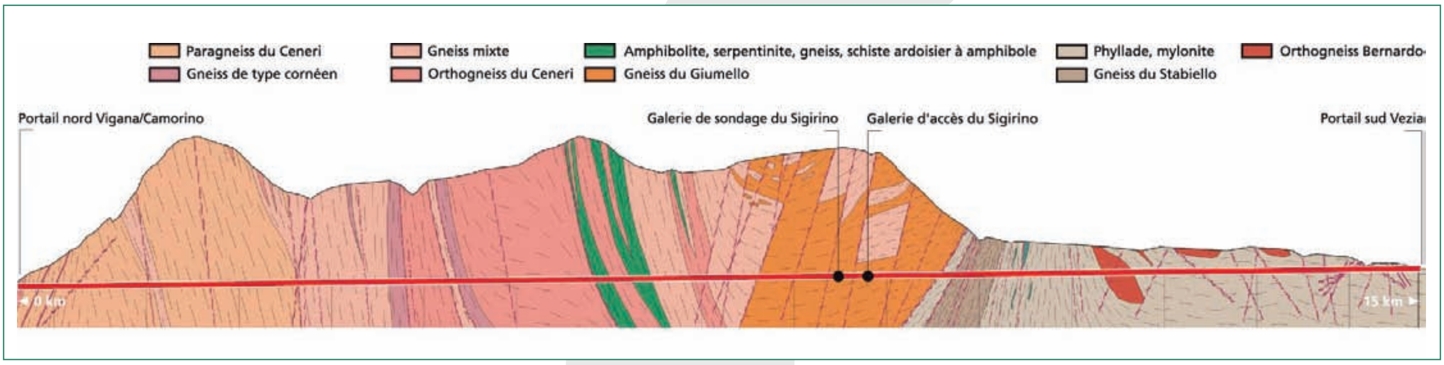
- General geological structure (homogeneous zones, sequence of different types of rock, stratification, foliation, density, fault zones etc.)
- Description of the discontinuities
- Degree of weathering, karst formation, hydrothermal transformations
- In situ stresses and assumed tectonic residual stresses
- Fault zones, such as zones of rock mechanically transformed by tectonic processes (kakirite, cataclasite), as well as karst formations must be specifically recorded. Such zones shall be described like homogeneous geological zones, depending on their extent and frequency. The geometrical data on the position in space of the discontinuities and fault zones shall be reported both as absolute in space as well as relative to the structure (e.g. with respect to the tunnel axis).

2.3.6. Description of the hydrogeological conditions

The local and regional hydrogeological conditions shall be described. In particular, aquifers, their possible interaction, groundwater build up and barriers as well as the regional flow conditions and the relationship to surface waters shall be summarized.

In particular the following items shall be described:

- The possible effects of the structure during its realisation and service life on the existing hydrogeological conditions (quantitative and qualitative effects)
- The possible effects of the groundwater on the facilities during construction (water inflow) and operation due to pressure effects, chemical aggressiveness, sintering etc.
- The type of circulation (pores, discontinuities, karst), the permeability parameters, the level of the groundwater, the flow direction of the water, etc., in each aquifer



Geology

2.3.7. Description of the geotechnical properties
The geotechnical properties of soil and rock shall be described. Measurements of geotechnical properties, such as the strength, deformability, swelling and permeability parameters, abrasivity shall indicate the number of specimens, range of values and spatial validity. Comparative values, empirical values and estimates shall be denoted as such. The sources of information shall be given.
For rock it is necessary to distinguish clearly between the geotechnical data for the intact rock itself and the discontinuities (rock boundaries, fracture surfaces, karst) as well as the filling in of the cavities.

2.3.8. Description of gas occurrences
The occurrence of rocks with a potential gas source or with gas reservoirs together with the corresponding migration paths as well as already known indications of gas in similar geological formations shall be investigated within the design framework. From this it must be clear, whether the possibility of gas deposits and flooding exists and what effects are to be expected due to the escape of gas in order to plan appropriate hazard mitigation measures.

2.3.9. Further Information
The descriptions shall include the following data, as applicable:
- In situ stresses
- Creep movements / areas with landslides
- Neotectonic movements
- Temperature of rock mass
- Seismic risk
- Substances that present a health hazard (quartz, asbestos etc.)
- Radioactivity (including radon)
- Zones of residual waste or contaminated ground
- Ground water contamination

2.3.10. Extent of the description of the ground conditions
Experience has shown that full disclosure of geotechnical information would reduce the risk to both the client and the contractor. Therefore full disclosure of geotechnical information (see 5.3.7) is recommended for Conventional Tunnelling.

2.4. LAYOUT OF UNDERGROUND STRUCTURES

2.4.1. Choice of tunnel system and of alignment
The tunnel system comprises all underground works that are necessary to achieve the planned use and ensure the safety of persons and material assets. Besides the main tunnel tube(s), the tunnel system may comprise, e.g. cross-passages, adits and shafts as escape routes or other ancillary structures such as ventilation shafts or caverns for technical equipment. The choice of the tunnel system is based mainly on operational, organisational and safety considerations. The ground conditions and the topography (layout of the access tunnels and shafts) may also have an influence on the selection of the tunnel system. For example, construction time and cost risks may be different for a twin tunnel than for a double-track tunnel.

The vertical and horizontal alignment of the tube(s) also depends on several factors such as:
- The use of the tunnel (maximum longitudinal gradient, minimum curvature)
- The drainage considerations during construction and operation
- The accessibility and natural hazards in the portal areas
- The ground conditions

If possible, the alignment should be adapted to the ground conditions in an early phase of the project, as hazards and the respective construction time and cost risks can be avoided or re-duced by the choice of a different alignment.
Aspects of execution or operation and safety (such as the necessity of intermediate adits, ventilation shafts or escape adits) may also influence the choice of the alignment. This is particularly true for long tunnels.
The ground conditions shall be taken into account when specifying the spacing between two adjacent tunnel tubes. In special cases (e.g. branching, portal region) other criteria may be decisive.
Similar considerations apply to the selection of the location and axis orientation of caverns.

Twin tube tunnel

2.4.2. Shape of the cross section

The shape and the dimensions of the cross section of underground openings are determined essentially by

- The serviceability requirements associated with the use of the underground works,
- The geological-geotechnical conditions and
- Construction aspects

The required clearance profile is a key factor in the determination of the cross section of the underground opening. The clearance profile is defined according to the scope of the structures, e.g. railways, metro, highways, utility lines, access, escape routes, storage, power plants, protective shelters or military installations. Besides the scope of the structure, further serviceability criteria required by the client can be decisive for the choice of cross section, e.g.:

- Additional space requirements for operating and safety equipment (cable installations, signalling systems, signage, lighting, ventilation, etc.)
- Aerodynamic requirements
- Required water-tightness with respect to water inflow from the ground or water losses from the opening (e.g., the requirement of a complete sealing against pres-surised groundwater necessitates an invert arch or even a circular cross section)
- Maintenance requirements
- Requirements arising from the safety and rescue concept (escape routes within the tunnel, availability of the facilities in emergencies)

The shape and the size of the cross section depend also on the ground conditions, as the latter determine the extent of the required support measures in the construction stage (tunnel support) and in the service stage (permanent lining). Inadmissible reduction in size of the opening due to ground convergence must be avoided by means of additional excavation to account for ground deformations and corresponding support measures.
Weak rock zones, squeezing or swelling rock and soft ground (soils) require a circular cross section or at least a horseshoe-shaped cross section including an invert arch.
Economic considerations and the availability of the necessary equipment may be decisive for the construction method and have, therefore, a considerable influence on the shape of the cross section. In contrast to TBM or shield tunnelling, the cross section of tunnels excavated by conventional methods can be freely chosen within the constraints of the geological conditions.
The main shapes are:

- Horseshoe cross section
- Horseshoe cross section with an invert arch
- Circular cross section

In the determination of the shape and dimensions of the cross section attention must be given to tolerances with respect to driving accuracy, construction tolerances and surveying tolerances.

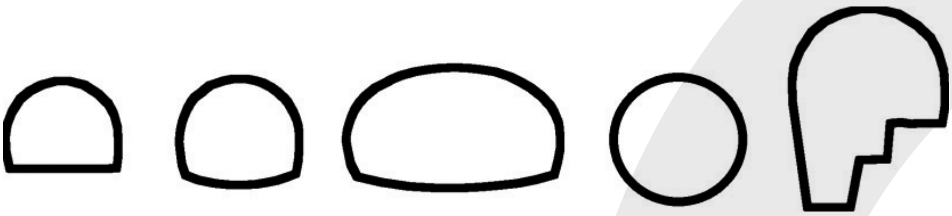


Fig. 1: Typical cross sections in conventional tunneling

2.5. EXCAVATION AND SUPPORT

2.5.1. General

The aim of the structural design is the determination of an economic final structure and a construction method fulfilling the safety, serviceability and environmental protection requirements for the given ground conditions. The design engineer is responsible for an accurate design.
The structural design serves as a basis for the approval procedures, the tender documents (determination of excavation and support classes and their distribution) and the determination of the excavation and support methods used on site.
The design work consists of the preparation of different structural alternatives taking into consideration the relevant boundary conditions, checking the feasibility and assessing the implementation possibilities with regard to fulfilling the design requirements.
The successful construction of underground works depends on the detailed consideration of all factors that are relevant for the structural behaviour. In underground works, the structure comprises the ground surrounding the opening and all temporary or permanent support ele-ments necessary for equilibrium or limitation of deformations. The factors governing struc-tural behaviour can therefore be summarised as follows:

- Ground structure and properties, hydrogeological conditions
- Initial stresses
- Dimension and shape, location and alignment of the opening
- Method of excavation (in the cross section and in the longitudinal direction)
- Support measures (temporary and permanent)

The information required for the structural design of underground openings is manifold and can be grouped according to the following sources:

- Geological explorations and field tests
- Laboratory investigations
- Structural analyses using ground-structure interaction models
- Field tests and measurements
- Engineer's own experience

2.5.2. Outline of the design process

The design is developed stepwise, beginning with the determination of zones with the same ground behaviour during construction (homogeneous zones) and ending with the definition of the excavation and support classes.

In a first step, the project area is subdivided into homogeneous zones having similar conditions with respect to:

- Geological and hydrogeological conditions (based upon the description of the ground, see Section 2.3).
- Topographical conditions (e.g. depth of cover, slopes in the vicinity, etc.).
- Environmental aspects (e.g., structures at the surface, nearby underground structures, groundwater resources to be protected, etc.)

The definition of the tunnel segments has to be based on the current knowledge in each project stage. The number of tunnel segments is project-specific and depends on the design stage, as well as on the complexity of the geological conditions in the project area. In general, a rough subdivision of the alignment will be sufficient in early planning stages, while the increased information in subsequent stages may necessitate a higher resolution.

In the following step, based upon experience and simplified calculations, a rough assessment of the project conditions, potential hazards and necessary measures is carried out and a preliminary decision is made concerning the cross section and the construction method for each section of the alignment.

The usual cross section in sufficiently firm rock and without any water pressure is often a horseshoe-shaped cross section. In this case the rock around the hollow space is part of the load-carrying structure and the final lining has to carry only small loads due to the ground. In less firm or more weathered rock the loads due to the ground increase so that the tunnel's cross section has to be closed at the base.

As a first improvement a straight horizontal invert would be planned, becoming more and more vaulted with increasing weakness of the ground. In soft ground or under high water pressure the shape of the cross section has to become a circular ring.

In the next step excavation and support classes have to be developed for each alignment section. These are determined by the predicted ground and water conditions.

These classes describe the way to proceed with the excavation steps and the support measures, each within a certain and defined range. The classes vary according to the anticipated ground behaviour upon excavation.

The main features of the excavation and support classes are the period between excavating and supporting, the requirement of subdividing the cross section, the necessity of advanced support measures and the requirement of supporting the face. In poor ground conditions, for example, the cross section will be divided in several headings, which must be carefully excavated and supported in short steps. In another alignment section there may be medium firm rock, so that here an excavation and support class must be planned with two partitions of the cross section, with drill and blast excavation in long steps and with reduced support measures. For each alignment section the planned excavation and support classes should be adapted to the most likely ground conditions as well as to potential hazard conditions that can be deduced from the geotechnical conditions.

Subsequently, the potential hazards are described and analysed in detail by taking into account the selected preliminary technical solution. Depending on the results of this assessment, mitigation measures are planned. These may be modifications of the initial method, introduction of additional support elements

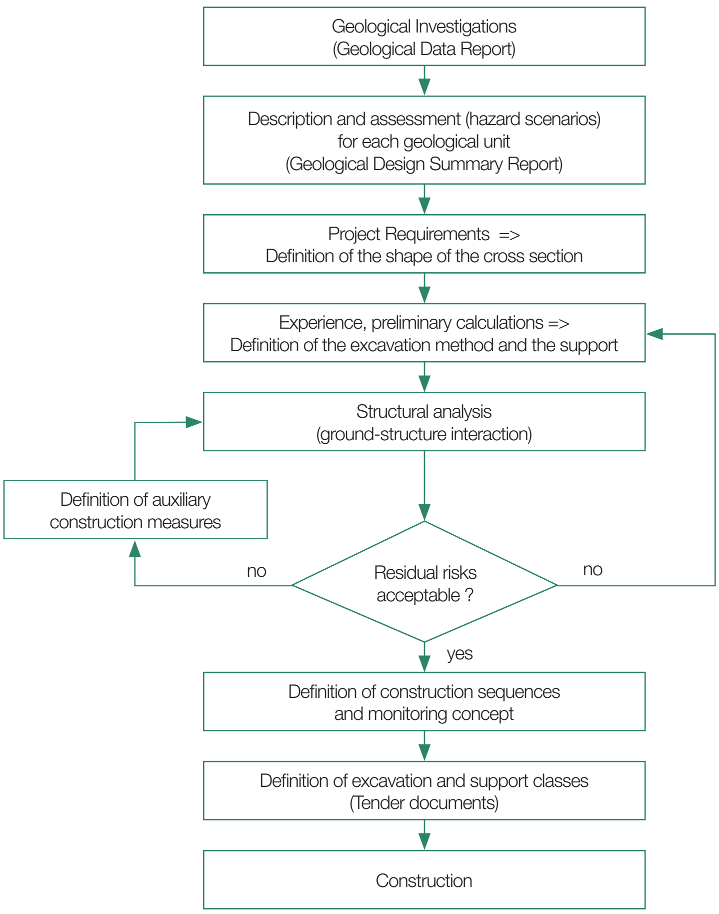


Fig. 2: Design steps for conventional tunnelling

or even the selection of another method. The assessment is carried out for each construction stage as well as for the operation phase.
As a rule all types of construction procedure have their own specific hazard scenarios that have to be evaluated in detail. Therefore, the technical solution is developed iteratively by repeating the procedure sketched above until a solution is determined that fulfils all safety and serviceability criteria for the alignment section under consideration.

It may be useful to work-out technically equivalent design alternatives for each alignment section and select the final solution in the next stage, where a synthesis over the different alignment sections is carried out by also taking into account construction time and cost considerations as well as aspects of the execution of the construction works. For example, frequently changing the excavation and support methods will, in many cases, be technically and economically unfeasible.
Based on the results of the previous steps the alignment is divided into regions with similar excavation and support requirements. Both excavation and support have to be determined to a large extent prior to the construction. All possible geological conditions should be addressed with a defined range of excavation and support methods as well as the probability of occurrence.

In the final step, the design must be transformed into a cost and time estimate for the tendering process. Excavation and support classes are specified, based on the evaluation of the excavation and support measures. The excavation and support classes form the basis for compensation clauses in the tender documents. An excavation and support class may be assigned to more than one tunnel section, as the same measures can be appropriate for different conditions.
To establish the bill of quantities a prediction of the distribution of excavation and support classes is required. This distribution has to be established for the most probable conditions and should also include the likely variations of excavation classes resulting from the ground conditions. When establishing the distribution of excavation and support classes along the alignment the heterogeneity of the ground has to be considered.

2.5.3. Hazards and their mitigation
The recognition and the assessment of potential hazards as well as the planning of appropriate mitigation measures are fundamental to the design of underground structures.
In the present document, the term "hazards" means an event that has the potential to impact on matters relating to a project, which could give rise to consequences associated with:
a) Health and safety
b) The environment
c) The design
d) The design schedule
e) The costs for the design
f) The execution of the project
g) The construction schedule
h) The costs associated with construction
i) Third parties and existing facilities including buildings, bridges, tunnels, roads, surface and subsurface railways, pavements, waterways, flood protection works, surface and subsurface utilities and all other structures/infrastructure that can be affected by the execution of the works.

Hazards shall be identified and evaluated on a project-specific basis and their consequent risks must be identified and quantified by risk assessments through all stages of a project

Possible hazards in underground construction include, but are not limited to:
- Collapse of roof or of the ground above the opening up to the ground surface
- Rock fall
- Rock burst
- Failure of the working face
- Reduction of section (convergence)
- Heave of the tunnel floor due to swelling
- Deterioration of lining due to aggressive groundwater
- Ground surface settlement or heave causing damage (e.g. when tunnelling under traffic routes, buildings, bridges, dams, etc.)
- Inflow of water or mud
- Escape of gas (methane, radon, etc.) or release of dangerous substances into the atmosphere like dust affecting the lungs (quartz, asbestos)
- High temperatures in the rock mass or in the groundwater
- Seismic actions (e.g. at transition between underground construction and cut-and-cover construction)
- Effects on springs and surface waters

The hazards, individually or in combination, constitute possible hazard scenarios. The description of the hazards in the form of hazard scenarios is primarily qualitative and should, if possible, be augmented by quantitative data. Causes and mechanisms shall be reported.
The assessment of hazards should cover the different locations of the underground construction works and the surrounding rock mass as well as the different construction stages and the planned service life.
The assessment shall be carried out in close cooperation with the experts involved in the project (designer, engineering geologist and resident engineer).

In general, hazards can be counteracted by avoidance, prevention or hazard reduction hazard, for example:
- Choice of a different alignment
- Choice of a structure with less susceptibility with respect to the considered hazards
- Choice of a structure that is able to suffer local damage and the loss of an individ-ual structural element or a whole section of the structure without total failure
- Choice of a structure that does not fail without prior warning
- Choice of suitable geotechnical auxiliary measures
- Choice of suitable construction materials
- Appropriate structural analysis and dimensioning
- Careful detailing of the structural elements including waterproofing and drainage
- Execution as planned and carried out with proper care
- Suitable execution checks and warning systems (monitoring with instruments, see section 4)
- Special protective measures for neighbouring structures and plant
- Measures to deal with critical events
- Appropriate monitoring and maintenance

Both the assessment of the hazards and the design of the mitigation measures are based on an in-depth understanding of the underlying mechanisms, experience from previous similar projects, structural analysis and careful dimensioning (Section 2.5.4). The planned construction method must be evaluated with respect to safety and serviceability (i.e., that displacements are within acceptable limits) requirements for all construction stages. Furthermore, the compliance with environmental requirements (surface settlements, vibrations, ground water disturbance, etc.) should be checked. The variability of the influencing factors has to be considered in the assessment.

As far as possible, the structural concept, the dimensioning and the construction methods shall be checked and assessed according to experience gained on comparable projects. Substantial divergences from normal construction practice shall be analysed and substantiated. In the assessment of the implementation possibilities special attention shall be paid to

- The simplicity of execution,
- The insensitivity to unavoidable execution inaccuracies or possible errors in execution,
- The ability to adapt to possible changes in ground behaviour, and
- The execution of the works

For the evaluation of the individual hazards (with and without remedial measures), a risk analysis should be carried out by means of a qualitative assessment of the probability of occurrence and by a quantification of the impact. Due to the fact that the different input data is normally quite inaccurate a simple qualitative method is recommended rather than complicated mathematical models. A simple and efficient method of risk evaluation is to determine the risk R as the product of the probability of occurrence P multiplied by the impact or loss of time I (Risk R = P x I). The probability of occurrence and the amount are estimated values.

The evaluation criterion and intervention strategy are project-dependent. The value of risk R after implementing planned remedial measures is the remaining, unavoidable risk.

2.5.4. Structural analysis and dimensioning

Design decisions should be made on the basis of a cautious qualitative and quantitative analysis of all relevant factors. Besides engineering judgement based upon the engineer's own fund of experience, modern methods of structural analysis (i.e. numerical methods) may be applied.

The quantitative verification of structural safety or serviceability may be dispensed with if the respective design requirements can be adequately ensured using well-proven design and/or execution measures. In fact, some actions can often be mitigated better using design measures or by eliminating the action than by dimensioning according to limit states. In order to judge the effectiveness of constructional and execution measures, reliable, comparable and transferable experience must be available.

The proper use of structural analysis requires a thorough understanding of and a "feel" for the complex processes involved in the construction of underground openings as well as a good background in geotechnical and structural engineering. Therefore structural analysis cannot make up for inadequate experience or intuitive insight into the problems. Thus the information provided by structural analysis supplements the basic knowledge that is expected of a tunnelling engineer. One is less likely to go wrong, therefore, when one starts from the wealth of knowledge already accumulated in tunnelling practice and then fits the information obtained by structural analyses into the framework of this basic knowledge.
The goal of the analysis is to investigate quantitatively the behaviour of the structure (described basically in terms of deformations and stresses) in the considered dimensioning situations taking into account the critical influence factors. The starting point of the structural analysis is the conceptual design.

A clear formulation of the particular problem facing the en-gineer is required before carrying out a structural analysis. On the basis of this formulation of the problem a suitable structural model can be established and the variations in the required input data can be defined by specifying of the upper and lower bounds.
The structural model idealises the complex reality with respect to the static system, the material behaviour and the loads. The structural model comprises the entire structure, i.e. the ground surrounding the opening and the temporary or final support elements. It connects actions, geometrical quantities and the properties of the construction materials and the ground for the purpose of structural analysis. The ground model is part of the structural model and comprises, in an idealised way, the geological structure and properties of the ground. The structural model must be suitable for predicting the structural behaviour in the dimensioning situations under consideration. It should, on the one hand, approximate as closely as possible the real situation and, on the other hand, be as simple as possible. The methods of structural analysis should be based on standard engineering practice or empirically proven theory.

Depending on the particular questions to be answered by the analysis, different structural models may be decisive. It is possible to assume different models for the same problem (behaviour hypotheses) and carry out a variation of parameters of each model. In this manner it is possible to single out the important factors and to compare the results corresponding to pessimistic and optimistic estimates.
Attention shall be given to parameters exerting a large influence. The results of the structural analysis shall be checked for plausibility, keeping in mind that they do not refer to the actual conditions but to the model considered. The validity of the computational results is conditioned by how well the model corresponds to reality. Since structural modelling also involves subjective assumptions, the final results are not beyond doubt. Structural analyses provide useful indications, but not proofs of the structural behaviour. Computations, therefore, cannot dictate important decisions but only provide a reason for these decisions.

The dimensioning situations, the assumptions made in the structural analysis, the analytical models and the verifications of structural safety and serviceability shall be clearly documented in the technical report. Computational results should be presented in the form of diagrams, which give a good summary of the results and allow various computed cases to be easily compared. For conclusions affecting constructional decisions the computed deformations in the ground and stresses in the lining are of especial value.

2.5.5. Modifications to construction method on site

Depending on the geological complexity, the extent of the geological pre-investigation and available experience from other projects in similar geological conditions, the information concerning the ground may be subject to uncertainties.
If the structural behaviour cannot be predicted with sufficient reliability based on site investigations, structural analysis and comparable experience, the design may permit or foresee construction method modifications during construction, provided that the relevant hazards can be detected and localised in time by observations and they do not lead to sudden or uncontrollable failure (s. Section 4 "Monitoring").
Otherwise, constructional measures and suitable types of supports must reduce the potential hazards in order to comply with the safety requirements.
For the purpose of construction method modifications on site, the information gained during execution both on the ground properties and on the structural behaviour shall be introduced into the current process of design and execution.

In particular, the design should specify:
- Relevant mechanisms endangering safety or impairing serviceability during construction
- The information to be collected on site during construction, for example geological records of the tunnel face, results of advance probing, qualitative observations (such as signs of excessive stress in the lining or failures of the face) or monitoring results (see Section 4 "Monitoring")
- Criteria for the selection of excavation, support or auxiliary measures
- That the criteria may be based upon qualitative observations or control values (in general the deformations of the opening and its surroundings) determined by structural analyses or experience
- The actions to be taken for every foreseeable significant deviation of the observational findings from the expected ones
- A management concept with all technical and organisational provisions to allow a timely decision-making process during construction

During construction, all relevant data, concepts, considerations and decisions shall be recorded in such a way that a review of the decision making process is possible.

2.6. FINAL LINING

An underground space ex cavated by the Conventional Tunnelling Method may need a final (secondary) lining in addition to the primary lining according to the requirements of the project to
- Cater for all the final load cases
- Fulfil the final safety margin
- Include the necessary protection measures (e.g. water tightness)
- Guarantee the required service life time

Generally two options exist to construct the final (secondary) lining:
- Installation of an independent secondary lining that is normally dimensioned to withstand all the final load cases.
  The secondary lining can consist of shotcrete or cast in situ concrete.
  According to the requirements of the project the final lining consists of unreinforced concrete or reinforced concrete (steel bars or fibres).
- Installation of additional layers of shotcrete to strengthen the primary lining for all the final load cases.

2.7. TENDER DESIGN DOCUMENTS

Based on the individual reports submitted by the various expert teams involved in the project, a technical report should be prepared documenting the decision-making process and summarising the results of the design.
Finally the tender design documents should contain:
- The contract documents
- A summary of the results of geological and geotechnical investigations, and the interpretation of the results
- A description of the ground and the associated key parameters
- A description of the possible hazards, the relevant influencing factors, the analyses performed, and the underlying geotechnical model
- The specification of excavation and support, relevant scenarios considered, analyses applied, and results
- A baseline construction plan
- Detailed specifications concerning the baseline construction plan (including measures to be determined on site if any)
- The determination of excavation and support classes, their distribution along the alignment
- The bill of quantities
- The technical specifications
- The drawings

The baseline construction plan in the tender design documents describes the expected ground conditions (geological model with distribution of ground types in the longitudinal section), the excavation and support types (round length, excavation sequence, overexcavation, invert distance, support quality and quantity, ground improvements, etc.) as well as zones, where specific construction requirements have to be observed. The baseline construction plan also has to contain clear statements describing which measures cannot be modified during construction, as well as the criteria and the actions for possible modifications and adjustments during construction.

3 >> CONSTRUCTION METHODS

3.1. EXCAVATION METHODS

The excavation methods for Conventional Tunnelling are:
- Drilling and blasting mainly applied in hard rock ground conditions
- Mechanically supported excavation mainly used in soft ground and in weak rock conditions (using roadheaders, excavators with shovels, rippers, hydraulic breakers etc.)

Both excavation methods can be used in the same project in cases with a broad variation of ground conditions. In both excavation methods the excavation is carried out step by step in rounds. The round length generally varies from 4 m in good conditions to 1 m or less in soil and poor ground conditions (e.g. squeezing rock). The round length is the most important factor for the determination of the advance speed.

The design engineer shall prescribe or limit the choice of the method of excavation only if there are compelling reasons based on project restrictions. The responsibility of the selection of the excavation method should be left to the contractor, based on the owner's description of the ground conditions and the limits set by the design engineer.

Drilling and blasting, full face

Mechanical excavation, top heading bench

Excavation with shovel

Face support with shotcrete

Road header excavation

Conventional tunnelling in squeezing ground (full face)

3.2. EXCAVATION SEQUENCE

Conventional Tunnelling allows full-face and the partial excavation of the tunnel cross sec-tion. Besides the structural analysis an important criterion for selecting the adequate excava-tion sequence is the length of the individual excavation-steps/rounds, which depends on the stand-up time of the ground without support. In good ground conditions the maximum round length is limited by the acceptable tolerance for overbreak, which is mainly an economic criterion when overbreak has to be filled up to the design line of the tunnel circumference.

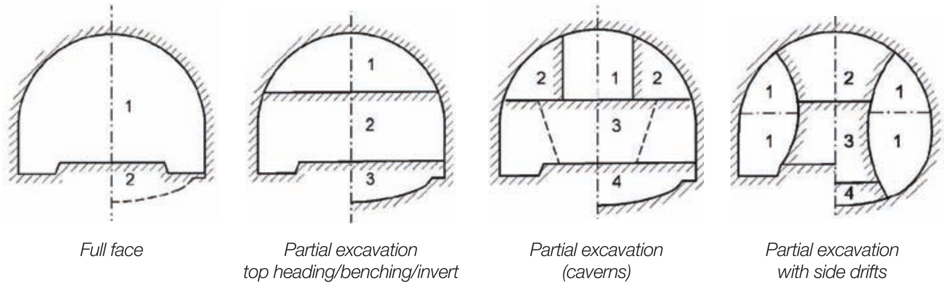


Fig. 3: Typical excavation sequences in conventional tunneling

Re profiling

Re profiling

Both excavation types (full-face and the partial excavation) allow exploratory drillings from the face at any time.

Full-face excavation is used for smaller cross sections and in good ground conditions with long stand-up times. Since a high degree of mechanisation of the work and the use of large, high performance equipment has become common, also bigger cross sections (70 to 100 m^2 and more), even in difficult rock conditions (e.g. squeezing rock) are excavated with the full-face method. In any case, face stability shall be given serious consideration and often face support - bolting, shotcrete etc. - becomes necessary.

Full-face excavation allows the immediate closure of the primary support ring, close to the excavation face.

Partial excavation is mainly used for big cross sections in soils and unfavourable ground conditions. There are several types of partial excavation such as top heading, bench- and invert-excavation, side drifts, pilot tunnel, etc. Partial excavation, allows the combination of the different excavation methods in the same cross section, e.g. blasting in the top heading and excavating the bench by using a mechanical excavator e.g. a roadheader.`

The choice whether full-face or partial excavation is preferable depends on ground properties but also on environmental aspects, on the magnitude of settlements at the surface and economic considerations. In special cases both excavation sequences can be used. However, frequent changes in the type of excavation are uneconomical.
Today full-face excavation is also becoming possible in difficult ground conditions.

Partial excavation

Top heading

3.3. PRIMARY SUPPORT

The purpose of the primary support is to stabilize the underground opening until the final lining is installed.
Thus the support placement is primarily a question of occupational health and safety but it is also a question of the usability of the tunnel itself as well as of the protection of the envi-ronment (neighbouring buildings, lines of communication in or above ground facilities, etc.).
In many cases it may become necessary to apply the support system in combination with auxiliary constructional measures.

The most common elements for the primary support are
- Rock bolts
- Shotcrete (not reinforced and reinforced with fibres or wire mesh)
- Steel ribs and lattice girders
- Meshes
- Lagging

These elements are applied individually or in combination in different types of support depending on the assessment of ground conditions by the responsible site engineers and by taking into account the corresponding design. In each round, elements of the primary support have to be placed up to the excavation face for reasons of safety and health and according to the structural analysis and the assessment of the actual ground conditions. The selection of the support elements has to consider the onset of effect and the support pressure of each element.
Additional elements for the primary support can be placed in the rearward area according to the requirements of the structural analysis, the ground conditions and the construction sequence.
The baseline construction plan indicates the support types available for each homogenous zone in the geotechnical model and contains limits and criteria for possible variations or modifications on site. The baseline construction plan also contains warning criteria and remedial measures for the case when acceptable limits of behaviour are exceeded.

Rockbolts

3.4. AUXILIARY CONSTRUCTION MEASURES

In special cases, the excavation work can only be carried out by means of additional auxiliary construction measures. The auxiliary construction measures can be classified in the following categories:

- Ground improvement
- Ground reinforcement
- Dewatering

3.4.1. Ground improvement

Ground improvement means the application of methods that improve the mechanical or hydraulic properties of the ground.
The main methods are

- Grouting
- Jet grouting
- Ground freezing

Ground improvement has normally to be carried out alternately to the excavation and leads to interruptions of the excavation work. In special cases ground improvement can be carried out from the surface or pilot tunnels outside the future tunnel cross section.

Grouting

The different techniques for grouting are consolidation grouting, fissure grouting, pressure grouting and compensation grouting.
Grouting can be carried out in the tunnel excavation as face grouting or as radial grouting from the excavated tunnel or from a pilot tunnel. The most commonly used grout material is cement. In special cases chemical products such as resins or foams are also applied. In these cases the environmental and safety restrictions have to be considered specially.

Face grouting

Jet Grouting

Jet grouting is applied mainly horizontally or at a slightly upward or downward angle from within the face of the tunnel. An improvement of the roof arching behaviour is achieved by applying one or more layers of jet grouting columns in stages corresponding to the excavation operations.

An improvement of the stability of the face is achieved by placing individual jet columns parallel to the direction of advance in the working face.

Less common in tunnelling is vertical or steeply inclined jet grouting, except in shallow tunnels where it is applied from the surface. From within the tunnel vertical or steeply inclined jet grouting is mainly applied to underpin the bottom of the roof arch.

Jet grouting

Jet grouting

Ground Freezing

The following ground freezing techniques are known to waterproof or stabilize temporarily the ground:

- Continuous frozen bodies which provide long-term load-bearing
- Short-term, immediately effective local freezing of damp zones close to the face or in the immediate vicinity outside the excavated cross section

Short-term, immediately effective freezing is achieved by means of injection lances with liquid nitrogen cooling.

A long-term frozen body is produced along the top and side boundaries of the excavated cross section, and in some cases in the invert region. The freezing is achieved by a drilled tube system, through which coolant is pumped. The frozen bodies can be installed alternately to the excavation work from the extended tunnel face in an overlapping way or in advance from separate adits and from the ground surface in cases of small overburden.

Ground freezing

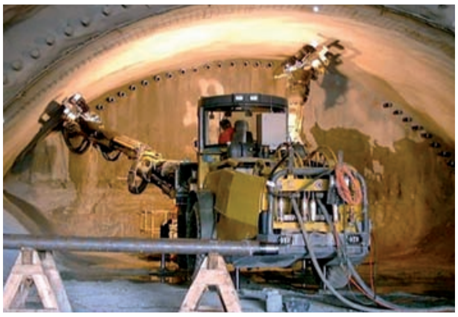
3.4.2. Ground reinforcement

Ground reinforcement involves the application of methods that use the insertion of structural elements with one predominant dimension. Bolts, anchors, micro piles and spiles are such elements. The main methods of application are pipe umbrellas; face bolting or radial bolting from a pilot bore.

Pipe umbrella

Pipe umbrellas are specified to supplement the arch structure in the roof and springline regions as well as stabilization of the face and in advance of the face immediately after the excavation.

Portal pipe shields are drilled at the portal wall along the cross section parallel the direction of advance and serve to bridge zones of disturbance behind the walls. Fan-like, overlapping pipe shields are installed in stages alternately with the excavation for the tunnel driving. The pipe umbrella shall extend at least 30% beyond the face of the next excavation.



Pipe umbrella

Spiles

Spiles are steel rods left in the ground for the local short-term stabilisation of the roof section and at the working face on the boundary of the excavation.

The spiles rest on the first steel arch in front and should be at least 1.5 times as long as the subsequent advance in the excavation. Depending on the type of soil, the spiles can be jacked, rammed or inserted in drillholes. To improve the ground conditions spiles can be used with a central borehole and lateral es-cape openings (cf. bored bolts). After grouting, this creates an optimum bond with the sur-rounding material. Spiles are placed during the excavation cycle in predefined steps.

Face bolts

Face bolts are often necessary to stabilize or reinforce the face. Depending on the relevant hazard scenario, the relevant bolt type and length have to be determined in the design. Practically any bolt type or length is possible. As a protection against rock fall, spot bolts may be sufficient whereas in difficult ground conditions (e.g. squeezing rock and soils) systematic anchoring with a high number of long, overlapping steel or fibreglass bolts may be necessary. Face bolts are placed during the exca-vation sequence, if necessary in each round or in predefined steps

Face anchors

3.4.3. Dewatering and drainage

In some cases the tunnel construction is only possible with the application of special dewatering measures. According to the ground conditions and other boundary conditions conventional vertical or horizontal wells or vacuum drains can be used.
In the design of the dewatering measures environmental aspects have to be considered, such as limits on lowering the ground water table, settlements, etc.

In the case of low overburden, dewatering measures can be carried out from the ground surface. In the other cases, dewatering has to be done from the tunnel cross section or from pilot tunnels.

4 >> Monitoring

4.1. OBJECTIVES OF MONITORING

Field monitoring is an indispensable element of modern tunnelling. The purpose of the instrumentation may be:

- Checking the structural behaviour with respect to safety and/or serviceability criteria, mainly during construction and in some cases during service life.
- The quantification of structural response to a specific method of construction and checking the effectiveness of specific support measures.
- The comparison of theoretical predictions with the actual structural behaviour and the assessment of the material parameters of the ground.
- Checking adjacent structures and facilities for their safety and serviceability as a result of the construction of the tunnel.

Due to its quality, monitoring data can also be used for clearing disputes between contractual partners or between the client and third parties. Therefore a further objective is:

- Documentation of evidence related to the tunnel construction and the effects on adjacent facilities.

Instrumentation can also help to advance the state-of-the-art in technology in a particular geotechnical context (e.g. urban subway construction). The monitoring results often provide a very valuable insight into the ground deformation pattern and failure mechanisms, contribut-ing thus to the project optimisation in terms of safety, construction time or cost.

The results of field measurements can be used to assess structural behaviour with respect to safety and/or serviceability requirements. In such cases, the determination of acceptable behaviour should include threshold values of key indicator parameters. The monitoring results should be evaluated in combination with other observations in order to decide whether corrective measures are necessary or not.

Deviations from past deformational behaviour, such as an unexpected acceleration over several readings without ongoing construction activities in the vicinity of the monitoring section, must be analysed immediately. Such procedures can obviously be applied only for the case of ductile structural behaviour.

Decision-making based on measurements is impossible when the structural behaviour is brittle (e.g. rock bursts or tunnel face instabilities), as the predictions of deformation values close to collapse are highly unreliable.

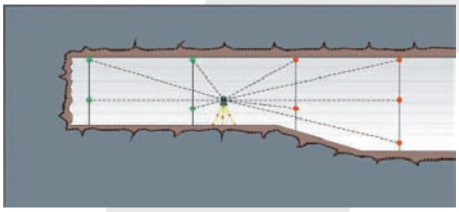
The planning of a monitoring program should include the following steps:

- Prediction of the mechanisms that control behaviour
- Selection of parameters to be monitored
- Prediction of the magnitude of change
- Selection of instrumentation and its accuracy
- Instrument location plan
- Redundancy of instrumentation
- Data collection plan
- Data processing,
- Interpretation and report plan

4.2. PHYSICAL QUANTITIES AND INSTRUMENT SELECTION

The most important physical quantities to be monitored can be subdivided in following groups:

- Deformations (displacements, strains, changes in inclination or curvature)
- Stresses (contact stresses, boundary stress on a beam, state of stress) and forces on structural elements (bolt force, normal load on a compression element or steel arch)
- Piezometric levels
- Temperatures



Monitoring

The most common monitoring method is the measurement of displacements, for example convergence of the underground opening or ground surface settlements.

Displacements have the advantage that in a mathematical sense they represent integrated quantities and are basically not subjected to local effects. Stresses, strains or changes in curvature, on the other hand, are differential quantities, whose validity is limited to local regions (scale effect). Therefore the observation at several successive points will be necessary to obtain a distribution over a sufficiently large area.

In some cases, for example the construction of underground openings in swelling rock or in the presence of difficult groundwater conditions, measurements of contact stresses or groundwater pressure can be a very relevant and sensitive measurement. With the selection of the parameters to be monitored the instrument types are also selected. The instrument resolution and required range are given by the predicted maximum of the magnitude change.

However, the accuracy depends not only on the resolution, but also on the measuring principle used for the instrument. Additionally, instruments with a large range often have a lower resolution and accuracy. Finally, in selecting the instrument, availability, durability, maintenance and calibration requirements as well as costs also have to be considered.

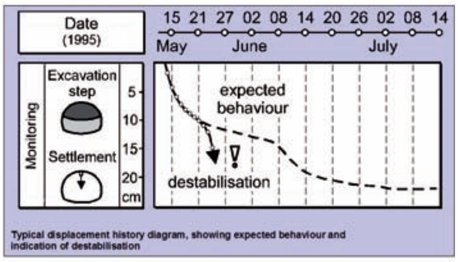
Monitoring

4.3. MONITORING LAYOUT AND PLAN

The monitoring layout is determined by the predicted mechanism or behaviour of the structure. For example, it must be established if pointwise measurement is adequate or if measurements along profiles or over surfaces are necessary. For the definite location of the instruments the zones of primary concern and critical areas, in which additional instrumentation may be required to get meaningful results, should be identified. The layout and spacing between instrumentation arrays will depend on factors such as the stratigraphy, level of detail and degree of redundancy required as well as the location of the tunnel in relation to nearby existing structures.

The plan of data collection includes details about frequency of readings, data transmission and data storage. Readings may be taken at intervals, continuously (real time), depending on specific construction stages or time events.

In addition, the plan shall include data reduction, analysis, interpretation and assessment of the response of the structure.



Monitoring

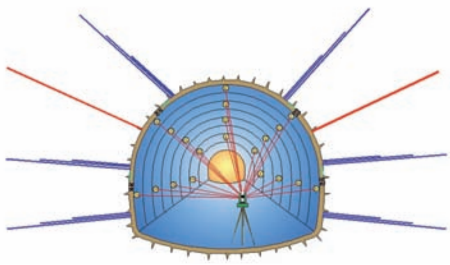
4.4. ORGANISATIONAL ISSUES

In contract documentation, responsibilities for installation and commissioning, calibration, provision of baseline data, monitoring, information flow, data interpretation and reporting must be clearly defined.

For the owner it may be advantageous to appoint an independent monitoring contractor who carries out the monitoring work and, on a real-time basis, delivers the results to all par-ties involved in the execution of the tunnel project (client, designer, construction manager if present, contractor, etc). The evaluation of the monitoring results should also be carried out on a real-time basis by a suitably experienced third party (either the client with his consultant or the design engineer responsible for the detailed design of the tunnel). The contract conditions should provide the necessary empowerment to persons to implement immediate stabilizing works according to the results of the monitoring.

4.5. OTHER OBSERVATIONS AND MEASUREMENTS

- Geotechnical observation of the rock mass (type of rock mass, weathering, geotechnical classification, the strength of the rock mass, zones of weakness, foliation, etc.)
- Hydrogeological observation (changes of the water table level, changes of the water level in wells, verification of the connection between ground water in the zone near the surface and at depth, quality of the water, chemical composition)
- Quantity of the water (inflow to the tunnel)
- Geophysical measurements (indicates failure in homogeneous rock mass)
- Seismic and acoustic measurements (in case of drill and blast excavation)



Monitoring

5 >> Construction contract

5.1. INTRODUCTION

Underground construction is clearly different from any other type of construction because of its inherent nature: uncertainties in the ground conditions, unforeseen conditions, dependency on the means and methods, and the high construction risk associated with this type of construction. It is very important to deal with contracting practices in tunnels and underground construction differently from other types of construction.

The contract conditions for Conventional Tunnelling shall allow for approval and certification for these changed and/or varied works within a short time. With reference to the possibility of encountering changed conditions the contract should be based on a measurement version. This will help to respond to changed conditions even if relevant provisions for all possible consequences could not be already included at the time of contract award.

There is a need for a creative, progressive and fair contracting form in underground con-struction and especially in Conventional Tunnelling. The advantages of the high flexibility of Conventional Tunnelling can only be realised with corresponding contracts with a fair risk sharing between the client and the contractor. The processes for improving underground contracting practices (see 5.3) is highly recommended for contracts dealing with Conventional Tunnelling.

The main issue that overrides all aspects of underground construction is risk. Risk can be related to construction means and methods, to ground type and behaviour, to unforeseen conditions, or to external factors such as third party approvals or imposed limitations.

The level of risk sharing is a major factor in deciding the type of procurement practice to be implemented. Other issues that influence the type of procurement include the size and the complexity of the project, the definition of its scope, and the identification of imposed constraints. Clients need to weigh various factors in reaching a decision regarding the procurement system. They have to weigh quality vs. costs vs. schedule. These factors are often mutually exclusive.

5.2. RISK MANAGEMENT

Many claims in tunnel construction are often related to unforeseen conditions. Therefore, it is recommended to provide a viable trigger by means of the Differing Site Condition (DSC) clause, culminating in the use of the Geotechnical Baseline Report (GBR) and Geotechnical Data Report (GDR) (see 5.3.7).

It is important from a risk-sharing perspective that the contractual language for the DSC, GBR, and GDR are harmonized.

5.2.1. Risk allocation/sharing

The allocation of risk between the client and the contractor will have a direct relationship to the contractor contingency as part of the contractor's bid. Therefore, it is important to identify a risk-sharing mechanism that is fair and equitable and that will result in a reasonable contingency by the contractor and a sufficient reserve fund to be provided by the client to address unforeseen conditions.

It is customary, for example, because the ground belongs to the client, that unforeseen conditions due to ground conditions are paid for by the client if certain tests are met, while means and methods are generally the contractor's responsibility and their inability to perform under prescribed conditions are risks to be absorbed by the contractor.

5.2.2. Contractor's contingency/client's reserve

With a proper contracting form and an equitable allocation of risks between the client and the contractor, the contractor contingency, which is part of the bid price, will be reduced. Similarly, the client's reserve will be used only if certain conditions are encountered, resulting overall in smaller costs for the client.

5.3. PROCESSES FOR IMPROVING UNDERGROUND CONTRACTING PRACTICES

The following processes aim at improving underground contracting practices. They include prequalification of contractors, geotechnical disclosure, Dispute Review Board (DRB) and amicable settlement process, the use of differing site condition clauses, escrow bid documents, unit prices and contingent bid items, value engineering, client-controlled insurance program, and partnering.

5.3.1. Pre-qualification of contractors

This involves technical and financial qualifications of the potential bidders to ensure their ability to perform the work effectively, economically, and to a high quality. The pre-qualification would include the company's or the joint venture's technical ability to perform the work. Items to be evaluated include: approach, experience with similar projects, with similar ground and similar proposed methodology.
Pre-qualification of the key staff, such as the contractor project manager, the field manager or superintendent, etc., is critical for a successful project.
In addition to the bidder's financial ability to obtain bonding, and his solvency, his history of completing projects on time and within the budget is a factor in the bidder's financial qualifications.
Pre-qualification should extend to major subcontractors and major suppliers.

5.3.2. Dispute Review Board (DRB)

The dispute review board process has been used in the tunnelling industry in certain areas for many years. In this process a board of independent, experienced, and impartial members is selected to hear and address disputes. Generally, the board consists of three members, one representing the client, one representing the contractor, and the third who acts as the chairperson of the board, selected by the other two members. The board provides recommendations to resolve disputes that participants are unable to solve. It is found that this process results in lower bids, better communication and less acrimony at the job site, fewer claims, and more timely and cost-effective resolutions.

5.3.3. Differing Site Condition clause

The Differing Site Condition (DSC) clause was established as a measure of allocation of risk between the client and the contractor relative to the ground condition. In exchange for lower initial bids, the client bears some portion or all of the risk regarding subsurface conditions.

In the case of low bids, bid contingencies are paid by the client, regardless whether adverse conditions are encountered or not. On the other hand, DSCs are paid only if they are encountered.

There are two categories of DSCs:

- Category 1 governs when subsurface conditions differ from those indicated in the contract. This is based solely on what is stated in the contract, including geotechnical data or geotechnical interpretation included in the contract documents.
- Category 2 applies when conditions, which were not known to the contractor at the time of contracting, differ from those normally encountered in the area. It is generally related to unusual conditions and not based on contract documents.

It is important to note that to recover on the DSC clause the contractor must demonstrate the impact on costs and time and must show causality.

5.3.4. Escrow bid documents

In this process the selected contractor's bid documents are placed in an escrow, that is, held in trust by a third party and turned over to the grantee only upon fulfilment of a condition. These documents would then be utilised if needed to assess a fair entitlement and adjustment for additional work, differing site condition issues and claims. However, in this process clients fear that the contractor could condition the bids while contractors fear that they could lose confidentiality of means and methods. This process has not been used extensively.

5.3.5. Partnering

In the last few years partnering clauses have been included in tunnelling contracts. The goal of this process is to minimise disputes and to prevent them from escalating in time and value by resolving them at the lowest possible level in the project organization.
It attempts to establish a win-win attitude between the project participants, including the client, the contractor, the engineer, and the construction manager. This process encourages dialogue among the various participants and relies on reasonable people to resolve disagreements reasonably. It seeks to eliminate adversarial posturing and positioning that often develop when disputes and claims arise. Through this process a series of dialogues and interactions are developed whereby the team members are encouraged to work out differences in the best interests of the project. When an issue is not resolved at the lowest level, it is brought up to a higher level for resolution.

5.3.6. Value Engineering

To stimulate innovative approaches within the limitations of the contractual requirements, cli-ents opt to include a value engineering clause in the contract. Relaxing the design criteria where not critical or meeting the intent of the design more efficiently via creative approaches achieves efficiency. The savings achieved by value engineering are shared between the client and the contractor. It is important to assess the potential effects of differing site conditions on the design as modified by the value engineering.

5.3.7. Full geotechnical disclosure

Experience has shown that full disclosure of geotechnical information would reduce the risk to both the client and the contractor and thus the project costs. Therefore it is important for clients to invest in a comprehensive geotechnical program.

The information should be included in the contract documents. The intent of the disclosure of geotechnical information is to share and allocate construction risk between the client and the contractor.

The Geotechnical Data Report (GDR) contains all the raw data, including boring logs, records of measurements, and field and laboratory tests and their results. It is recommended that the Geotechnical Data Report be made a contract document in order to provide to the potential bidders with the same information that the design engineer used in the design.

In the Geotechnical Design Summary Report (GDSR) the design engineer's interpretation of the data, anticipated ground behaviour, and the identification of the conditions which affect the design and which may impact on construction should be shown.

The Geotechnical Baseline Report (GBR) establishes quantitative values for selected conditions anticipated to have great impact on construction. These values are established through technical interpretation of the data and financial considerations of risk allocation and sharing. The advantages of this report are ease of administration of contractual clauses, unambiguous determination of entitlement, clear basis of contractor's bid, and clear allocation of risk between client and contractor. If baseline values are defined optimally, this can result in minimising contingency of the bidders while limiting the client's risk to a reasonable level.

6 >> Organisation of project execution

The organisation of the project execution is highly dependent on the selected contract model. Nevertheless the organisation of a project is decisive for its success.
For design bid build-contracts the following model is often used in Conventional Tunnelling:

6.1. Client
The overall project management is primarily the responsibility of the client and comprises the general supervision of the construction work, basic decision-making, determining which measures to adopt in the case of technical supervision, financial or schedule variations and deciding upon remedial measures to correct defects.

The ground belongs to the client. Therefore unforeseen ground conditions are the client's risk.

In certain circumstances the client may augment his staff by hiring a project manager and/or a construction manager. The project management team will act as an extension of the client's staff and will oversee the project performance technically, financially and in accordance with the project schedule. The construction manager oversees the implementation of the project during construction.

6.2. Design engineer
The client commissions a design engineer to perform the tasks of planning and design of the project during the different project stages. After obtaining the corresponding construction and design approval the tendering is carried out. The design engineer prepares the tender documents, evaluates the tenders and formulates the contract award document on behalf of the client.

During construction the design engineer prepares the detailed design. If the design engi-neer is not the same consultant as for the site supervision a close relation to the site shall be established and the roles and responsibilities clearly defined. It is recommended that for conventional tunnelling, the design engineer is also the site supervision consultant for the tunnel excavation.

The design engineer and site supervision accompany the structure into service after com-pletion of the work. They prepare the asbuilt documents and maintenance plans.

6.3. Site supervision
The client commissions a consulting engineer with the overall supervision of the construction work (site construction manager). In many cases the site engineer is the same consultant as the design engineer in order to ensure an unshared engineering responsibility towards the client. The site supervision safeguards the interests of the client and carries out the agreed work observing the recognized rules in that field while optimising costs and schedules.

The site construction manager is responsible for the general management and supervision on the construction site with regard to quality and costs. This involves checking the proper use and handling of construction materials, helping to implement the integral occupational health and safety concept, establishing measurement schedules and checking the contractor's bills. Finally the site engineer supervises the rectification of defects.

6.4. Specialists and experts
In addition, as needed, the client uses specialists and consultants, who are usually commissioned directly by the client. These are, e.g., geologist, geotechnical engineer, hydrogeologist, environmental specialist, architect, building physicist, surveyor, safety officer, gas engineer, independent checking engineer, etc. The work of the specialists includes, among other things, giving expert advice to the design engineer and the client.

6.5. Contractor
The selected contractor carries out the work according to the contract documents. Means and methods are generally the contractor's responsibility and the inability to perform under the prescribed conditions is the risk to be absorbed by the contractor.

6.6. Dispute review board
The commitment of a dispute review board (see 5.3.2) is highly recommended.

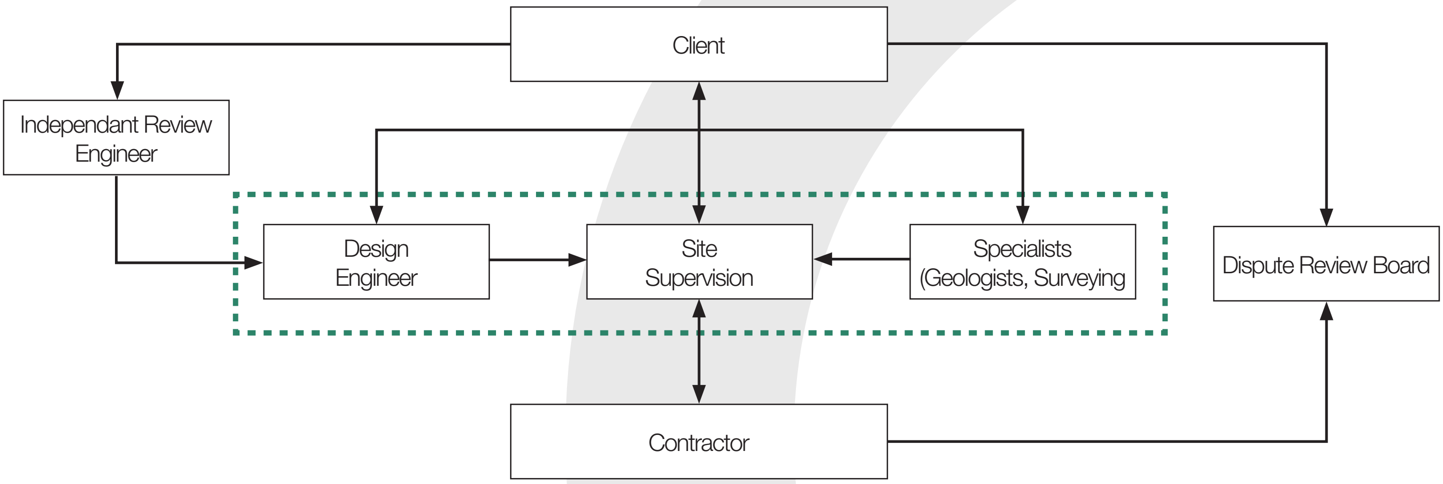


Fig. 4: Example of a Project Organisation

ITA WG 2, Research; ITA Guidelines for tunnelling risk management, Tunnelling and Underground Space Technology, Vol.19 (2004), No. 3, pp 217 – 237

ITA WG 3, Contractual Practices; ITA Recommendations on Contractual Sharing of Risks, Tunnelling and Un-derground Space Technology, Vol.7 (1992), No. 1, pp 5 – 7

ITA WG 3, Contractual Practices; ITA Position Paper on Types of Contract, Tunnelling and Underground Space Technology, Vol. 11 (1996), No. 4, pp 411 – 429

ITA WG 7, General approaches to design; Guideline for the Design of Tunnels, Tunnelling and Underground Space Technology, Vol. 3 (1988), No. 3, pp 237 – 249

ITA WG 12, Shotcrete use; Guideline for the Design of Tunnels, Tunnelling and Underground Space Technology, Vol. 3 (1988), No. 3, pp 237 – 249

ANNEX A1: SELECTED NATIONAL CODES AND GUIDELINES

Country	Code			Languages
Austria	Ö-Norm B2203:	"Werkvertragsnorm für Untertagbauarbei-ten mit zyklischem Vortrieb"	2001	German
	ÖGG-Richtlinie:	„Richtlinie für die geomechanische Planung von Untertagbauarbeiten mit zyklischem Vortrieb"	2001	
	ÖGG-Richtlinie:	„Richtlinie für die Kostenermittlung für Projekte der Verkehrsinfrastruktur unter Berücksichtigung relevanter Projektrisiken"	2005	
Japan	Japanese Society of Civil Engineers, Japanese Standard for Mountain Tunnelling		1996	English
Switzerland	Swiss Society of Engineers and Architects (SIA)			French English German
	Code SIA 118:	General conditions for construction	1993	
	Code SIA 198/118:	General conditions for underground construction	2004	
	Code SIA 197:	Design of Tunnels	2004	
	Code SIA 197/1:	Design of Railway Tunnels	2004	
	Code SIA 197/2:	Design of Road Tunnels	2004	
	Code SIA 198:	Underground structures, Execution	2004	
UK	British Tunnelling Society: 'The Joint Code of Practice for Risk Management of Tunnel Works in the UK',		2003	

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Towards an improved use of underground Space

In Consultative Status, Category II with the United Nations Economic and Social Council
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Topic

MECHANIZED TUNNELLING

Title

Recommendations and Guidelines for Tunnel Boring Machines (TBMs)

Author

ITA WG Mechanized Tunnelling

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Abstract: -

Résumé: -

Remarks: This report contains four individual reports prepared by ITA Working Group No. 14 ("Mechanized Tunnelling"). The purpose of the reports is to provide comprehensive guidelines and recommendations for evaluating and selecting Tunnel Boring Machines (TBMs) for both soft ground and hard rock. The reports are contributed by representatives from seven countries as follows:

I. "Guide lines for Selecting TBMs for Soft Ground", Japan and Norway
II. "Recommendations of Selecting and Evaluating Tunnel Boring Machines", Germany, Switzerland and Austria
III. "Guidelines for the Selection of TBMs", Italy
IV. "New Recommendations on Choosing Mechanized Tunnelling Techniques", France

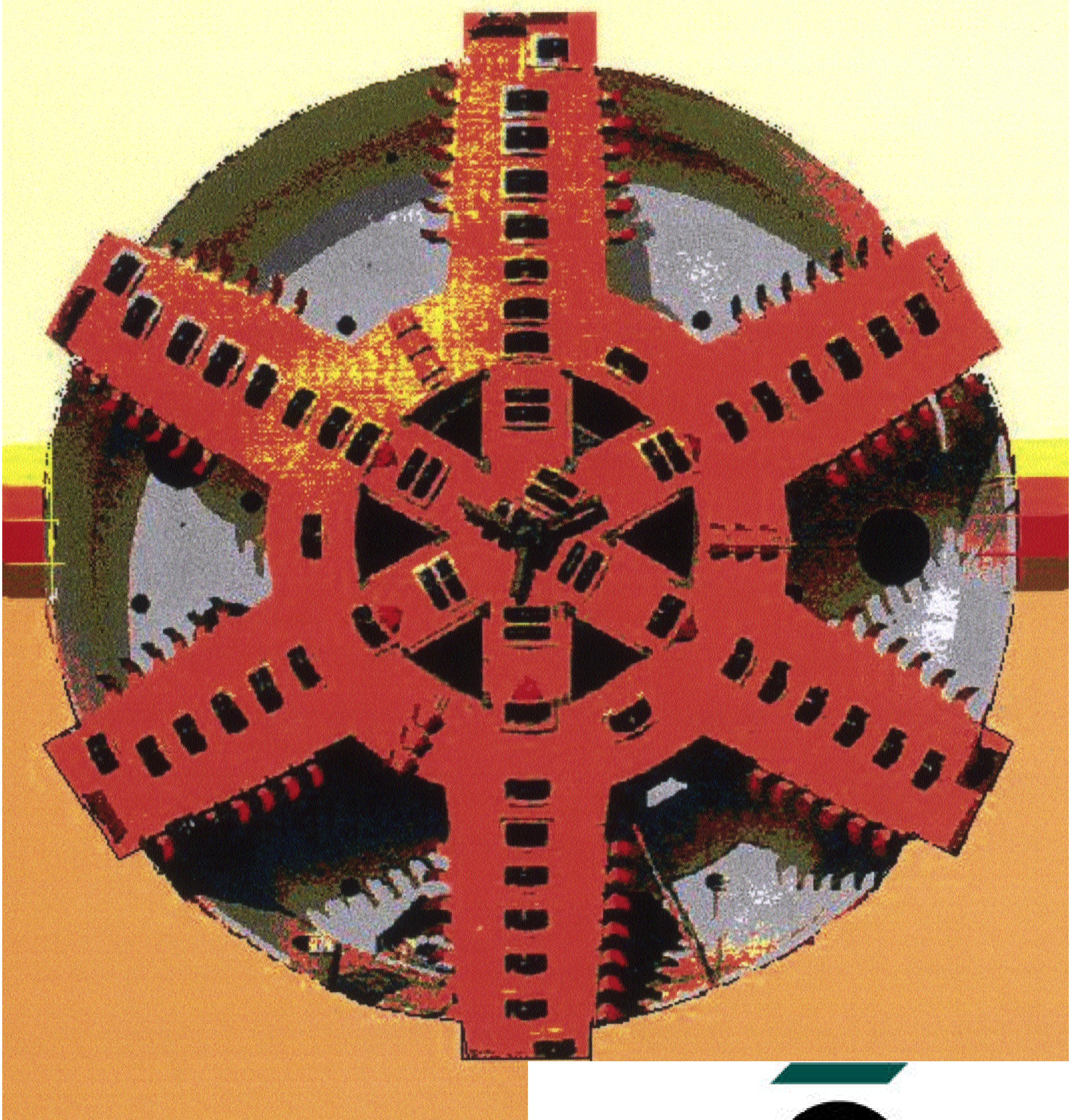
Each report offers up to date technologies of mechanized tunneling for both hard and soft ground and includes, among others, classifications of TBMs, their application criteria, construction methods, ground supporting system and other equipment necessary for driving tunnels by TBMs.

Since a cylindrical steel shield was first used for the construction of the Themes River Tunnel Crossing in England in 1823, tunnel works have been steadily mechanized. Especially, as urban tunneling was developed in the latter half of the 20th century, technological progress seen in this area was remarkable. Meanwhile, the circumstances surrounding tunnel construction have become increasingly complex and difficult. Tunneling technologies in recent years are developed by sophisticated and multi-disciplinary engineering principles to cope with the diverse physical, environmental and social circumstances. This report is intended to provide fundamental and useful knowledge of mechanical tunneling that can be used by designers, manufacturers and the end users of tunnel boring machines.

It is hoped that this report provides common ground for understanding tunneling technologies among international tunneling communities and eventually helps establish a standard set of criteria for designing and utilizing tunnel boring machines.

RECOMMENDATIONS AND GUIDELINES FOR TUNNEL BORING MACHINES (TBMs)

WORKING GROUP N°14 - MECHANIZED TUNNELLING - INTERNATIONAL TUNNELLING ASSOCIATION



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Preface

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Shoji Kuwahara
Tutor, Working Group No. 14
International Tunnelling Association

GENERAL CONTENTS

I. GUIDELINES FOR SELECTING TBMS FOR SOFT GROUND
by Japan and Norway

II. RECOMMENDATIONS OF SELECTING AND EVALUATING TUNNEL BORING MACHINES
by Germany, Switzerland and Austria

III. GUIDELINES FOR THE SELECTION OF TBMS
by Italy

IV. NEW RECOMMENDATIONS ON CHOOSING MECHANIZED TUNNELLING TECHNIQUES
by France

Japan and Norway

Guidelines for Selecting TBMs
for Soft Ground

ITA Working Group No. 14
Mechanized Tunneling

PREFACE

Tunnels are playing an important role in the development of urban infrastructures. Several construction methods for tunneling have been developed to cope with various geological conditions. Those methods can be categorized in two types; drill and blast method and by the use of Tunnel Boring Machine (TBM). This report focuses on tunneling by TBM and is prepared to offer guidelines and recommendations for selecting types of TBMs for urban tunnel construction. Its main purpose is to help project owners, contractors and manufacturers evaluate the applicability and capability of TBMs and other factors that should be taken into consideration for selecting of TBMs.

ITA has been collecting data and information from its member countries, in hope of providing a comprehensive international "manual" for TBM tunneling methods. As the contents of this report represent Japanese and Norwegian versions of the subject, they may be revised or supplemented as necessary to meet particular conditions of the respective countries.

This report consists of two parts; one is for the TBMs in soft ground prepared by Japanese Working Group and the other is for the TBMs in hard rock prepared by Norwegian Working Group. The Norwegian version is an excerpt from the "Project Report 1-94, Hard Rock Tunnel Boring" published by University of Trondheim, Norway, and is included in Appendix. Small diameter tunneling is not included in this report (e.g. micro-tunneling with pipe jacking etc.).

Technologies surrounding TBMs have been receiving great deal of attention. They have been primarily aimed at mechanization and automation of tunnel boring under various geological conditions, with the combined technologies of soil, mechanic and electronic engineering. The technological progress will continue to come from innovative commitments of tunnel builders, teaming with tunnel designers and manufacturers.

It is hoped that this report will assist the members of ITA publish the comprehensive international manual for TBMs and will further contribute to the development of tunneling technologies.

CONTENTS

FIGURES

TABLES

1 CLASSIFICATION OF TUNNEL EXCAVATION MACHINE

Tunnels are constructed under many types of geological conditions varying from hard rock to very soft sedimentary layers. Procedures commonly taken for tunneling are excavation, ground support, mucking and lining. Variety of construction methods have been developed for tunneling such as cut and cover, drill and blast, submerged tube, push or pulling box, and by the use of tunnel boring machine (TBM).

TBM was first put into practical use for mining of hard rock, where the face of the tunnel is basically self-standing. For tunneling through earth, open type machine was used, in which a metal shield was primarily used for protective device for excavation works.

For tunneling through sedimentary soil, tunnel face is stabilized by breasting, pneumatic pressure or other supporting means. Closed type- tunneling machine was developed, which utilizes compressed air to stabilize tunnel face. The closed type-machine started to dominate for soft ground tunneling, especially in the countries where many tunnels are driven through sedimentary soil layers.

Tunnel excavation machines can be classified by the methods for excavation (full face or partial face), the types of cutter head (rotation or non-rotation), and by the methods of securing reaction force (from gripper or segment). Several types of tunnel excavation machines are illustrated in Fig. 1.1 and Fig. 1.2,

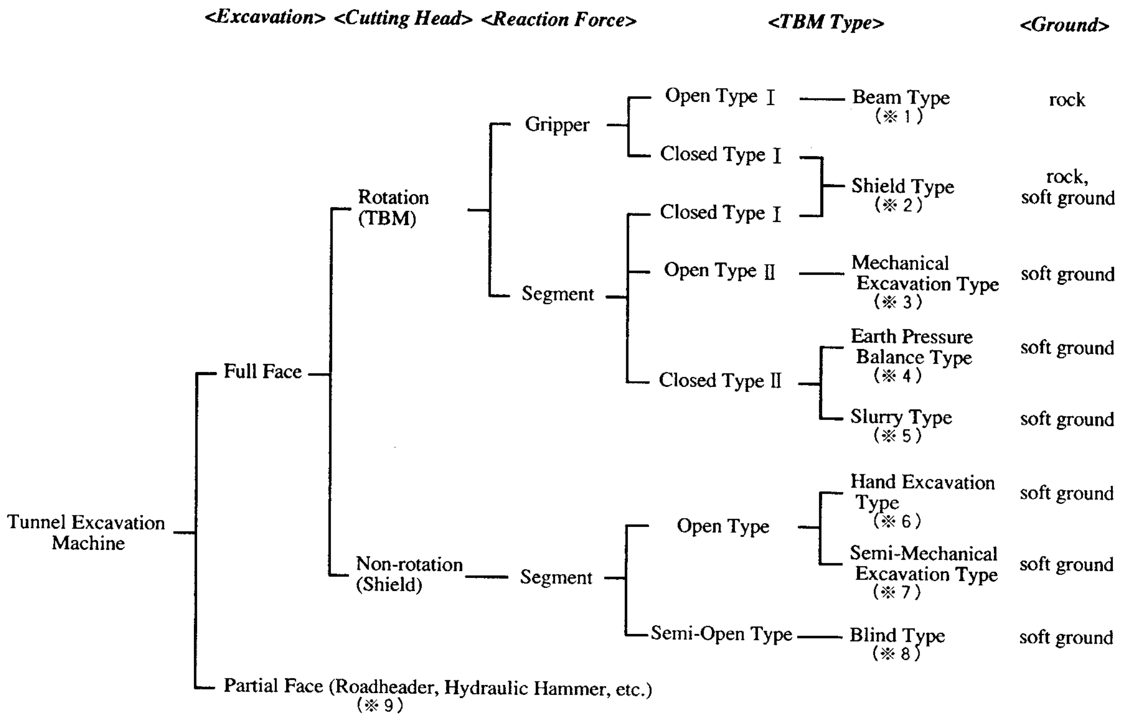
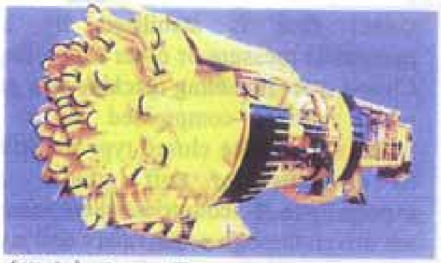


Fig. 1.1 Classification of Tunnel Excavation Machines

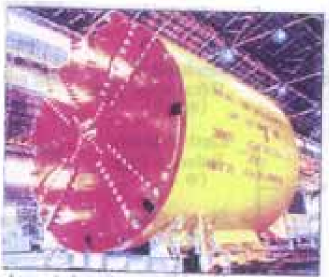


(※ 1) Beam Type

(※ 9) Roadheader

(※ 2) Shield Type

(※ 4) Earth Pressure Balance Type



(※ 3) Mechanical Excavation Type

(※ 5) Slurry Type

(※ 7) Semi-Mechanical Excavation Type

(※ 6) Hand Excavation Type

(※ 8) Blind Type

Fig. 1.2 Tunnel Excavation Machines

1.1 Mechanical Excavation Type (Fig. 1.3)

The mechanical excavation type-tunneling machine is equipped with a rotary cutter head for continuous excavation of tunnel face. There are two types of cutter heads; one is the disk type and the other is the spoke type (rod style radiating from the center). The disk type is suitable for large cross section tunnels where tunnel face is stabilized by the disk cutter head. This type of machine is capable of excavating soils containing gravel and boulders with the openings in the disk, which are adjustable according to the size of gravel and boulders. The spoke type is frequently used for small cross section tunnels where the tunneling face is relatively stable. Gravel and boulders are removed by the rotating spoke cutter.
The mechanical excavation type-tunneling machine is suitable for the diluvial deposit that has a self-standing face. Application of this type of machine to the alluvial deposits, which usually do not form a self-standing face, requires one or more supplementary methods such as pneumatic pressure, additional de-watering, and chemical grouting.

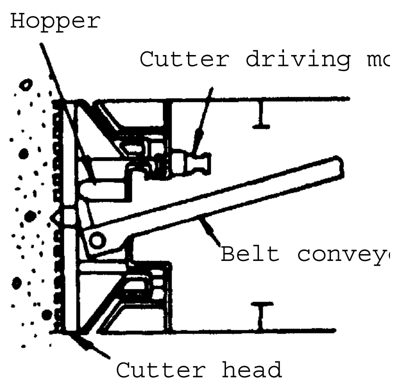


Fig. 1.3 Mechanical Excavation Type Tunneling Machine

1.2 Earth Pressure Balance (E.P.B.) Type (Fig. 1.4)

Earth pressure balance type tunneling machine converts excavated soil into high-density slurry mix. The face of the tunnel is supported by the pressurized slurry mix injected into a space between its cutter head and a watertight steel bulkhead. It consists of the following four components:

i) A cutter head for excavating the ground
ii) A slurry mixer for mixing the excavated muck with high-density slurry
iii) Soil-discharging devise for removal of the muck
iv) Pressure controlling devise for keeping the pressure of slurry-soil mix steady

The earth pressure balance type is classified into two types by the additives injected to convert the excavated muck into high-density slurry. One is earth pressure type and the other is high-density slurry type.

(1) Earth pressure type
Earth pressure type machine cut the ground with a rotary cutter head. Clay-water slurry is injected into the cutter chamber and is mixed with excavated muck. The slurry mix is pressurized to stabilize the tunnel face and create the driving force of the machine. The excavated muck is later separated from the slurry and discharged by a screw conveyor. This type is suitable for clayey soil layers.

(2) High-density slurry type
High-density slurry type machine cut the ground with a rotary cutter head. The excavated muck is mixed with clay-water slurry. by the rotating cutter. Highly plastic and dense additive is added to the slurry mix in the cutter chamber. The additives are used to increase the fluidity and to reduce the permeability of the soil. The high-density slurry mix stabilizes the tunnel face. The excavated muck is discharged by a screw conveyor. This type is suitable for sand or gravel layers.

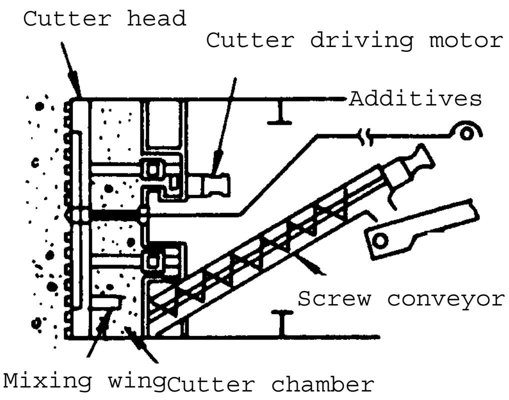


Fig. 1.4 Earth Pressure Balance Type Tunneling Machine

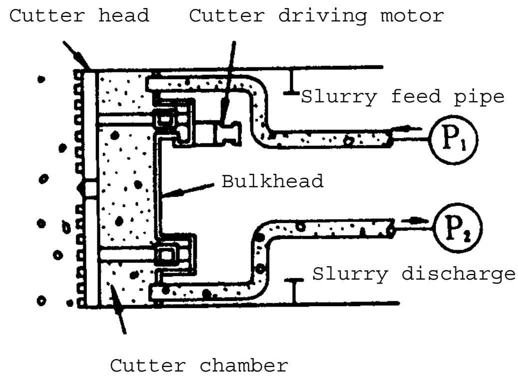


Fig. 1.5 Slurry Type Tunneling Machine

1.3 Slurry Type (Fig.1.5)

Slurry type tunneling machine cut the ground with a rotary cutter head. The cutter chamber is filled with pressurized slurry mix to stabilize the face of the tunnel. The slurry mix is circulated through pipes to transport it to a slurry treatment plant where the excavated muck is separated from slurry mix. The excavated muck is discharged through pipes and the slurry is circulated back to the cutter head for re-use. The slurry type machine consists of the following three components:

- i) A rotating cutter head for excavating ground
- ii) A slurry mixer for the production of slurry mix with desired density and plasticity
- iii) Slurry pumps to feed/discharge, circulate and to pressurize slurry mix
- iv) Slurry treatment plant to separate excavated muck from slurry

2 INVESTIGATIONS OF EXISTING CONDITIONS AND APPLICABILITY OF TBM

2.1 Site Investigations

Site investigations are conducted to obtain basic data necessary for determining the project scale, selection of a tunnel route and its alignment, applicability of TBMs, and its environmental impact, and for planning, designing and construction of TBM tunnels. Results of the investigations are also used for operation and maintenance of TBM. The major items of investigation are indicated in the following subsections.

2.1.1. Existing site conditions

Existing site conditions along the proposed tunnel route are investigated to survey the following site conditions

- i) Land use and related property rights
- ii) Future land use plan
- iii) Availability of land necessary for construction
- iv) Traffic and the type of the roads
- v) Existing rivers, lakes and ocean
- vi) Availability of power, water and sewage connections

Results of the investigation are mainly used for determining the tunnel route, its alignment, locations and areas of access tunnels and temporary facilities.

2.1.2. Existing structures and utilities

Existing structures and utility lines near the tunnel are investigated for their future preservation and for securing the safety of TBM tunneling.

- i) Existing surface and underground structures
- ii) Existing utilities
- iii) Wells in use and abandoned
- iv) Remains of removed structures and temporary structures

2.1.3. Topography and geology

Topographical and geological conditions are the most important factors affecting the TBM design and construction. In particular, the following items should be investigated by field survey, boring, etc.

- i) Topography
- ii) Geological structure
- iii) Ground conditions
- iv) Groundwater

2.1.4. Environmental impact

Environmental impact analysis of the tunnel construction should be carried out to select and design construction methods that minimize the environmental impacts to the existing ecosystem.

- i) Noise and vibration
- ii) Ground movement
- iii) Groundwater
- iv) Oxygen deficient air and hazardous gas such as methane gas
- v) Chemical grouting
- vi) Discharge of excavated muck

2.2 Applicability of TBMs

Three types of excavation methods, drilling and blasting, TBM for hard rock, and TBM for soft ground, are compared in terms of tunnel dimensions, geological conditions and environmental impacts, and are shown in Table 2.1. The shaded portions of this table indicate the application of TBMs for soft ground.

Among the soft ground TBMs, the mechanical excavation type, earth pressure balance type and slurry type is compared in **Table 2.2** in terms of their applicability to various types of soft ground. This table also indicates the items that should be taken into consideration when applying TBM to soft ground. .

As indicated in **Table 2.2**, earth pressure balance and slurry types are suitable for alluvial deposits that generally are not self-standing. Slurry type is effective for driving through grounds with high groundwater pressure, such as those under river or seabed because the stability of tunnel face can be maintained by properly mixed and pressurized slurry mix. On the other hand, earth pressure balance type is not suitable for grounds with high groundwater pressure because it is difficult to maintain the pressure balanced against ground water pressure due to the opening for the soil discharging screw conveyor.

Table 2.1 Comparison of Excavation Methods

Conditions	Excavation Method	Drilling and Blasting	TBM: For Hard Rock	TBM: For Soft Ground
Tunnel Features	tunnel length	Equipment cost is relatively low. Excavation cost is not greatly influenced by the tunnel length.	The cost of tunnel boring machines is generally high. It is suitable in longer tunnel excavations.	The cost of tunnel boring machines is generally high. It is suitable in longer tunnel excavations.
	shape of the cross section	Basically, the shape of excavation has an arched shape at the crown. The shape of the section can be changed during the construction.	Basically, the shape of the excavation is a circle. After boring, other shapes are possible using drilling and blasting as the result of enlargement.	Basically, the shape of the excavation is a circle. Semicircle, multi-circle, oval etc. are also possible using special tunneling machines for excavation.
	size of the cross section	Generally, it is possible up to 150m ^{2} The largest record is bigger than 200m ^{2}.	The largest record is approximately 12m for the maximum diameter of the tunnel.	The largest record is approximately 14m for the maximum diameter of the tunnel.
Geological Conditions	hard rock	Suitable	Suitable except for the extra-hard rock (s>200MPa)	Not applicable
	semi-hard rock	Suitable	Suitable	Not applicable
	Weak layers such as fractured zones and aquifer zones	Various countermeasures become necessary.	It is not suitable in area where weak ground or water inflow will be frequently encountered.	Applicable See Table.2.2
	Soil	Not applicable	Not applicable	Most suitable See Table 2.2
Environmental Conditions	Noise and Vibration	Due to noise and vibration, it is not suitable in the vicinity of houses and important structures. A supplementary method is necessary to reduce the effects of noise and vibration.	Compared to the drilling and blasting there is less effect of noise and vibration to the houses and important structures.	There is less effect of noise and vibration to the houses and important structures than other excavation methods.

Table2.2 Applicability of TBMs to Soft Ground Conditions

TBM type / Ground condition	N-value	water content or permeability	Open type: Mechanical excavation type		Closed type: Earth pressure balance type: Earth pressure type		Closed type: Earth pressure balance type: High-density slurry type		Closed type: Slurry type	
Alluvium clay	0 – 5	300% – 50%	s	-Face stability -Ground settlement	l	-Difficulty in extremely weak clay -Volume control of discharged soil	–	-Earth pressure is more suitable.	s	-Difficulty In extremely weak clay -Slurry spouting on surface -Increase of secondary slurry treatment plant
Diluvium clay	7 – 20	W < 50%	l	-Existence of water bearing sand -Blockage in slit chamber	l	-Liquidity of soil -Volume control of discharged soil	–	-Earth pressure is more suitable.	l	-Increase of secondary slurry treatment plant
Soft rock (mudstone)	> 50	W < 20%	l	-Existence of water bearing sand -Wear of cutter bits	–	-Earth pressure with slurry is more suitable when there is water bearing sand.	–	-Suitable when there is water bearing sand	–	-Suitable when there is water bearing sand
Loose sand	5 – 30	$10^{-2} – 10^{3}$ (cm/s)	x	-Unstable face	s	- Contents of fine particles	l	-Highly-advanced excavation control	l	-Highly-advanced excavation control -Quality control of slurry solution
Dense sand	> 30	$10^{-3} – 10^{-4}$ (cm/s)	s	-Face stability -Groundwater level, permeability	s	- Contents of fine particles	l	-Wear of cutter bits -Dosage of additives	l	-Quality control of slurry solution
Sand gravel	> 30	$10^{0} – 10^{2}$ (cm/s)	s	-Face stability -Groundwater level, permeability	s	- Contents of fine particles	l	-Wear of cutter bits -Dosage of additives	l	-Running away of slurry -Gravel crusher -Fluid transportation system
Sand and gravel With boulders	> 50	$10^{0} – 10^{1}$ (cm/s)	x	-Face stability -Boulder crusher -Wear of cutter bits and face	s	- Contents of fine particles -Wear of cutter bits and fac -Boulder crusher -Boulder diameter for screw-conveyer	l	-Wear of cutter bits -Boulder crusher -Boulder diameter for screw-conveyer	s	-Running away of slurry -Boulder crusher -Fluid transportation system
Applicability for ground Condition changes			It is impossible to change excavation system.		Applicable Additive injection equipment becomes necessary.		Applicable In general, it is widely applicable for various soil conditions.		Applicable In general, It is widely applicable for various soil conditions.	

Note: l Applicable, s Consideration required x Not applicable
Items to consider when applying
In case of x, reasons for not applicable

3 TUNNEL BORING MACHINE (TBM)

3.1 Machine Specifications

3.1.1. Essential parts of TBM

TBMs are normally manufactured in drum-shaped steel shield equipped inside with excavation and segment erection facilities. The essential parts of the machine include the following items:

i) Rotary cutter head for cutting the ground
ii) Hydraulic jacks to maintain a forward pressure on the cutting head
iii) Muck discharging equipment to remove the excavated muck
iv) Segment election equipment at the rear of the machine
v) Grouting equipment to fill the voids behind the segments, which is created by the over excavation.

3.1.2. Structure of TBM

TBM is composed of the steel shell (so called the shield) for protection against the outer forces, equipment for excavation of soil and for the installation of the lining at the rear. The power and control devices are mounted partly or totally on the trailing car behind the machine, depending on the size and structure of the machine. Steel shell, made of the skin plate and stiffeners, is composed of three portions; hood, girder and tail portion (see Fig. 3.1).

In case of the closed type machine, hood and girder portions are separated by a bulkhead. The soil excavated by the cutter head is taken into the mucking device through the hood portion. In some cases, man-lock is installed at the bulkhead in order to change the cutter bits or to remove obstacles under the pneumatic pressure.

For manual type, breasting is provided at the hood portion. The reaction force is supported by the girder portion where the thrusting devices are installed.
The tail portion of the machine is equipped with erector of the segments. Tail seal for water stop is inserted between the skin plate and the segment ring.
In case of the articulating system, the girder portion is made flexible by dividing the portion into two or more bodies with pins and jacks. Such flexible separation of the body is adopted to allow a smooth turn along the curved alignment of the tunnel with different diameters of the machines, degrees of allowance of over cutting and under various soil conditions,.
When two tunneling machines are connected underground, the alignments and the relative positions of the two machines have to be carefully monitored and adjusted. The final connection normally requires some soil improvement work such as ground freezing, or else with extendable cutter head or hood equipped on either one of the machines.

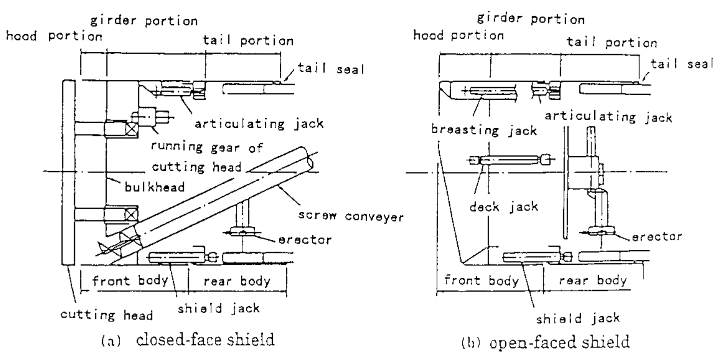


Fig. 3.1 Components of Tunneling Machine

3.1.3. Types of TBMs for soft ground

As described in the previous section, TBMs for soft ground are classified into three types; earth pressure type, slurry type and mechanical type. These three types of TBMs are summarized in Fig 3.2. Before those three types were developed, other types of TBMs such as the open type, blind type, manual type, and half-mechanical type were used for soft ground. The open type TBM is now mostly replaced by closed type for soft ground tunneling.

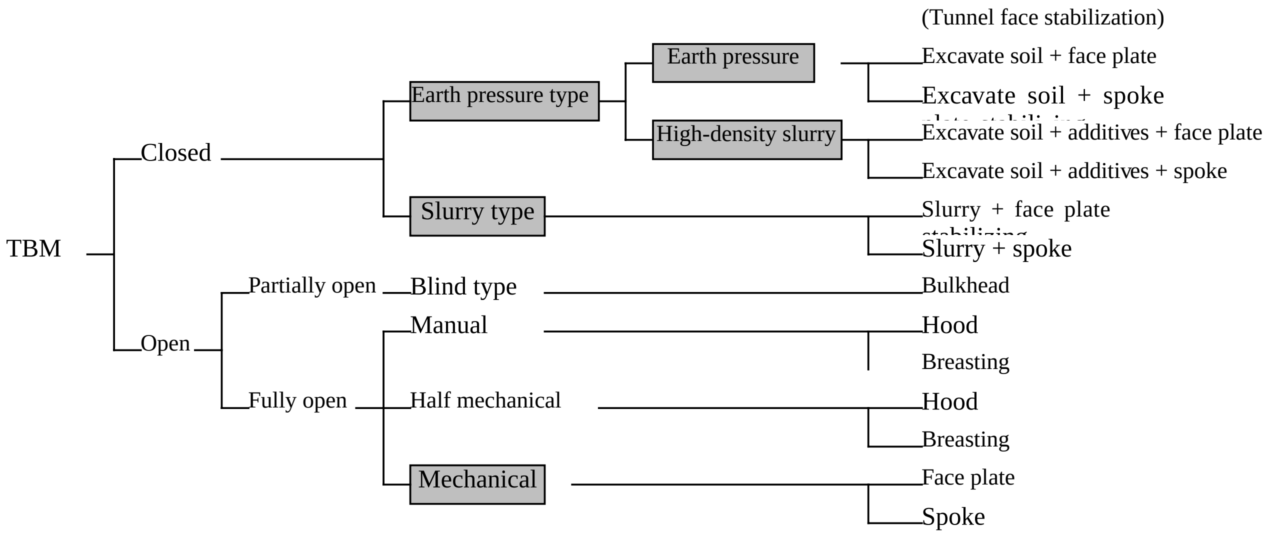


Fig. 3.2 Type of TBM for Soft Ground

3.1.4. Selection of TBM

Careful and comprehensive analysis should be made to select proper machine for soft ground tunneling taking into considerations its reliability, safety, cost efficiency and the working conditions. In particular, the following factors should be analyzed:

- i) Suitability to the anticipated geological conditions
- ii) Applicability of supplementary supporting methods, if necessary
- iii) Tunnel alignment and length
- iv) Availability of spaces necessary for auxiliary facilities behind the machine and around the access tunnels
- v) Safety of tunneling and other related works.

Fig. 3.3 indicates a flow chart for selecting TBM for soft ground. In selecting the type of TBM, it is important to consider geological and groundwater conditions that affect the stability of the tunnel face.

Geological condition along the tunnel route is a primary factor to be considered for selecting the type of machine. Particularly, the degree of consolidation of the ground and the size of gravel and boulders in the soil should be thoroughly investigated. Table 3.1 shows the general relationship between the closed type of tunneling machine and soil conditions. In a case where a tunnel is very long or is under complex geological conditions, uniform layers could not be expected throughout the entire length of the tunnel. In such case, a tunneling method is selected based on the geological condition prevailing throughout the tunnel. Special attention should be paid to the following local geological conditions:

i) Soft clayey soil that is sensitive and easy to collapse

ii) Sand and gravel layers with high water contents

iii) Layers which contain boulders

iv) Layers which may contain driftwood or ruins

v) Strata which are composed of both soft and hard layers

Slurry type is easy to be automatically controlled and is the most advanced excavation

method for soft ground tunneling because of its reliability, safety and the minimum disturbance to surrounding ground.
Both earth pressure balance type and slurry type generally does not require supplementary supporting methods under ordinary conditions. The supplementary methods should be considered, however, for tunneling at starting and arrival area where the face of the tunnel is difficult to be stabilized. Also, some supplementary methods such as chemical grouting, ground freezing, pneumatic pressure and boulder crushing are required to drive through grounds with boulders or gravel, under thin overburden or any other special conditions.

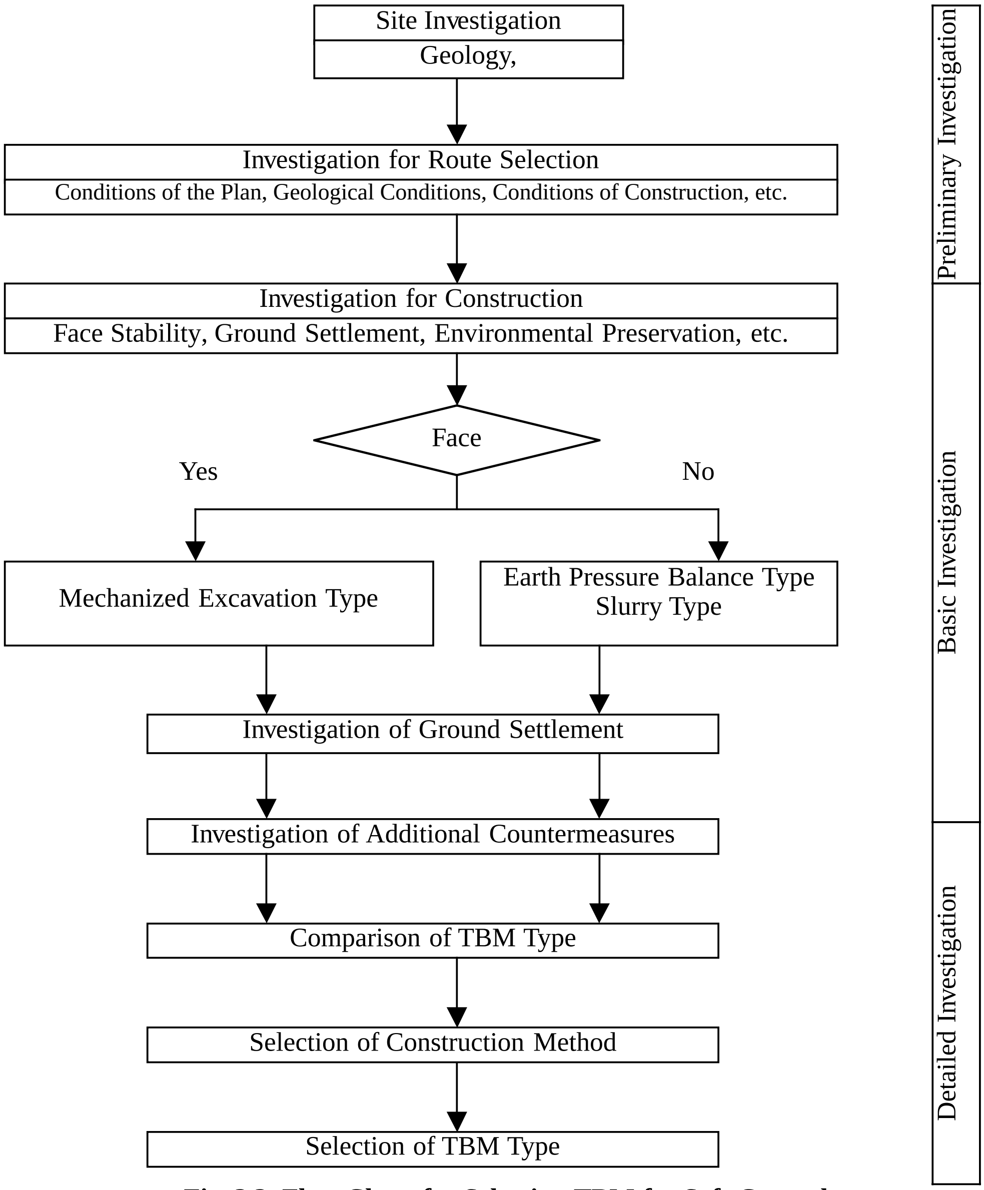


Fig. 3.3 Flow Chart for Selecting TBM for Soft Ground

Table 3.1 Relationship between Closed Type Tunneling Machine and Soil Conditions

Soil conditions \ Type of machine		N-value	Earth pressure balance type				Slurry type	
			Earth pressure type		High-density slurry type			
			Suitability	Check point	Suitability	Check point	Suitability	Check point
Alluvial clay	Mold	0	x	_	s	settlement	s	Settlement
	Silt, Clay	0 – 2	l	_	l	_	l	_
	Sandy silt	0 – 5	l	_	l	_	l	_
	Sandy clay	5 – 10	l	_	l	_	l	_
Diluvial clay	Loam, Clay	10 – 20	s	Jamming by excavated soil	l	_	l	_
	Sandy loam	15 – 20	s	ditto	l	_	l	_
	Sandy clay	Over 25	s	ditto	l	_	l	_
Solid clay	Solid clay (muddy pan)	Over 50	s	ditto	s	Wear of bit	s	Wear of bit
Sand	Sand with silty clay	10 – 15	l	_	l	_	l	_
	Loose sand	10 - 30	s	Content of clayey soil	l	_	l	_
	Compact sand	Over 30	s	ditto	l	_	l	_
Gravel Cobble stone	Loose gravel	10 – 40	s	ditto	l	_	l	_
	Compact gravel	Over 40	s	High water pressure	l	_	l	_
	Gravel with cobble stone	_	s	Jamming of screw conveyor	l	_	s	Wear of bit
	Large gravel Cobble stone	_	s	Wear of bit	s	Wear of bit	s	Crushing device

l :normally applicable s:applicable with supplementary means x :not suitable

3.2 Orientation and operation of machine

3.2.1. Excavation Control System

Since the closed type machine was developed, tunnel excavation has been mostly controlled by computerized system rather than manually. In addition, various supporting systems necessary for tunneling operation require sophisticated controlling system. A real time computerized system equipped with various sensors is developed for tunneling, in which orientation and operation of machine, excavation, backfill grouting and operation of auxiliary facilities are controlled by a centralized computer system. The system realized accurate alignment, excavation control that maintains the stability of the face of the tunnel, and minimized the disturbance of the surrounding ground. For slurry type tunneling machine, operation of pumps and valves for slurry transportation is computerized based on the data fed by pressure gauges, flow meters and other measuring devices for fluid transportation. Thus, steady pressure of slurry is maintained throughout the tunneling operation.

In the near future, all operation of the machine will be entirely controlled by computerized system from above ground.

3.2.2. Direction Control and Measurement System

Automatic direction control system has been put to practical use that utilizes survey data obtained by real time measurement device instead of the conventional transit-level survey. The system consists of measurement and direction control systems, and comprises of four functions; survey, monitor, analysis and control. The measurement system utilizes laser beam (laser, infrared or diode) or gyrocompass, and measures the location of the machine in three-dimensional coordinates and its attitude (pitching, rolling and yawing).

Direction of the machine is normally controlled by jacks that introduce proper thrust force and rotation moment. Each jack on a cutter disk is controlled by a computerized system based on the target amount of thrust and the direction of machine. In the process of determining the amount of thrust required for each individual jack, a mathematical theory of “fuzzy control theory” has been applied based on the date accumulated through the past performance of the machine. Recent automatic direction control system realizes accuracy of plus or minus 30 mm both horizontally and vertically.

3.3 Cutter Consumption

3.3.1. Bit types and Arrangement

There are several types of bits for TBM, such as teeth bit, peripheral bit, center bits, gouging bit, wearing detection bit, etc. Bits are generally made by steel or hard chip alloy that is highly wear resistant. Selection of material and types of bits is made based on the ground conditions, excavation speed and length of the tunnel. Arrangement of bits on the cutter head is decided based on construction conditions, past experience in similar geology, cutting depth and the number of passes of rotating bits.

3.3.2. Wear of Bit

Generally, the amount of wear of bits is proportional to the product of number of passes of rotating bits and length per pass, and is influenced by ground conditions and other factors such as type of machine, geology, material and arrangement of the bits on a cutter head. The amount of wear can be estimated by the following formula;

$$d =(K.\pi.D.N.L)$$

here, d: amount of wear (mm)
K: wear coefficient (mm / km)
D: distance between the center of cutter disk and bit (m)
N: number of revolution of cutter disk per minute (rpm)
L: excavation distance (m)
V: rate of excavation (mm / min)

The wear coefficient, K, above is given by manufacturers based on the pressure applied to bits, the rotating speed, geological conditions, number of passes and material of bits to be used.

3.3.3. Long Distance Excavation

Sometimes, a tunneling machine is required to drive through entire length of tunnel when access tunnels for installation of two or more

machines cannot be constructed due to the lack of land available. In that case, the tunneling machine, especially the cutting bit and tail seal, is required to be highly durable.
For higher durability of the bits, new chipping material such as hard chip alloy has been developed, which are two or three times durable than those of conventional material. Bits can be changed from inside the TBM. Durability of tail seals and the method of changing them are being improved as well.

3.4 Ground Support and Lining

3.4.1. Design of Lining

The linings of the tunnel must withstand the soil and water pressure acting on the tunnel. Primary tunnel lining is usually constructed by prefabricated concrete segments erected around the periphery of the tunnel. Those segments are connected each other to form circular rings which are installed side by side continuously to form a cylinder. The second lining, when required, is normally constructed by in-situ concrete.

Usually primary lining is designed as a main structural member against the final load, because the secondary lining is installed long after the erection of segments. Therefore, the role of the secondary lining is mostly not for the main structural member, but for the supplementary member for water proofing, anticorrosion, etc. Secondary lining is omitted to save costs when the primary lining is watertight enough or the ground conditions are favorable.

For the design of the segment, several loads and their combination should be considered (see Table 3.2). Temporary loads that vary during the construction such as thrust force by jacks and grouting pressure should be also taken into consideration.

The effects of joints between segments and rings should be carefully assessed when designing segment lining. As several segments are pieced together to produce a ring, the ring may not deform uniformly against the surrounding loads due to weakness at segment joints. The same can be said to the joints between rings. Staggered arrangement is made to reduce these effects of the joints.

Under present design method, segment ring assumes to be a uniform flexural ring, a multi-hinged ring or a ring with rotational springs.

Table 3.2 Loads on Segments

Main load	vertical and horizontal earth pressure water pressure dead load surcharge load ground reaction
Secondary load	internal load temporary load during execution seismic load
special load	Influences of adjacent tunnel of adjacent structures of ground settlement others

3.4.2. Types of Segment

As the cost of segments shares significant portion of total tunneling cost, type of segment should be carefully selected from both engineering and economical points of view. Segments are classified into several types; reinforced concrete (RC), steel, cast iron (ductile), composite, and others.

Reinforced concrete prefabricated segments are most commonly used for tunnels driven by TBMs. Reinforced concrete segment is an excellent lining member with high compressive strength against both radial and longitudinal forces. It also has high rigidity and water tightness. On the other hand, it is heavy and has less tensile strength and more fragile than steel ones. Therefore, extreme care should be taken to the removal of forms during fabrication and to the erection during construction in order to avoid possible damages to segments, especially to their corners. Rectangular shaped segments are commonly used, but hexagonal or other shapes are also produced. They can be either solid or box type.

Steel segment is flexible and is relatively light and easy in handling. Because of the flexibility of steel segment, they should not be subjected to high thrusting force of jacks or grouting pressure to avoid buckling or unnecessary deformation. When the second lining is omitted, proper anticorrosion measures should be taken.

Cast iron (ductile) segment is produced with precise dimensions and therefore can be erected with good water tightness. Because of its strength and durability, it is commonly used at locations under heavy loads or for reinforcing tunnel openings.

In addition to above three types of segments, various types have been used or proposed, such as composite segments (steel and RC, steel and plain concrete), flexible segment that allows certain degree of deformation caused by earthquake or uneven ground settlement. Also, there are several types of radial and longitudinal segment joints such as bolt, cotter, pin and pivot, knuckle and other joint types.

3.4.3. Fabrication of Segment

Fabrication of segments has to be carried out under strict quality control to ensure compliance with specified dimensions and strength. Automated fabrication of segments is desired that provide adequate quality control to ensure structural integrity and precise dimensions of segments.
Table 3.3 provides allowable stresses of concrete for pre-fabricated reinforced concrete segment.

Table 3.4 provides typical dimensions of steel and concrete segments.

Table 3.3 Allowable stresses of concrete for pre-fabricated concrete segments

	Allowable stress (N/mm)				
Design compressive strength	42	45	48	51	54
Bending compressive stress	16	17	18	19	20
Shearing stress	0.71	0.73	0.74	0.76	0.77
Bonding stress to deformed re-bar	2.0	2.1	2.1	2.2	2.2
Bearing stress (overall load)	15	16	17	18	19

Table 3.4 Typical Dimensions of Segments (mm)

	Steel Segment			Concrete Segment		
Outer Diameter	Width	Thickness	NO/Ring	Width	Thickness	NO/Ring
1,800 _ 2,000	750	75 100	6	900	100 125	5
2,150 _ 2,550	900 1,000	100 125	6	900 1,000	100 125 150	5
2,750 _ 3,350	900 1,000	125 150 175	6			
3,550 _ 4,050	900 1,000	125 200 225	7	900 1,000	125 150 175 200	6
4,300 _ 4,800	900 1,000	150 175	7			
5,100 _ 5,700	900 1,000	175 200 225	7	900 1,000	175 200 225 250 275 300	6
6,000	900 1,000	200 225	7			
6,300 _ 6,900	900 1,000	250 275	7	900 1,000	250 275 300	7
7,250 _ 8,300	900 1,000	300 325 350	8	900 1,000	275 300 325 350	8

3.4.4. Erection of Segments

The process of primary lining consists of transportation and erection of segments. Segments are usually transported through the tunnel by cars on rails. Automatic transportation system of segments is used to recent projects that transport segments from a depot above ground to the rear end of the machine through access shaft and tunnel.

The erection of segments is done by an erector at the rear room of the machine. The segment erector is equipped with gripping, shifting, rotating and setting devices. Longitudinal joints of segment rings are normally made manually.

3.5 Auxiliary Facilities

Generally, tunneling operation by TBM consists of cutting ground by cutter head, jacking to push machine forward, muck transportation, segment erection and grouting of voids behind segments. Auxiliary facilities that are typically required throughout this operation are shown in Table 3.5.
Common facilities are gravel treatment plant, grouting facilities, segment depot and treatment facilities. For the discharge of excavated muck, different types of facilities are required depending on the type of tunneling machines as follows.

3.5.1. Earth pressure balance type machine

The excavated muck is removed from the cutter chamber by a screw conveyor and sent out by mucking car or belt conveyor. For small diameter tunnels where working space is quite limited, the excavated muck is mixed with plasticizer and pumped out through the pipe. For these operations, additive mixing plant, a screw conveyor and belt conveyor or mucking cars are required.

3.5.2. Slurry type tunneling machine

Sequence of discharging the excavated muck for this type of machine consists of; (i) pouring slurry to the cutter chamber while the soil is excavated and the machine is pushed forward, (ii) mixing excavated soil with slurry and pumping the slurry mix from the cutter chamber to a treatment plant where the slurry mix is separated into soil and slurry, (iii) discharging the separated soil out to the disposal area and circulating the slurry back to the tunnel face for reuse. Auxiliary facilities required for these operations are slurry mixer, feed and discharge pumps and pipes, and slurry treatment plant.

Table 3.5 Auxiliary Facilities

<table>
<tr><th>Earth pressure balance type</th><th>Slurry type</th></tr>
<tr><td colspan="2">- Segment pool and transportation facilities for segments and materials
- Central control room
- Gravel treatment facilities, such as crushing device
- Grouting facilities for back fill</td></tr>
<tr><td>- Belt conveyor, mucking cars or pumps
- Additive mixing plants</td><td>- Slurry transport facilities, such as slurry pumps and pipes.
- Slurry treatment facilities, such as centrifugal classifier and filter plant</td></tr>
</table>

4 TUNNELS CONSTRUCTED BY TBM IN JAPAN

4.1 Soft ground tunneling in Japan

Soft ground tunnels driven by TBMs in Japan since 1994 are shown in Table 4.1 below.

Table 4.1 TBMs in soft ground performed in Japan

Tunneling Annual Report (JTA 1994)

	Mechanical				Total	Ratio(%)	Length/ Project
	Mechanical	Slurry	EPB	EPB with Slurry			
Railway	1	23	13	38	75	13.8	
	2600	26864.9	14703.7	43957	88125.6	14.4	1175.0
Road		7			7	1.3	
		7961			7961	1.3	1137.3
Water supply		11	5	16	32	6.0	
		12975.3	6586.6	23293.4	42855.3	7.0	1339.2
Sewer	1	87	43	209	340	62.7	
	421	101393.2	41653.3	217278.4	360745.9	59.1	1061.0
Utility		35	3	39	77	14.2	
		48473.5	2215.5	36112.2	86801.2	14.2	1127.3
Other		2	3	6	11	2.0	
		12432	5316	6633.8	24381.8	4.0	2216.5
Total	2	165	67	308	542	100.0	
	3021	210099.9	70475.1	327274.8	610870.8	100.0	1127.1
Ratio(%)	0.4	30.4	12.4	56.8	100.0		
	0.5	34.4	11.5	53.6	100.0		
Length/Project	1510.5	1273.3	1051.9	1062.6	1127.1		

Upper: No. of projects Lower: Tunneling length (m)

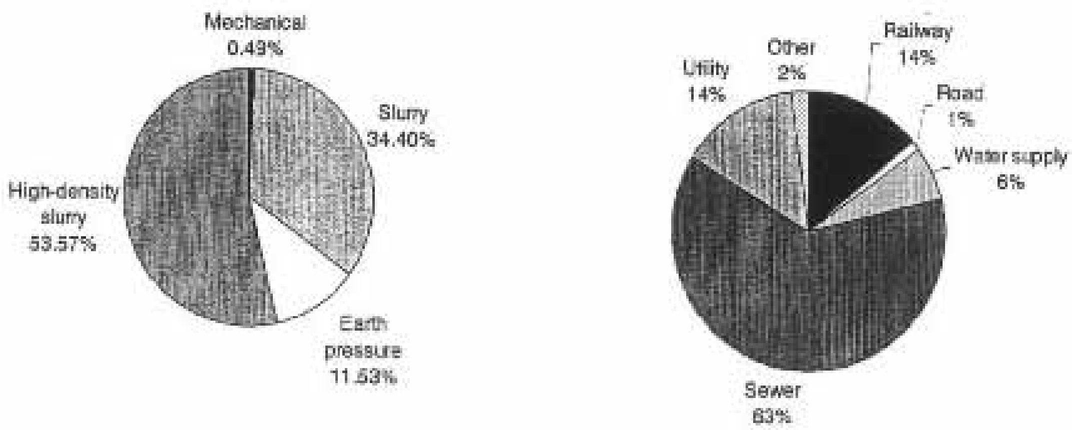
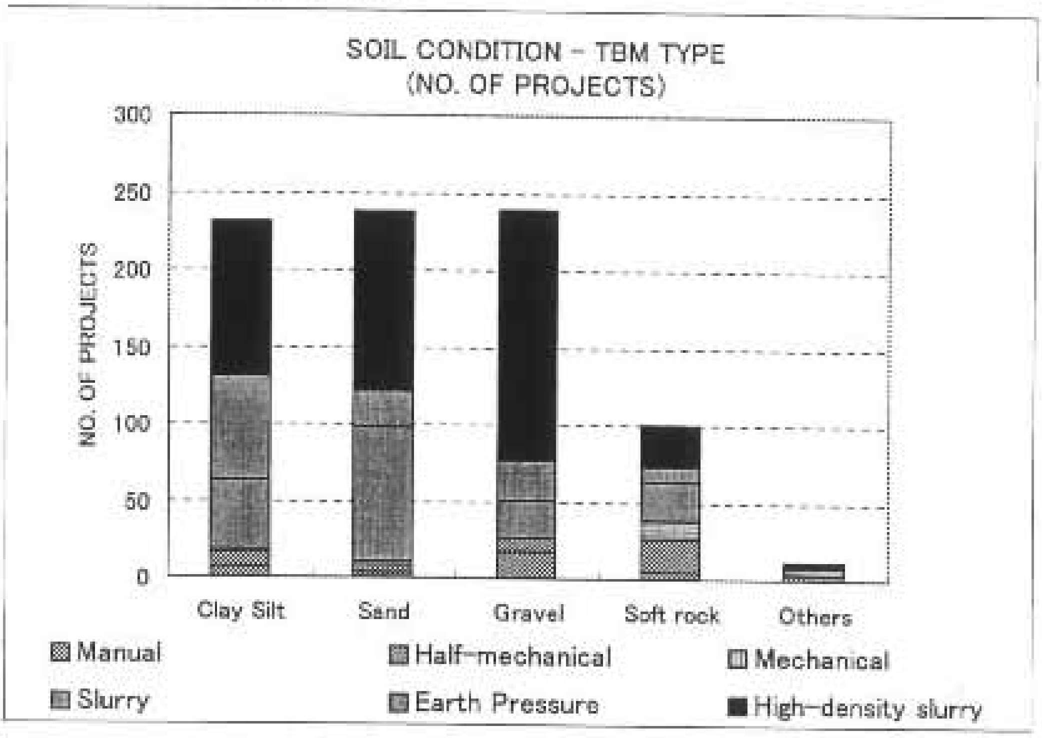


Fig. 4.1 Tunnels driven by TBMs in Japan

4.2 Types of TBMs and ground conditions

4.2.1. Soil Conditions and Types of TBMs



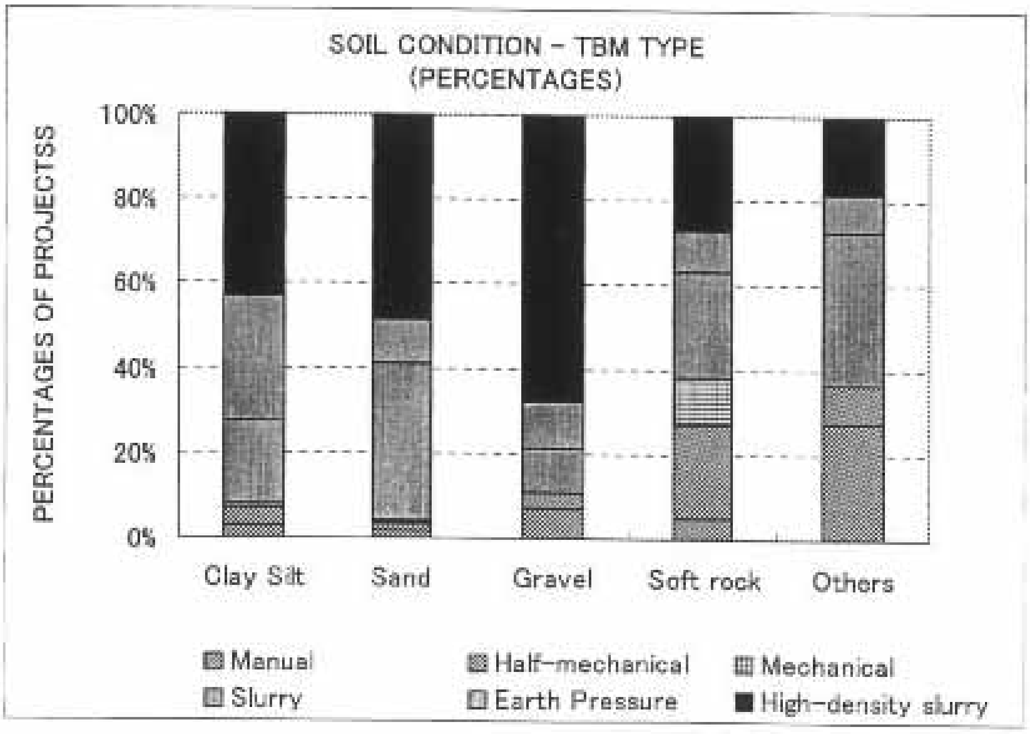
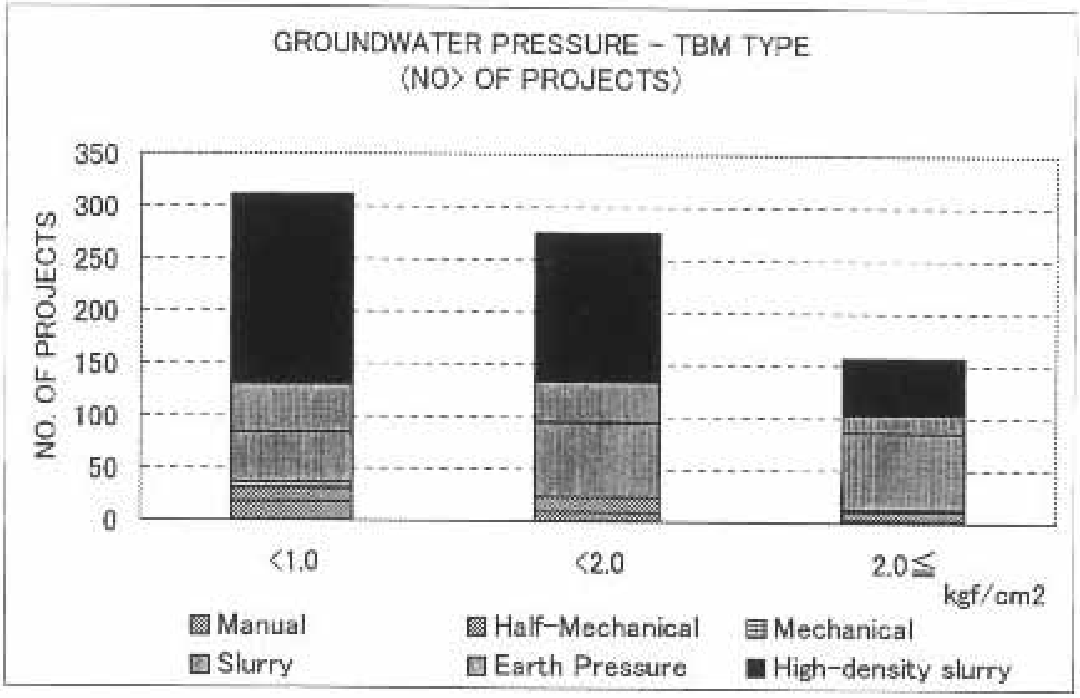


Fig. 4.2 Soil conditions and TBM methods

4.2.2. Groundwater pressure and Type of TBMs



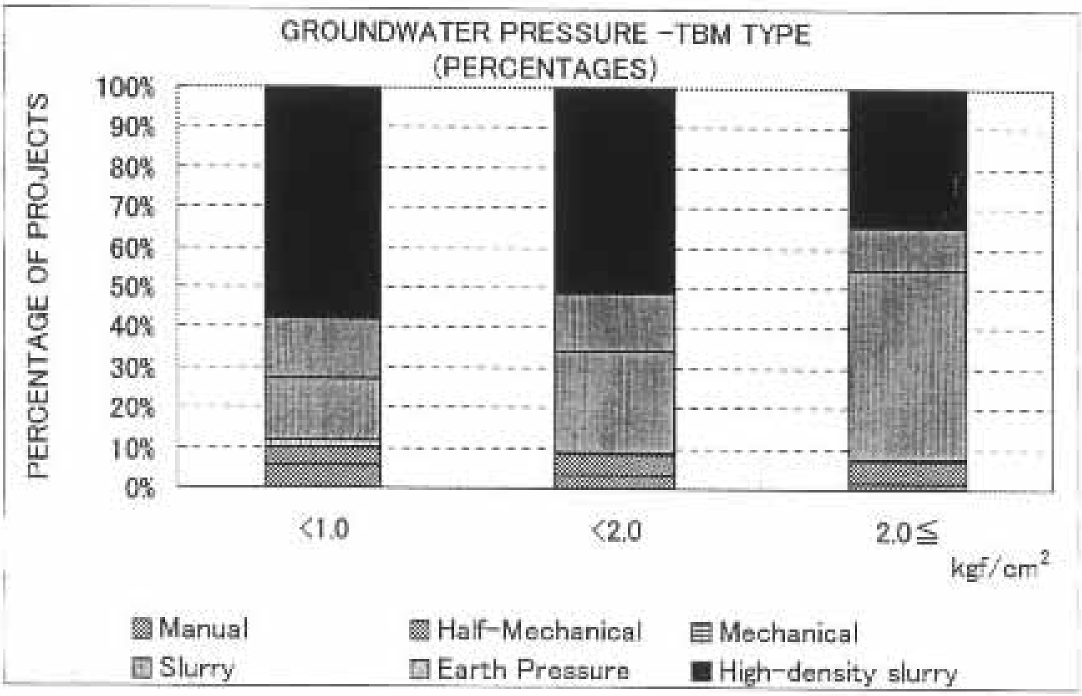
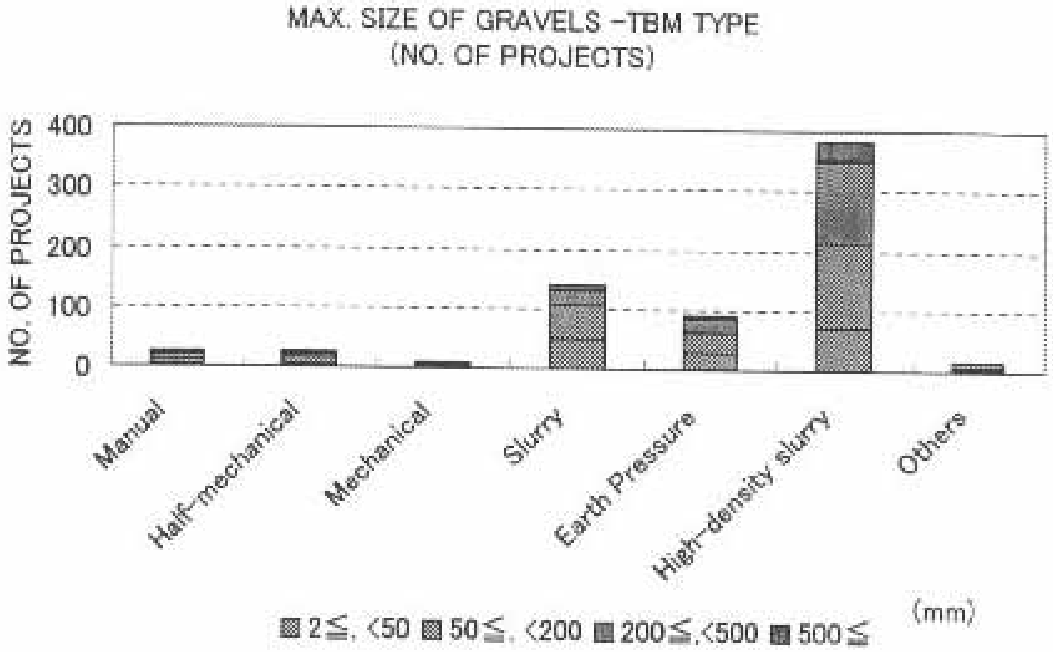


Fig. 4.3 Groundwater Pressure and Type of TBMs

4.2.3. Max. Size of Gravel and TBM Type



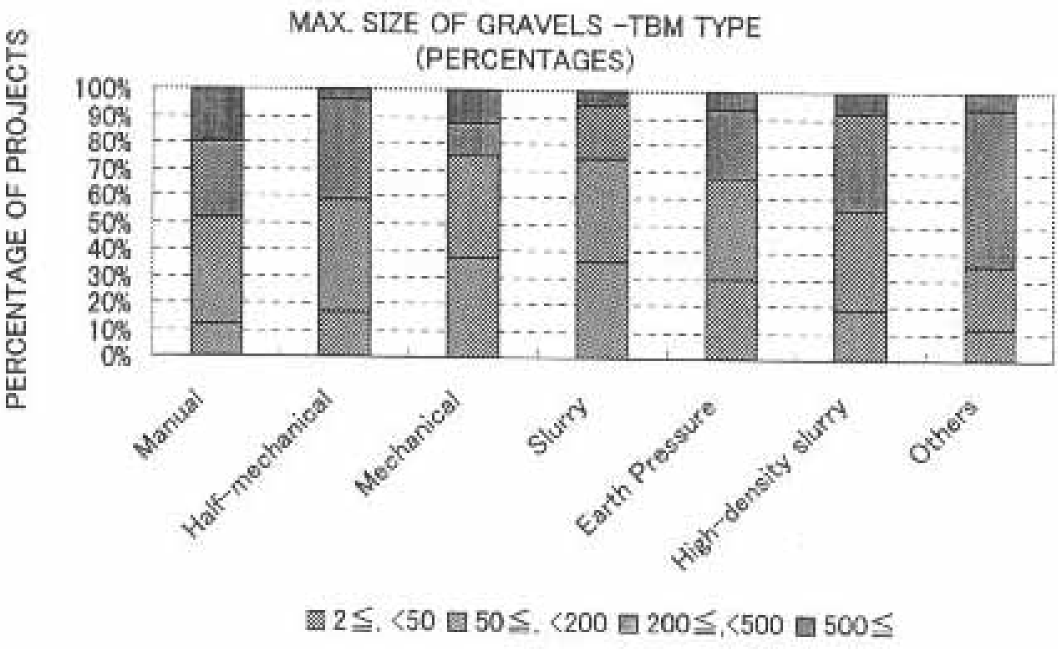


Fig. 4.4 Gravel Size and Type of TBMs

4.3 Size of TBM

4.3.1. Weight

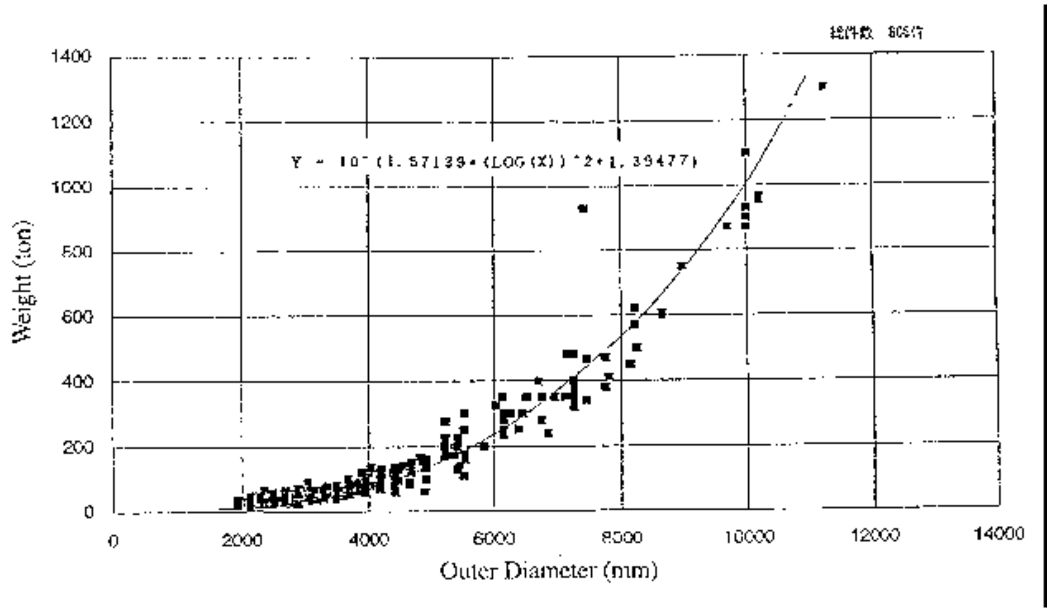


Fig. 4.5 Diameter and weight of TBM (EPB, Slurry)

4.3.2. Length/Diameter (L/D)

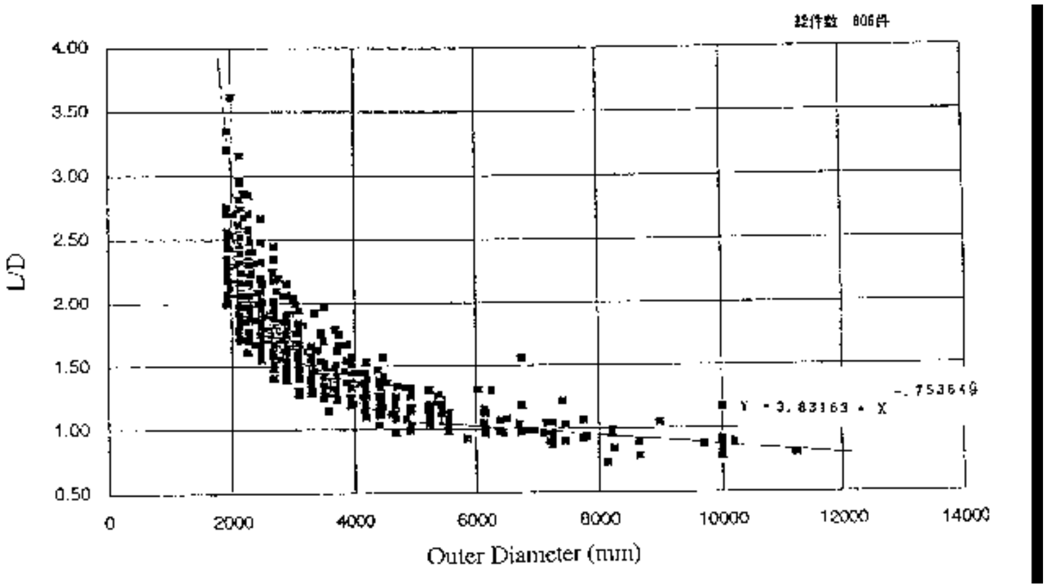


Fig. 4.6 Diameter and length/diameter (L/D) of TBM

APPENDIX: TBM PERFORMANCE IN HARD ROCK

A-1 General

The following prognosis model is a summary of "Project Report 1-94, Hard Rock Tunnel Boring", published by University of Trondheim, The Norwegian University of Science and Technology, NTH Anleggsdrift.

The prognosis model is based on job site studies and statistics from 33 job sites with 230 km of bored tunnels in Norway and other countries. Data have been carefully mapped, systematized and normalized and the presented results are regarded as representative for well organized tunneling. It should be noted that the prognosis model is valid for parameter values in the normal range. Extreme values may, even if they are correct, not fit the model and give incorrect estimates.

The prognosis model has been developed continuously since 1975 and has in the period up till now been through several phases and adjustments in accordance with increased knowledge and improvements of TBMs, auxiliaries and methods. The model is today considered as a practical and useful tool for pre-calculation of time consumption and costs for TBM bored tunnels in hard rock. The model is based on the use of TBM, Open type.

A-2 Advance

A-2.1 Rock Mass Properties

1. DRILLING RATE INDEX, DRI: Index related to the properties of the rock mass. Together with Fracturing, DRI is the rock mass factor that has the major influence on Penetration Rate.
DRI is calculated from two laboratory tests,

- the Brittlenes Value S20
- Sievers J-Value SJ

The two tests give measures for the rock's ability to resist crushing from repeated impacts and for the surface hardness of the rock. Recorded Drilling Rate Indexes for some rock types are shown in Fig. A.1.

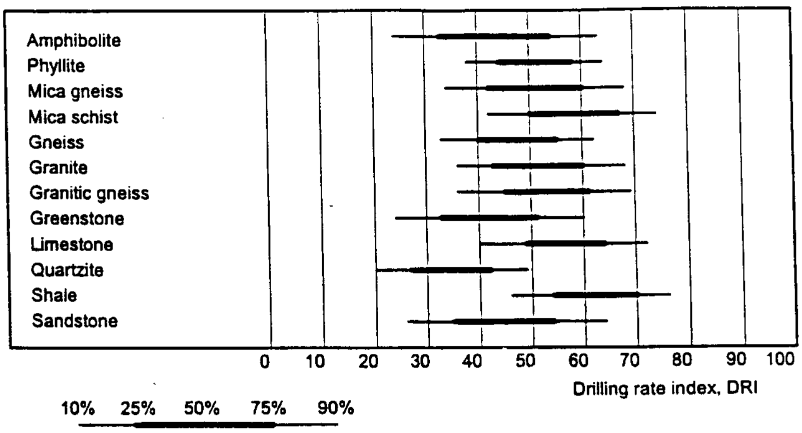


Fig. A.1 Recorded Drilling Rate Index for various rock types

2. CUTTER LIFE INDEX, CLI: Cutter Life Index is calculated on the basis of Sievers J-Value and the Abrassion Value steel. (AVS.) CLI expresses the lifetime in boring hours for cutter rings of steel on TBM. Recorded CLI for some rock types are shown in Fig.A.2

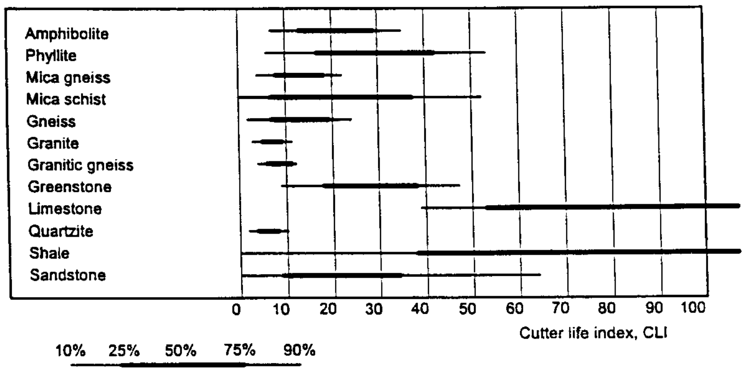


Fig.A.2 Recorded Cutter Life Index for various rock types

3. FRACTURING: The most important penetration parameter for tunnel boring. In this context, fracturing means fissures and joints with little or no shear strength along the planes of weakness. The less the distance between the fractures is, the greater the influence on the penetration rate. Rock mass fracturing is characterized by degree of fracturing (type and spacing) and the angle between the tunnel axis and the planes of weakness.

a) JOINTS in this respect are fractures that can be followed all around the tunnel profile.

b) FISSURES are non-continuous fractures which can be followed only partly around the tunnel profile.

c) FRACTURING is recorded in CLASSES with reference to the distance between the planes of weakness. The classes are shown in Fig. A.3. Recorded fracturing for some rock types are shown in.

Fracture Class (joints Sp/fissures St)	Distance between Planes of Weakness [cm]
0	-
0-I	160
I-	80
I	40
II	20
III	10
IV	5

Fig. A.3 Fracture classes with corresponding distance between planes of weakness

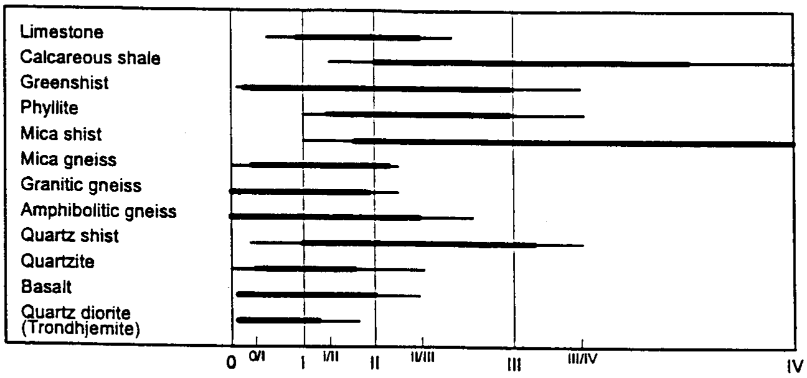


Fig. A.4 Recorded degree of fracturing for various rock types

d) FRACTURING FACTOR, Ks combines the effect of the fracturing class and the angle between tunnel axis and planes of weakness. See Fig.A.5. The factor Ks is used in a formula for calculation of penetration rate.

e) EQUIVALENT FRACTURING FACTOR expresses the rock mass properties as the Fracturing Factor Ks adjusted for DRI-value. See Fig.A.5. $K_{EQV} = Ks \times K_{DRI}$

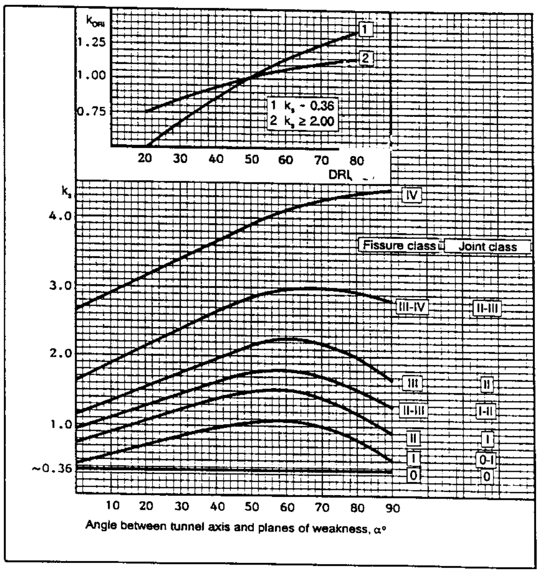


Fig. A.5 Fracturing factor, Correction factor for DRI≠49

A-2.2 Machine Parameters

1. BASIC CUTTER THRUST: (M_B) The gross thrust of the TBM divided by number of cutters, N. Thus, for practical calculating purpose the CUTTER THRUST in this model means the average thrust of all the cutters on the cutter head (kN/cutter). The friction between TBM and rock mass is disregarded. Recommended max. gross average thrust for TBMs with different diameters and cutter diameters are shown in Fig. A.6. For calculation of penetration the cutter diameter and cutter spacing must be taken into account.

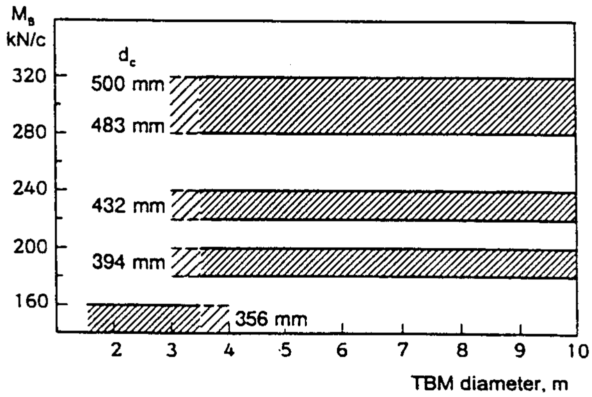


Fig. A.6 Recommended max. gross average per disc

2. CUTTER SPACING: The average distance between the cutter tracks on the face = Diameter of TBM /2N (N= number of cutters)._CUTTER SPACING is normally about 70 mm.

3. CUTTER HEAD R.P.M.: Revolutions per. Minute. Cutter head r.p.m. is inverse proportional to the cutter head diameter. This is in order to limit the rolling velocity of the peripheral cutters. Cutter head r.p.m. as function of TBM diameter is shown in Fig. A.7.

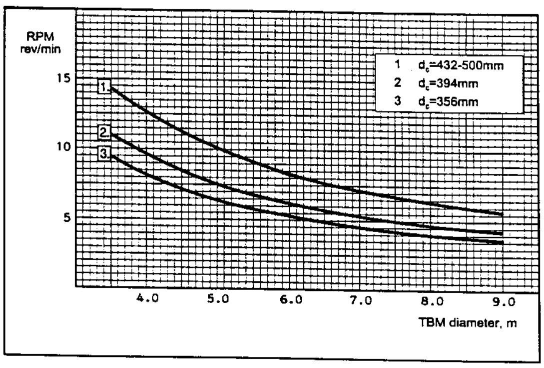


Fig. A.7 Cutterhead r.p.m. as function of TBM-diameter

4. INSTALLED POWER ON CUTTER HEAD: (kW) The rated output of the motors that are installed to give the cutter head its torque. The rolling resistance and thus the torque demand increases with increasing net penetration. The available torque may therefore be the limiting factor when the penetration is high and/or the TBM is boring in very fractured rock. See 3) (3) TORQUE - DEMAND below.

A-2.3 Other Definitions

1. BASIC PENETRATION RATE: Basic Penetration Rate (i) in mm/rev as a function of equivalent thrust and equivalent fracturing factor is shown in Fig. A.8. For cutter diameters and average cutter-spacing different from _=483 mm and 70 mm respectively the equivalent thrust is given by the formula: $M_{EQV} = M_x \times K_{DX} \times K_A$ (kN/cutter)

Fig. A.9 and Fig. A.10 give correction factor Kd for cutter diameters different from 483 mm and factor K_A for cutter spacing.

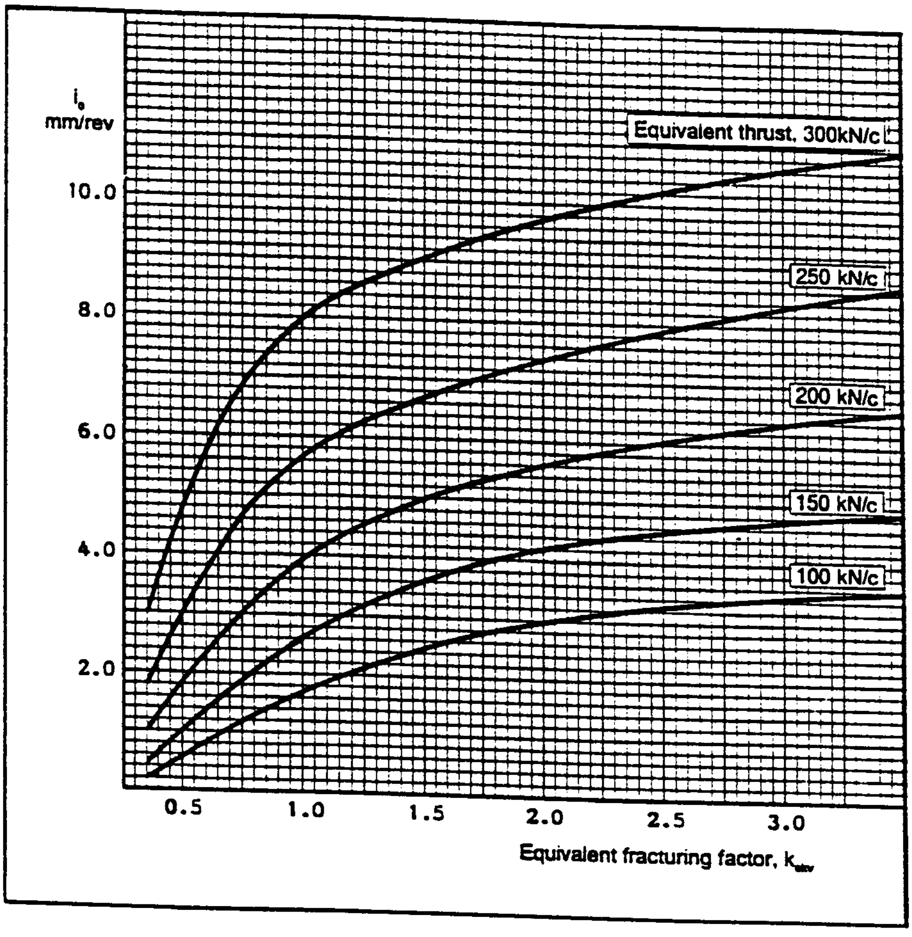


Fig. A.8 Basic penetration for cutter diameter = 48.3 mm and cutter spacing = 70 mm

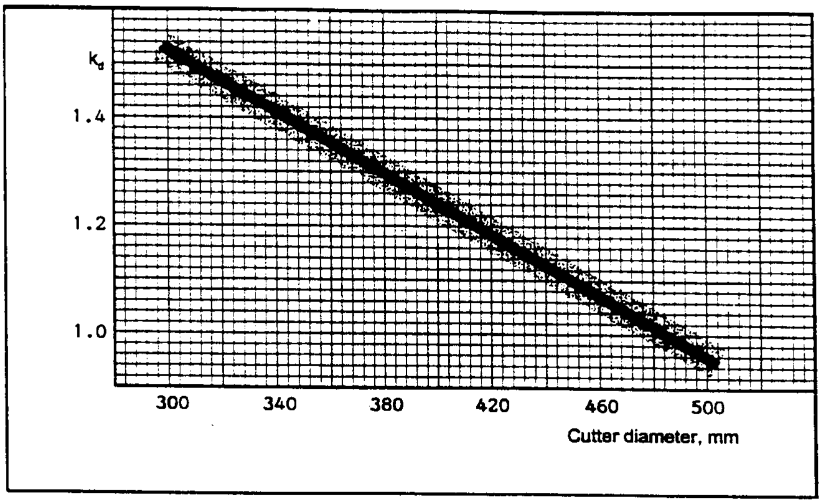


Fig. A.9 Correction factor for cutter diameter≠483 mm

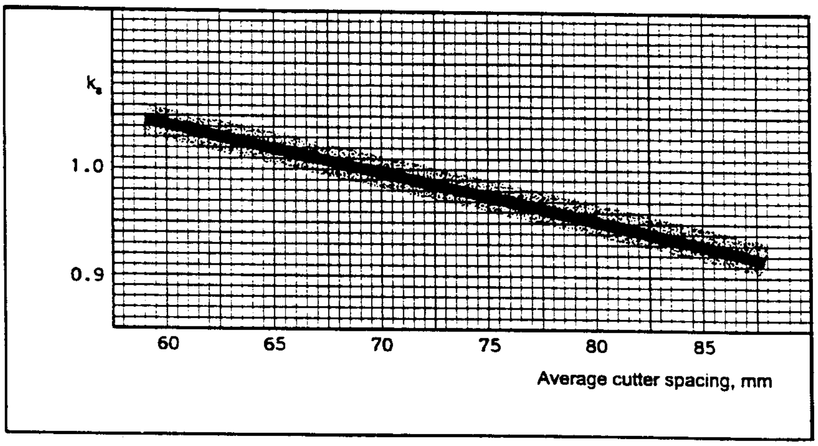


Fig. A.10 Correction factor for average cutter spacing≠70 mm

2. NET PENETRATION RATE: Net penetration rate (I) is a function of basic_penetration and cutter head r.p.m.

I = i x r.p.m. x (60/1000) (m/hr)

3. TORQUE DEMAND: For calculated high net penetration or when the rock is_very_fractured, one must check that the installed power on the cutterhead gives sufficient torque to rotate the cutterhead. If not the thrust must be reduced until the required torque is less than the installed capacity. Necessary torque is given by the following formula:

$T_{REQ.} = 0.59 \times r_{TBM} \times N_{TBM} \times M \times kc$ (kNm)
0.59 = Relative position of the average cutter on the cutterhead.
R_{TBM} = cutterhead radius.
N_{TBM} = number of cutters on the cutterhead.
M = Average thrust pr, cutter.
Kc = cutting coefficient (for rolling resistance) $kc = Cc \times i^{0.5}$
Cc is a function of cutter diameter and is found from Fig. A.11.

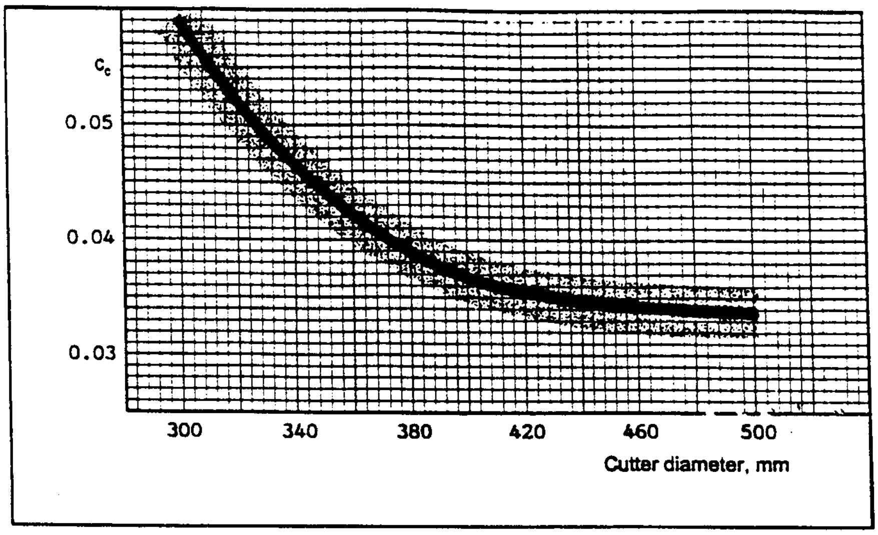


Fig. A.11 Cutting constant Cc as a function of cutter diameter

4. Other Limitations to Advance Rate

Besides limitations due to available torque, the system's muck removal capacity may be a limiting factor, particularly for large diameter machines. When boring through marked single joints or heavy fractured rock, it may be necessary to reduce the thrust due to too high machine vibrations and very high momentary cutter loads.

A-2.4 Gross advance rate

THE GROSS ADVANCE RATE is given in meters per week as an average for a longer period. Gross advance rate depends on net penetration rate, machine utilization and the number of working hours during the week. Machine utilization is net boring time in percent of the total tunneling time. Total tunneling time includes: Boring T_B (Depends on net penetration rate)

- Regripping T_T (Depends on stroke length, normally 1.5-2.0 m. As an average 4-5 minutes.)
- Cutter change and inspection Tc (Depends on cutter ring life and net penetration rate. Time needed for cutter change may vary from 30 to 60 minutes per cutter.)
- Maintenance and service of TBM, T_{TBM}, and back-up equipment T_{BACK} (Time consumption for maintenance and repair depends on net penetration rate as indicated in Fig. A.12.)
- Miscellaneous T_A (Miscellaneous include normal rock support in good rock conditions, waiting for transport, tracks or roadway, surveying or moving of laser, water, ventilation electric cable, cleaning, other things like travel, change of shift etc.) T_A as hours per km is indicated in Fig. A.12.

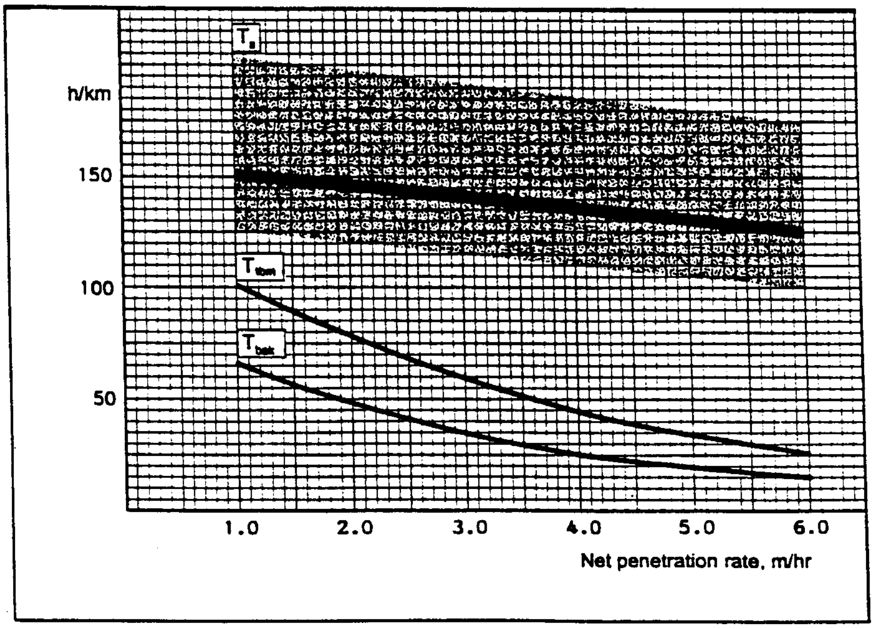


Fig. A.12 Maintenance as function of net penetration rate

A-2.5 Additional Time Consumption

Estimation of time consumption for a tunnel is based on weekly advance rate, estimated on the basis of net penetration rate and total utilization of the TBM. In addition, extra time must be added for

- assembly and disassembly of TBM and back-up equipment in the tunnel
- excavation of niches, branches, dump stations etc.
- rock support in zones of poor quality
- additional time for unexpected rock mass conditions
- permanent rock support and lining work
- downtime due to major machine breakdowns
- dismantling of tracks, ventilation, invert cleanup etc.

Example of application

Geometrical conditions:
Tunnel diameter: f = 4.5 m
Tunnel length: L= 3200 m

Geological conditions:
Type of rock: Mica Schist
Drilling Rate Index: DRI= 60
Degree of fracturing: St II
Angle between tunnel axis and planes of weakness: 45
Fracturing factor. ks = 1.40
Equivalent fracturing factor: k_{EQV} = 1.40 x 1.10 =1.54

Machine parameters:
TBM diameter: f = 4.5 m
Cutter diameter: 483 mm
Gross thrust pr. cutter: Fig. A.6
290 kN/cutter
Cutterhead r.p.m.: Fig. A.7
11.1 rev./min.
Number of cutters: 32
Average cutter spacing: 70 mm
Installed power: 1720 kW

Net penetration rate:
Equivalent thrust: Fig. A.9 and Fig. A.10
M_{EQV} = 290 x 1.00 x 0.975 = 283 kN/cutter
Basic penetration: Fig. A.8 i = 8.40 mm/rev
Net penetration:
8.40 x 11.1 x 60/1000 = 5.59 m/hour

Torque check:
Cutter constant: Fig. A.11

$$Cc = 0.034$$

Cutting coefficient:

$$kc = 0.034 \times 8.400.5 = 0.0985$$

Necessary torque:

$$TREQ = 0.59 \times 2.25 \times 32 \times 290 \times 0.0985 = 1213 \text{ kNm}$$

Necessary power:

$$PN = 1213 \times 2\pi \times 11.1/60 = 1410 \text{ kW}$$

A-3 Cutter Consumption

The cutter ring life depends mainly on the following factors:

1. Rock mass properties:
- CUTTER LIFE INDEX (CLI), see A-2, 1. (2)
- Content of abrasive minerals in the rock

2. Machine parameters:
- Cutter diameter
- Cutter type and quality
- Cutter head diameter and shape
- Cutter head rpm
- Number of cutters

The cutter ring life, in boring hours, is proportional to the Cutter Life Index. (CLI) Fig. A.13 shows the basic cutter ring life as a function of CLI and cutter diameter. Corrections must be made for varying cutterhead r.p.m. Also for TBM diameter as Center- and Gauge Cutters have a shorter lifetime than Face Cutters. (Fig. A.14). Corrections must also be made for number of cutters on TBM (Ntbm) deviating from normal (No). Finally correction must be made to quartz-content.(Fig. A.15)

Average life of cutter rings is thus given the following formulas:
Cutter ring life in h/c: $H_H = (H_0 \times k_{f\_} \times k_0 \times k_{RPM} \times k_N)/N_{TBM}$

Cutter ring life in m/c: $H_m = H_H \times I$ (I = net penetration rate)
Cutter ring life in sm^3: $Hs\,sm^3 = H_H \times I \times p \times d^2_{TBM}/4$

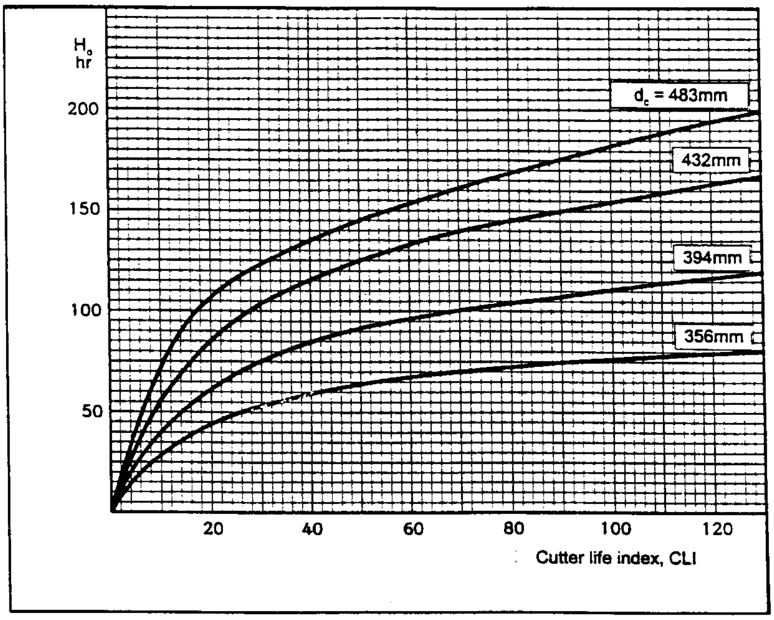


Fig. A.13 Basic cutter ring life as function of CLI and cutter diameter

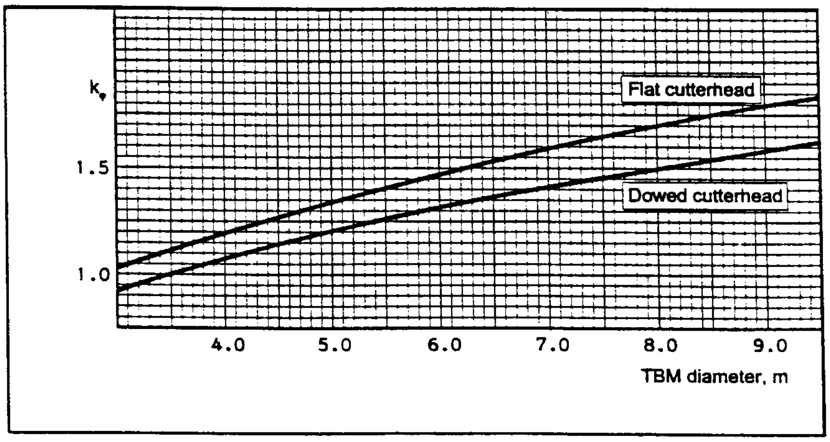


Fig. A.14 Correction Factor for Cutting Ring Life

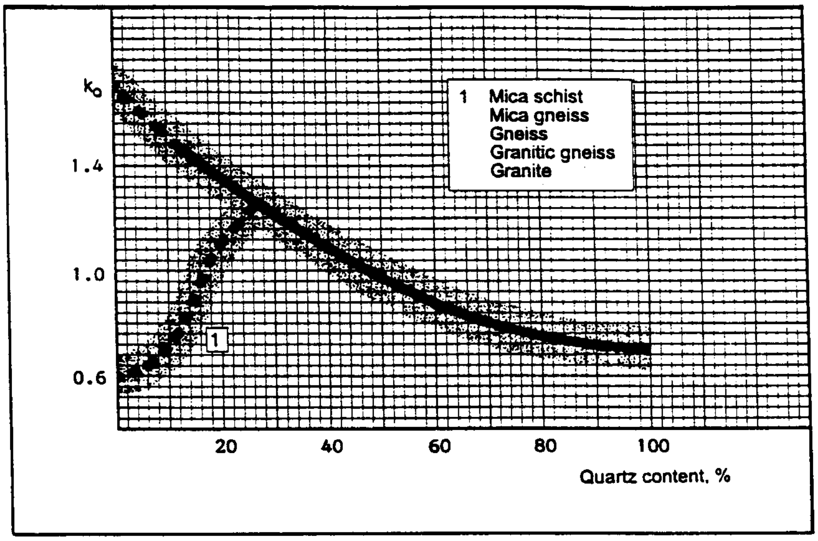


Fig. A.15 Correction Factor for cutting ring vs. Quartz Content

A-4 Troubles and Countermeasures

A-4.1 Causes for Trouble.

<<Trouble>> is caused by unforeseen incidents or conditions that may be difficult to tackle within estimated tunneling time. Trouble in hard rock boring come from:

- geological conditions
- reasons related to the TBM itself and/or from the rest of the machines and installation, - and/or trouble come as a consequence of lack of experience from similar works and general know-how in tunneling.

1. Geological Causes

a) Water Inflow is always a factor one shall have in mind. It counts for everything from occasional appearance of small amounts of water with no practical consequences, to total inundation with free flowing conditions, some times with material outwash and serious tunneling problems. If caught unaware, these problems are capable of completely disrupting tunneling activity and influencing the time schedules drastically. This is serious to conventional tunneling, - it may be even worse to TBM-tunneling with all the sophisticated electrical installations.

Water may come from groundwater, ores, leakage through the overburden from lakes, rivers etc. or even from underground lakes or from Artesian wells. Inflow of salt water may be damaging even in small amounts and calls for special precautions.

It creates a special atmosphere in the tunnel with damaging effect to the electrical equipment and rust and corrosion to the steel construction if not taken care of.

b) Boring in Hard Rock means normally boring in Sound, Solid Rock, and <<open>> TBMs are normally chosen. Nevertheless it is not unusual to meet Faulty Fractured Zones, Unconsolidated Weak Rock, Swelling Ground, Squeezing Ground and very often so called Mixed Faces which means that face consists partly of hard massive rock and partly of fractured rock. Even one single significant fracture may influence the drillability. Consequences to the boring may naturally vary from minor problems to the penetration rate to full stop with TBM stuck in the tunnel.

Swelling Ground very often comes from the influence of special rock materials as so called swelling clay which starts swelling when exposed to humidity. It may cause down-fall and dangerous conditions. Squeezing Ground is found in tunnels in soft rock with large overburdens, and consequently high rock pressure. Rock deformations may in extreme cases lead to total closing of the tunnel.

c) Even if TBM- technology and –know how is steadily improving and thus extending the frame as far as the geology and the geological parameters are concerned, there are still limits. The drillability is a function of a number of rock parameters, out of which fracturing and rock hardness are the most important. It is rare to get into rock which is so hard and so massive that boring is technically impossible with the most powerful TBMs on the market to day, but it may be a challenge to the economy.

If pre-investigations reveal occurrence of rock with the above properties it will be a matter of calculation to find out if the available TBM is able to do the job, and in case, -what will be the advance rate and what will be the cost. The above calculation model should be used carefully in this case since it is based on experiences from rock with not extreme properties, but it will normally be good enough. If the rock shows up unexpected parameters one might be in trouble if the TBM is too weak, and/or the cutters have insufficient quality.

d) High Temperature Ground is found in different parts of the world as for instance in the Alps, in tropical areas and/or when the tunnel goes with extreme overburden as in mines. The temperature is seldom a real obstacle to the boring itself as long as oil is chosen accordingly and cooling water for motors and cutters are available in sufficient quantities, but rather a challenge to the crew.

e) Combustible gases like methane and dust with high content of coal are dangerous and must be taken care of properly.

2. Machine Related Troubles

As is understood from the above a good result with respect to advance rates and tunnel-meter-costs are to a very great extent dependent on the TBM and the supplementary equipment. The geology related conditions in a tunnel are fixed as such. The result of the tunneling with respect to advance rate and tunnel-meter-cost is therefore in fact a question of doing the right choice of TBM and equipment and to be prepared for conditions as they appear.

The right choice of TBM and supplementary equipment is not only a question of looking at machine specifications. It is also a question to which extent it is possible to utilize the same machine parameters. It is an experience from hard rock boring that the TBMs have more power than can actually be utilized because components like for instance cutters in practice are not strong enough.

Breakdowns due to Main Bearing failure or due to failure on other important and expensive components or parts are naturally disastrous to time schedule and costs. (To change a main bearing may take from four to six weeks, provided the bearing is available.) Downtime caused by unskilled operation of the machinery, bad maintenance and repair, waiting for supply of spares, bad ventilation, cut in power supply, waiting for muck-transport, cutter change and cutter inspection should always be encountered and as far as possible be avoided or minimized.

A-4.2 Countermeasures

1. For trouble caused by water inflow there are basically two ways to go.

- To stop the water before it gets into the tunnel by Grouting Ahead of the Face for which purpose boring equipment has to be installed. The equipment should be able to make a 360°funnel with at least 25 m long holes for grouting.
- To take care of the water when it is in the tunnel by Increased Drainage Capacity. What is the best is depending on the amount of water and where it comes from. If the inflow effects change in ground water level and/or pressure grouting may be required. Large inflow of water may cause damage to the machinery, and to the electrical installation in particular, and protection of exposed components may be required.

2. Countermeasures to unusual and unsound ground depends on the actual case

Rock Bolting, Fiber-reinforced Shotcrete alone or together with Rock Bolts, Grouting or in situ Casting with Concrete or may be necessary. The real trouble comes if the equipment is not built for installation of necessary equipment to carry out the rock support in an efficient way. Fractures are discontinuities in the rock mass. The fractures are described by thickness, length, distance between the fractures, roughness in planes of weakness, sort of materials found in the fractures, if they are results of bedding or foliation, and strike and dip if there is a definable pattern. Fractures in the rock mass are an advantage with respect to advance rates as long as rock support is not required.

The above factors are strongly into the picture in the so called Q-method which is a method to define the rock mass quality with respect to stability and the need for rock support. Various classes of rock mass quality which go from exceptionally good to exceptionally poor will require from no support at all, spot bolting, systematic bolting, shotcrete etc. to cast concrete lining.

3. Too Hard Rock is normally a cutter-problem. The cutters are spoiling and/or heavily worn and the penetration rate is reduced. Due to frequent cutter inspections and -changes the utility time goes down and consequently also the Gross Advance Rate.

Great efforts are constantly made to increase the cutter quality. Much is achieved by improving the steel quality in the rings and by increasing the size of the cutters and thus be able to use bigger and better bearings.

To change the size of cutters is theoretical, but normally not a practical solution to meet a section of too hard rock in a tunnel.

If the <<too hard rock>> problem seems to be permanent it is a possibility to call for a special study of the rock in order to improve the cutter-result and/or to call for competing suppliers of cutters.

Cutters with tungsten-carbide inserts are expensive, but may be the solution for a short period. Tungsten-carbide inserted cutters can also be used together with normal cutters in positions that are most exposed, for instance as gauge cutters.

4. High Temperatures in the tunnel

In tropical areas the outdoor temperature may also be very high, at least during daytime. Cooling of the ventilation air will therefore be a necessity in such areas. Together with the cooling effect from the evaporation of the flushing water it is absolutely possible to achieve livable temperatures.

5. Combustible Gases

The TBM itself and supplementary machinery and all electrical installation must be insulated to prevent any explosion to come from sparkles or heating. Gas Detection instruments must be installed.

In dimensioning of the ventilation system the occurrence of gases and dust must be taken into consideration.

In countries where the above are frequent occurrences there are normally very strict regulations with regards to what to do to prevent explosion and also what to do if the accident should happen.

Dust with quartz appears frequently in connection with hard rock boring. It is not combustible, but a serious hazard to the health if breathed for a long period. It is partly a ventilation matter to remove the dust from the working area, and partly.

Germany, Switzerland and Austria

Tunnelvortriebsmaschinen
Tunnel Boring Machines

Empfehlungen zur Auswahl und Bewertung von Tunnelvortriebsmaschinen

Recommendations
for
Selecting and Evaluating Tunnel Boring machines

DAUB

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1. Purpose of the recommendations

The developments in mined tunneling are characterized by an increased trend towards fully mechanized tunneling with appropriate tunneling machines (TBM) in solid rock and soft ground. The creation of special methods such as face supporting with fluid or slurry as well as the successful utilization of cutter discs for removing rock-like intrusions and boulders have led to a considerable expansion of the field of application and to an increase in the economy of these tunneling systems.

The increasing application of tunneling machines and the related continuous improvement of the various extraction techniques had led to types of machines, which have the capacity to penetrate extremely heterogeneous subsoil, that is respectively a mixture of soft ground and solid rock. The clear distinction between tunnel boring machines (TBM) for solid rock and shield machines (SM) for soft ground, which resulted from their conceptional background and the special engineering and extraction technology, has lost its original significance. Past developments and the progress made in practice have produced tunneling machines, in which the typical features of both techniques have been integrated in a single unit. In this way, the possibility has been created to make available tunneling machines suitable for the entire geotechnical spectrum.

The anticipated geotechnical conditions in conjunction with the course of the route and gradient represent the decisive prerequisites for selecting the tunneling method. By comparison of the cross-section needed for the purpose of the tunnel, its length and the geotechnical conditions with the available technology, the most suitable tunneling machine can be devised. These recommendations apply to inter-relationships, which exist between the geotechnical circumstances and process and engineering techniques.

When selecting tunneling machines, the environmental compatibility of the tunneling methods must also be taken into consideration.

These recommendations should also be seen as an additional aid, designed to serve the engineer in arriving at a decision. A project-related analysis is, however, essential and represents the main basis for the approach. These recommendations do not apply, or only to a certain extent for micro tunneling.

2. Geotechnics

The knowledge of the geotechnical conditions is the most important principle for the planning and execution of a tunneling project. The evaluation of general and special maps leads to initial recognition about the geological and hydrogeological conditions and provide pointers for further investigatory measures. By means of suitable preliminary explorations, the nature and features of the subsoil that must be penetrated during the construction of a tunnel can be described. The accuracy of this description depends on the type and extent of these pre-investigations as well as their validity. Extremely variable geological conditions call for more intensive of preliminary surveys.

Conditions which restrict the
pre-investigations lead to a limited validity of a geotechnical report. This must be taken into account when assessing the projected geotechnical conditions. The aim of the geotechnical survey must be to present the geological and hydrogeological conditions required for the tunneling project as comprehensively and lucidly as possible.

The subsoil that has to be penetrated is, by and large, examined by means of:

_ investigatory boreholes and the obtaining of bore samples and cores
_ exploration and sample-taking on the surface
_ dynamic penetration tests, pressure probes
_ mechanical borehole examinations, e.g. borehole expansion tests, pressiometer
_ geophysical investigation methods
_ pump and water injection tests
_ exploratory tunnels

Through these investigations and, above all, through the samples that were taken, characteristic values are obtained or derived through further suitable investigations and corresponding evaluations.

The more comprehensively the preliminary investigations are carried out and the more valid they are the better the basis for selecting the tunneling method and the tunneling

machines.
The essential geotechnical parameters are listed in the following:

solid rock
- compressive strength (rock strength)
- tensile strength, cleavage strength
- shearing strength
- break and bedding planes
- degree of decomposition, degree of weathering
- fault zones
- mineralogy/petrography
- proportions of abrasive minerals
- wearing hardness/hardness
- water-bearing and water pressure (underground water)
- chemical analysis of the water

soft ground
- grain distribution curves
- angle of friction
- cohesion
- deposit thickness
- compressive strength
- shearing strength
- pore volume
- plasticity
- swelling behavior
- permeability
- natural and artificial intrusions and faults
- water-bearing and water pressure (ground water)
- chemical analysis of the water

special features
- primary stress state
- rock burst
- fault zones
- weakening due to leaching processes
- heaving/swelling rock
- subsidence and subsidence chimneys
- karst manifestations
- gases
- rock temperature
- seismic action

More detailed information relating to investigating the subsoil is contained in DIN 4020-Geotechnical Investigations for Construction Purposes. Further pointers are contained in the "Recommendations for Tunneling - Chapter 3: Geotechnical Investigations" , published by the DGGT.
From the cited geotechnical characteristic values and an overall appraisal of the geological and hydrogeological conditions of the subsoil, generally speaking, the following extremely important technical data can be obtained:
- ease of break-out of the subsoil
- stability of the subsoil
- stability of the face
- measures for supporting the face
- nature and extent of the supporting measures
- time lag between breaking-out and securing the subsoil
- deformation behavior of the subsoil
- influence of underground and/or groundwater
- abrasiveness of the subsoil
- stickiness of the excavated soil
- separability of the excavated soil (when using a supporting fluid)
- suitability for reutilization of the excavated soil

Factors, which influence the environment, must also be observed, such as e.g.:
- surface settlements
- interference with and changes to the groundwater conditions
- suitability of the excavated material for landfill
- contamination of the subsoil and groundwater
- health-jeopardizing influences

On the basis of the listed geotechnical characteristic values and constructional data including the environmentally relevant factors, it is possible the select the construction method and to divide the tunnel over its route into tunneling classes, which closely define the tunneling method, identify the performances to be applied per tunneling class and describe the degree of difficulty. Whereas the selection of the construction method is the prerequisite for allocation into tunneling classes (laid down by the client), the choice of the machine should be left open as far as possible and left up to the responsible contractor (choice of the construction company).

3. Construction methods for mined tunnels

3.1. Survey

Different construction methods are available for executing a tunnel by mining. They can be split up into the groups-universal headings, mechanical headings (tunneling machines) and micro-tunnel headings. In this connection, those methods for which the extraction resp, the cutting phase is decisive are allocated to solid rock. In the case of soft ground, on the other hand, the supporting and/or securing of the subsoil is accorded priority(Fig.1).

In conjunction with the special demands placed on a tunnel and taking environmental factors into consideration, a general assessment of the tunneling methods with respect to their suitability in individual cases can be carried out.

The remainder of these recommendations deal exclusively with the process technical features to be considered when using tunneling machines, and essential selection and evaluation criteria for the corresponding geotechnical fields of application.

3.2. Explanation of the Construction Methods

The “shotcreting construction method” is an independent method, whose possibilities or rather principles of supporting the cavity combine with various tunneling methods.

Under the term “tunneling with systematically advancing support”, we understand tunneling method which embrace the systematic and thus not simply the partial application of suitable supporting means, which are applied for the advance stabilization of the face area. These include: the forepoling method, methods with pipe screens, screens comprising injection lances, screens with freezing lances, screens comprising horizontal HPG columns.

Large-area freezing or grouting is methods designed to improve the subsoil, which then facilitate the application of a construction method such as shotcreting. The tunneling classification then relates to the improved subsoil conditions.

Whereas the form and size of the cross-section in the case of the “universal headings” can be as desired and in fact, can alter within a length of tunnel, this flexibility does not exist when tunneling machines are applied.

Generally speaking, tunneling machines in accordance with their function are circular and thus possess a given shape. This restricts their application should the utilization of a circular cross-section not be purposeful or necessary and therefore, increases the costs. Tunneling machines have also been developed which do not drive circular cross-sections.

Tunneling machines are, by and large, geared to their diameter. This applies, first and foremost, to shield machines. In the case of tunneling machines for solid rock, a certain variation of the diameter is possible if a shield body is not required.

Recent developments allow shield machines to be modified for different diameter ranges in a fairly straightforward fashion. In addition, shield machines have been devised which are fitted with two or three overlapping cutting wheels staggered one behind the other. In this way, cross-sections which are not circular can be driven. The installations in question are special forms of shield machines for special purposes.

Apart from these machines being geared to a circular form and diameter, the length of the sections to be driven represents a further important feature especially for the economic application of a tunneling machine.

The profile accuracy of the cavity cross-section is particularly high when tunnel machines are used. During heading, care should be taken to ensure that the predetermined driving tolerances are adhered to. Unscheduled deviations from the axis can,

in contrast to universal headings, by and large only be corrected with considerable difficulty.

Bautechnische Merkmale eines Tunnels / Technical features of a tunnel — Tunnel Bauverfahren / Tunneling method	Merkmale des Tunnels/Features of tunnel: Querschnitts-gr §e / Size of cross-section: gleich-bleibend uniform	ver nder-lich changing	Querschnitts-from / From of cross-section: gleich-bleibend uniform	ver nder-lich changing	Tunnell nge / Length of tunnel: kurz short	lang long	Sicherung Ausbau / Lining: zwei-schaling 2-shells	ein-schaling 1-shell	geford. Profilgenauigkeit / Required profile accuracy: gro§ high	Grund(G)-Schicht(S)-Wasser / Groundwater(G) Underground water(U): ohne Zusatz-ma§n. without extra measures	mit Zusatz-ma§n. with extra measures	Umwelt/Environment: L rm-ersch tterungen / Noise vibrations	Gas-Staub-Entwickl. / Gas dust devel-opment	Schutz d.Personals / Protection of labour force
Universeller Vortrieb Universal excavation Festgestein/Solid rock Sprengvortrieb Drill+blast	X	X	X	X	X	X	X	X	0	SX	GX	q	q	s
Teilschnittmasch.vortrieb Roadheader	X	X	X	X	X	X	X	X	X	SX	GX	s	q	s
Spritzbetonbauw./N T Shotcreting method/NATM	X	X	X	X	X	X	X	X	X	SX	GX	s	s	s
Messervortrieb Forepoling	X	0	X	0	_	X	X	X	X	0	GX	s	s	q
Vortrieb mit systematisch vorelender Sicherung Heading with systematic advance support Lockergestein/soft ground	X	X	X	X	X	X	X	0	0	0	GX	s	s	q
Maschineller Vortrieb Mechanical heading Festgestein/Solid rock Tunnelbohrmasch.-Vortr. TBM driving	X	0	Kreis circle	0	_	X	X	X	X	SX	GX	s	s	s
Schildmasch.-Vortrieb Shield machine driving	X	0	Kreis circle	0	_	X	X	X	X	X	X	s	s	q
Rohrvortrieb Pipe jacking	X	0	Kreis circle	0	X	X	_	X	X	X	X	s	s	q
Vorpre§verfahren Jacking method Lockergestein/soft ground	X	0	X	0	X	X	0	X	X	X	X	s	s	q
Microtunnelvortrieb Micro-tunnelling	X	0	Kreis circle	0	X	0	0	X	X	X	_	s	s	q
Eignung der Bauverfahren: Suitability of the construction method:	X gut geeignet X well suited		0 nicht geeignet 0 not suited		- nicht blich - not usual					Auswirkungen: Effects:		q gro§ q high	s geringer s slight	

1 Bauverfahren f r Tunnel in geschlossener Bauweise

Construction methods for mined tunnels

4. Tunneling machines TM

Tunneling machines (TM) either head the entire tunnel cross-section with a cutter head or cutting wheel full-face or in part segments by means of suitable extraction equipment.
During the excavation phase, the machine is moved forward either continuously or stroke-by-stroke.
A difference is drawn between tunnel boring machines TBM and shield machines SM.
Tunnel boring machines remove the rock at the face, with the support generally being installed afterwards, following up at a distance.
The machines are held in place during the excavation phase by means of grippers pressed laterally against the tunnel walls.
Shield machines generally support the subsoil that is being penetrated and the face by direct means during the excavation phase. The shield is advanced during excavation by jacking against the completed lining.
A systematic compilation of tunneling machines is provided in Fig. 2, which was based on the classification contained in this section.

4.1. Tunnel Boring Machines (TBM)

A distinction is drawn between tunnel boring machines without shield body and those with one(Fig.3).

4.1.1. Tunnel boring machines without shields

Tunnel boring machines are employed in solid rock with medium to high face stability. They do not possess a completely closed shield body. Economic application can be strongly influenced and restricted through high wear costs of the cutting tools.
Generally speaking, only a circular cross-section can be broken out by these machines. A rotating cutter head, which is equipped with roller bits (discs), possibly with tungsten carbide bits, is pressed against the face and removes the rock through its notch effect. In order to provide the contact pressure at the cutter head, the machine is held radially by means of hydraulically moveable grippers. Extraction is gentle on the rock and results in an accurate profile.
The machine occupies a large part of the cross-section. Systematic supporting is normally carried out behind the machine (10 to 15 m and more behind the face). In less stable and particularly in friable rock, it must be ensured that the placing of support arches, lagging plates and anchors, in certain cases, even shotcrete, is possible directly behind the cutter head. It should be possible to carry out preliminary investigations and rock strengthening from the machine.
In the case of bore diameters of > 10 m, so-called expansion machines can also be applied. Starting from a continuous pilot tunnel, the profile is expanded in one or two working phases using correspondingly designed cutter heads.
During excavation at the face, small pieces of rock, accompanied by an amount of dust, are produced. As a consequence, devices for restricting the dust development and dedusting are necessary for these machines:
_ wetting with water at the cutter head
_ dust shield behind the cutter head
_ dust removal with dedusting on the back-up
The material transfer and supplies for the machine call for what, in some cases, can be very long back-up facilities.

4.1.2. Tunnel boring machines with shields TBM-S

In solid rock with low stability or friable rock, tunnel boring machines are equipped with a closed shield body. In this case, it is advisable to carry out supporting within the protection of the shield tail skin (segments, pipes, etc.), against which the machine supports itself. The gripper system is then no longer needed.
Otherwise, the explanations already provided for tunnel boring machines also apply here.

4.2. Shield Machines SM

A distinction is drawn between Shield Machines with full-face extraction (cutter head) SM-V and shield machines with part extraction (milling boom, excavator)SM-T.
Shield machines are employed in loose soils with or without groundwater, in the case of which generally the subsoil surrounding the cavity and the face have to be supported. The characteristic feature of these machines is the type of face support (Fig. 3).

4.2.1. Shield Machines with full-face excavation SM-V

4.2.1.1 SM-V1 Face without support
If the face is stable, e.g. in clayey soils, so-called open shields can be employed. The

cutter head equipped with tools removes the soil; the loosened soil is carried away by means of conveyor belts or scraper chains.

4.2.1.2 SM-V2 Face with mechanical support

Supporting of the face is carried out via an almost closed cutter head. The plates arranged between the spokes are elastically supported; they are pressed up against the face. Extraction is executed full-face via the cutter head equipped with tools; the loosened soil passes through slits, whose opening width is variable, between the spokes and the supporting plates, into the working chamber.

The material is removed via conveyor belts, scraper chains or by hydraulic means.

Scraper disc shields possess a high degree of mechanization. Through the constant full-face contact of the cutting wheel with the face, high torque is required.

In the case of types of soil, which tend to flow, supporting in the vicinity of the slits is incomplete, which can lead to settlements. It is extremely difficult to remove obstacles.

4.2.1.3 SM-V3 Face with compressed air application

If groundwater is present, it has to be held back in the case of machines belonging to types SM-V1 and SM-V2 unless it can be lowered. Either the whole tunnel is subjected to compressed air or the machine is provided with a bulkhead so that only the working chamber is under pressure. Airlocks are essential in both cases.

Particular attention must be paid to the compressed air leakage via the shield tail seal and the lining.

The support which is realized by the application of compressed air acts directly. Through suitable measures, it is also possible to avoid an accumulation of compressed air, e.g. when sand lenses with water under pressure occur.

4.2.1.4 SM-V4 with fluid support

In the case of these machines, the face is supported by a fluid that is under pressure. Depending on the permeability of the subsoil that is present, effective fluids must be used for supporting, whose density and/or viscosity can be varied. Bentonite suspension has proved to be particularly effective.

The working chamber is closed to the tunnel by a bulkhead. The pressure needed for supporting the face can be regulated with great precision either by means of an air cushion or by controlling the speed of the delivery and feed pumps. Supporting pressure calculations are required.

The soil is removed full-face by means of a cutter head equipped with tools. Hydraulic conveyance with subsequent separation is essential.

If it is necessary to enter the working chamber (tool change, repair work, removing obstacles), the fluid must be replaced by compressed air. The supporting fluid (bentonite, polymer) then forms a slightly air-permeable membrane at the face, whose life span is restricted. This membrane facilitates the supporting of the face through compressed air and should be renewed if need be.

When the machine is at a standstill, mechanical supporting of the face is possible by means of segments, which can be shut, in the cutting wheel or through plates that can be extended from the rear. These solutions are advisable on account of the limited duration of the membrane.

Stones or banks of rock can be reduced to a size convenient for conveyance through discs on the cutting wheel and/or stone crushers in the working chamber.

4.2.1.5 SM-V5 Earth pressure balance face

The face is supported by earth slurry, which is formed from the material that has been removed. The shield's working chamber is closed to the tunnel by means of a bulkhead. More or less closed cutting wheels equipped with tools extract the soil. An extraction screw under pressure carries the soil out of the working area.

The pressure is checked by loadcells, which are distributed over the front side of the bulkhead. Mixing vanes on the rear of the cutting wheel and the bulkhead are intended to ensure that the soil obtains a suitable consistency.

The supporting pressure is controlled through the thrust of the rams and the speed of the conveyor screw. The soil material in the screw or additional mechanical installations must ensure a seal in the extraction equipment, as otherwise the supporting pressure in the working chamber cannot be retained due to the uncontrolled escape of water or soil.

Complete supporting of the face, especially in the upper zone, only then succeeds providing the supporting medium soil- can be transformed into a soft to stiff-plastic mass. In this connection, the percentile share of the fine grain smaller than 0.6 mm has a considerable influence.

In order to extend the range of application of shield machines with earth pressure balance support, suitable agents for conditioning the soil material can be applied: bentonite, polymer, foam from polymers. In such cases, the environmental compatibility of the material for landfill purposes must be taken into consideration.

4.2.2. Shield machines with partial axe excavation SM-T

4.2.2.1 SM-T1 Face without support

If the face is perpendicular or stable with a steep slope, it is possible to use this type of shield. The machine merely comprises the shield body and the extraction tool (excavator, milling boom or scarifier). The soil is removed via conveyor belts or scraper conveyors.

4.2.2.2 SM-T2 Face with partial support

The face can be supported by platforms and/or breasting plates.

In the case of platform shields, the front section is divided up by one or a number of platforms on which slope form, which support the face. The soil is removed manually or by mechanical means.

Platform shields possess a low degree of mechanization. Disadvantageous is the danger of major settlements resulting from un-controlled face support.

- Tunnelvortriebs-maschinen TVM *Tunnelling Machines*
  - Tunnelbohrmaschinen TBM *Tunnel Boring Machines*
    - TBM ohne schild TBM *TBM without Shield* — TBM
    - TBM mit Schild TBM-S *TBM with Shield* — TBM-S
  - Schildmaschinen SM *Shielded Machines*
    - Schildmaschinen mit Vollschnittabbau SM-V *Shield Machines with full-face*
      - Ortsbrust ohne Stützung *Face without support* — SM-V1
      - Ortsbrusut mit mechanischer Stützung *Face with mechanical support* — SM-V2
      - Ortsbrust mit Druckluft-Beaufschlagung *Face with compressed air application* — SM-V3
      - Ortsbrust mit Flüssig-Keitsstützung *Face with fluid support* — SM-V4
      - Ortsbrust mit Erddruck-Stützung *Face with earth pressure balance support* — SM-V5
    - Schildmaschinen mit teiltlächigem Abbau SM-T *Shield Machines with part heading*
      - Ortsbrust ohne Stützung *Face without support* — SM-T1
      - Ortsbrust mit Teilstützung *Face with purtial support* — SM-T2
      - Ortsbrust mit Druckluft-Beaufschlagung *Face with compressed air application* — SM-T3
      - Ortsbrust mit Flüssigkeits-Stützung *Face with fluid support* — SM-T4

Sonderformen und Kombinationen siehe Textteil / *Special forms and combinations are provided in the text*

2 Übersicht Tunnelvortriebsmaschinen(TVM).Sonderformen und Kombinationen sind in Text beschrieben
Survey of tunnelling machines(TM). Special forms and combinations are described in the article

In the case of shield machines with breasting plates, the face is supported through breasting plates, which are mounted on hydraulic cylinders. The breasting plates are partially retracted for removing the soil manually or by mechanical means.

A combination of breasting plates and platforms is possible. If supporting of the roof area is sufficient, extensible breasting plates can be used there.

4.2.2.3 SM-T3 Face with compressed air application

If groundwater is present, this must be held in check in the case of machines of the type SM-T1 and SM-T2. The tunnel is then set under compressed air or the machines are provided with a bulkhead. The material is removed hydraulically or dry via a material lock.

4.2.2.4 SM-T4 Face with fluid support

In the case of this shield type, the working chamber is also closed by a bulkhead. It is filled with a fluid, whose pressure is regulated via the speed of the delivery and feed pumps. The soil is removed via a cutter, which, in similar fashion to suction dredgers, also takes away the fluid-soil mixture.

4.3. Adaptable dual purpose shield machines

A large number of tunnels pass through strongly varying subsoil conditions, which can range from rock to loosely bedded soil. As a result, tunneling methods have to be geared to the geotechnical prerequisites and shield machines, which are correspondingly adaptable, employed.

a) Shield machines, in the case of which the extraction method can be changed without modification:

_ earth pressure balance shield SM-V5 _ compressed air shield SM-V3

_ fluid shield SM-V4 _ compressed air shield SM-V3

b) Shield machines, in the case of which the extraction method can be changed through modification. Findings are available with the following combinations:

_ fluid shield SM-V4 _ shield without support SM-V1

_ fluid shield SM-V4 _ earth pressure balance shield SM-V5

_ earth pressure balance shield SM-V5 _ shield without support SM-V1

_ fluid shield SM-V4_ TBM-S

4.4. Special forms

4.4.1. Finger shields

The shield body is split up into fingers, which can be extended individually. The soil is removed via roadheaders, cutting wheels or excavators. An advantage of finger shields is that they deviate from the circular form and e.g. can also excavate horse-shoe profiles. In the latter case, the base is usually open. The forepoling is also used.

4.4.2. Shields with multi-circular cross-sections

These shield types represent the latest state of development for fully mechanized headings. In the case of these machines, the staggered cutting wheels are designed to overlap.

4.4.3. Articulated shields

Practically all-existing shields can be provided with an articulating joint. If the ratio of the shield body length to the shield diameter exceeds the value l, generally a joint is incorporated in order to improve the steability. The arrangement can also be necessary if extremely tight curve radii are to be driven.

4.4.4. Cowl shield

The shield cutting edge is tapered to approximate the natural angle of slope of the soil. When tunneling under compressed air, this means that safety against blowout is enhanced.

4.4.5. Displacement shield

Only suitable for soft-plastic soils. The machine has no extraction tool. It is pressed into the soil, which results in this being partially displaced and partially removed through an aperture in the bulkhead.

4.4.6. Telescopic Shields

In order to arrive at higher rates of advance, telescopic shields have been designed.

Essentially, the objective is to install the lining during the removal of the soil.

4.5. Supporting and lining

As far as the process techniques referred to in these recommendations are concerned, the tunneling machine together with the support and/or lining represent a single unit in terms of process technology.

4.5.1. Tunnel boring machines TBM

Due to the excavation procedure which is gentle on the rock and the advantageous circular form, the extent of the necessary supporting measures is usually less than for example for drill + blast. In less stable rock, the exposed areas have to be supported quickly in order to restrict any disaggregation of the rock and thus retain the rock quality as far as possible.

Should breaks occur in the vicinity of the cutter head, the extent of the necessary supporting measures can increase considerably.

4.5.1.1 Rock bolts

Rock bolts are generally arranged radially in the cross-sectional profile of the tunnel, a rock matrix-oriented set-up enhance the effect of the shear dowels. Installed locally, they prevent the flaking or detaching of rock plates, arranged systematically, they prevent loosening of the exposed tunnel sidewall. Rock bolts are especially suitable for subsequently increasing the lining strength, as they can still be installed at a later stage.

The anchors are installed in the vicinity of the working platform behind the machine or in special cases, directly behind the cutter head.

4.5.1.2 Shotcrete

Shotcrete serves to seal the exposed rock surface either partially or completely (thick-ness 3 to 5 cm) or provide it with a supporting layer (thickness 10 to 25 cm, in exceptional cases, even more). In order to enhance the loadbearing capacity of the shotcrete lining, it is provided with a single-layer (on The rock side) or two-layers (rock and exposed side) of mesh reinforcement. Alternatively, steel fibber shotcrete can be applied. The shotcrete is generally installed in the vicinity of the working platform behind the machine.

4.5.1.3 Support arches

Support arches serve to effectively support the rock directly after the excavation and to protect the working area. As a consequence, they are, first and foremost, applied in friable and unstable, squeezing rock. Rolled steel sections or lattice girders are used as support arches. Support arches are normally installed directly behind the cutter head in sections in the roof zone or as a closed ring.

4.5.2. Tunnel boring machines with shield TBM-S and shield machines SM

In the case of tunnel boring machines with shield or shield machines, the support is installed within the protection of the shield tail. This usually consists of prefabricated segments.

Apart from supporting the surrounding subsoil, it serves in the case of most machines of this type as the abutment for the thrust rams.

The load transfer between the lining and the subsoil is created by grouting the annular void at the shield tail as continuously as possible. This does not apply to lining systems, which are directly pressed against the subsoil.

In general, it must be ascertained whether a lining comprising an inner shell made of reinforced or un-reinforced concrete is needed. Segments and pipes are normally utilised as single skin linings.

4.5.2.1 Concrete and reinforced concrete segments

The customary precast elements are concrete or reinforced concrete segments. Alone the stresses caused by transport and installation makes it necessary for the segments to be reinforced. Segments with steel fibber reinforcement have also been designed in order to strengthen the edges and corners, which cannot be reinforced by rods, through steel fibbers.

4.5.2.2 Cast steel and steel segments

Through the development of casting technology, segments today can be supplied made of cast steel, e.g. with the material designation GGG 50, with low overall thickness, sufficient dimensional accuracy and sufficient elasticity.

In exceptional cases, as e.g. extremely narrow curves and in the vicinity of apertures in the lining, welded steel segments can represent a technical solution for overcoming load con-contortions on the lining.

4.5.2.3 Liner plates

Pre-formed steel plates in the form of liner plates can represent an economic solution as a full surface provisional support in extremely friable rock.

4.5.2.4 Extruded concrete

Extruded concrete is a tunnel lining, which is installed, in a continuous working
process as an unreinforced or steel-fibre reinforced concrete support behind the tunneling machine between the shield tail and a mobile inner form. Thus, the extruded concrete in its fresh state already supports the surrounding rock, also in groundwater. An elastically supported stop-end formwork, which is pushed forwards concrete pressure, assures a constant support pressure in the liquid concrete.

4.5.2.5 Timber lagging

In non-water bearing soil, the primary support can comprise a wooden or reinforced concrete slatted construction, which is installed between steel profiles (ribs and lagging), which is assembled protected by the shield tail. When the shield tail releases the steel ribs, they and, in turn, the lagging, are pressed against the soil using hydraulic jacks. The tunneling machine can be advanced by thrusting against this pre-stressed construction.

4.5.2.6 Pipes

Pipe-jacking represents a special method, in the case of which reinforced concrete or steel pipes are thrust forward from a jacking station to serve as a support and/or final lining.
For certain construction projects, rectangular cross-sections are also employed with the jacking method.

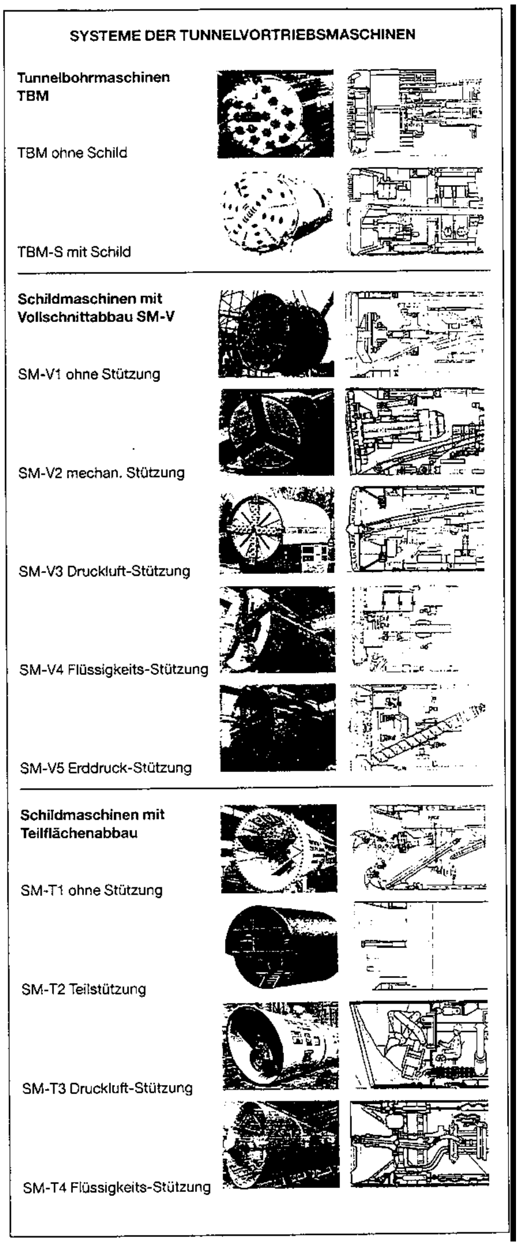
4.5.2.7 Reinforced Concrete

Reinforced concrete is only used n conjunction with blade shields. In the same way as shotcrete, reinforced shotcrete can be applied in conjunction with tunneling machines for supporting purposes when they do not transfer the thrusting forces onto the lining. The reinforced concrete is produced in 2.50 to 4.50 m wide sections protected by so-called trailing blades, which are supported on the last concreted section by conventional means with mobile formwork.

5. Relationship between geotechnics and tunneling machines

5.1. Ranges of application for tunneling machines

The individual tunneling machines are suitable for certain geotechnical and hydro-geographical ranges of application in conjunction with their process-related and technical features.
The specific types of machines are related to their main ranges of application in Fig.4 with geo-technical terms and parameters as the basis. In addition, it is shown there just how far an extension of the range of application is possible should this present itself as a result of simplified methods, in order to increase the economy or with regard to the heterogeneity of the subsoil that is present.
As one of the most essential influencing factors for the application is the lack or presence of groundwater, the fields of application are divided into subsoil with or without groundwater.
Extremely varied extraction tools can be used for removing the subsoil that is present. They are listed in accordance with their suitability for the geotechnical ranges of application and the machine types.
The forms of supporting and lining suit able for the individual machines are presented under 4.5. As a result, they have not been listed separately in a table.



3. Systeme der Tunnelvortriebsmaschinen
Tunneling machine systems

5.2. Important selection and evaluation criteria

TBM

The main range of application is in stable to friable rock, in the case of which underground and fissure water inbursts can be mastered. The uni-axial compressive strength should amount roughly to between 300 and 50 [MN/m^2]. Higher strengths, toughness of the rock and a high proportion of abrasion resistant minerals represent economic limits for - application (abrasiveness according to Cerchar, Schimanek, et al). A restriction of the gripper force of the TBM can also place its application in question.

To assess the rock, the cleavage strength $_z$ ≈25 to 5 [MN/m^2] and the RQD value are required. Given a degree of decomposition of the rock with RQD of 100 to 50 [%] and a fissure spacing of > 0.6 m the application of a TBM appears assured.

Should the decomposition be higher, the stability has to be checked.

TBM-S

The main field of application is in friable to unstable rock, also with inbursts of underground and fissure water. The bonding strength is greatly reduced given possibly the same rock strength in stable rock. This corresponds to a fissure gap of ≈ 0.6 to 0.06 [m] and a RQD value between approx. 50 and 10 [%]. Generally, however, an application of the TBM-S is possible given lower rock compressive strength $_D$ between approx. 50 and 5[MN/m^2] and correspondingly less cleavage strength of $_z$ between approx. 5 and 0.5 [MN/m^2].

SM-V1

This type of machine is mainly used in over-consolidated and thus dry, stable clay soils. In order to make sure that no harmful surface settlements occur even given thin overburdens, the compressive strengths $_D$ of the material should not be less than approx. 1.0 [MN/m^2]. The cohesion cu accordingly registers values above approx. 30 [kN/m^2].

Only in rock which is relatively immune to overbreak can underground and fissure water ingress be coped with.

SM-V2

On account of the full-face supporting cutter head, easily removed, largely dry types of soil can be mastered, first and foremost non-stable cohesive soils or interstratifications comprising cohesive and non-cohesive soils. Major intercalation such as boulders is extremely difficult to cope with.

The cohesion c_u of these soils amounts to between 30 and 5 [kN/m^2]. The grain size is restricted upwards due to the slit width in the cutter head. In order to ensure that surface settlements are kept to a minimum, the slit width and contact pressure have to be optimized.

SM-V3
This machine under compressed air working is mainly used when types SM-V1 and SM-V2 have to operate in groundwater. Its main application must be regarded as in soils with interstratification. The air permeability of the rock and the air consumption and the related blow -out danger are the governing criteria for the application of this type of machine.

SM-V4
Its main range of application is tunneling in non-cohesive types of soil with or without groundwater.
During the excavation process, a fluid under pressure e.g. bentonite suspension supports the face. Layers of gravel and sand are the typical subsoil. Coarse gravel can in certain cases prevent membrane formation. In the event of high permeability, the supporting fluid must be adapted to suit. Major stratification, which cannot be pumped, is reduced in advance crushers. The proportions of ultra-fine grain < 0.02 mm should amount to ≈ 10%. Higher quantities of ultra-fine material can lead to difficulties during separation.

SM-V5
Types of machines with earth pressure balance supporting are especially suitable for with cohesive fractions. In this case, the proportion of ultra-fine grains < 0.06 mm should amount to at least 30 %. In order to produce the desired earth slurry, groundwater has to be present or water must be added. The necessary consistency of the spoil can be improved through the addition of suitable conditioning agents such as bentonite or polymer. In this way too, the danger of sticking is considerably reduced.

SM-T1
This type of machine can be used providing the face is thoroughly stable. Refer also to SM-T1.

SM-T2
This type of machine can be used when the support due to the material lying on the platforms at a natural sloping angle suffices for a conditional control of deformations during tunnel advance. Breastplates can be used for supporting purposes in the roof and platform zone. Slightly to non-cohesive clay-sand soils with a corresponding angle of friction are the main range of application.

SM-T3
The application of this type of machine is given when types SM-T1 and SM-T2 are to be used in groundwater. Either the entire working area, including the excavated tunnel or solely the working chamber is subjected to compressed air.

SM-T4
When clay-sand mixtures are to be removed under water, this type of machine is used. The requirements concerning the ground correspond to type SM-T4. Obstacles can be cleared using the cutting boom. Supporting plates are arranged in the roof zone.

Geo- technische kennwerte / Geotechinical Parameters	Baugrund subsoil	Fels/Festgestein/Hard rock/soil: standfest bis nachbr chig / competent to caving in	nachbr chig bis gebr ch / caving in to unstable	Boden/Lockergestein/Soft rock/soil: bindig standfest / cohesive stable	bindig nicht standfest / cohesive not stable	Wechsellagerung / mixed conditions	nicht bindig / non-cohesive
Gesteinsfestigkeit / Rock Compressive strength	σ D[MN/m²]	300 bis 50	50 bis 5	1,0	0,1		
Zugfestigkeit / Tensile strength	σ z[MN/m²]	25 bis 5	5 bis 0,5				
RQD-Wert / RQD value	RQD[%]	100 bis 50	50 bis 10				
Kluftabstand / Fissure spacing	[m]	>2,0 bis 0,6	0,6 bis 0,06				
Koh sion / Cohesion	Cu[kN/m²]			≥30	30 bis 5	30 bis 5	
Kornverteilung / Grain distribution	<0,02[%] / <0,06[5]			30 / ≥30	30 / ≥30		10
TBM	o.W.						
TBM	m.W.						
TBM-S mit Schild	o.W.						
TBM-S with shield	m.W.						
SM-V1 ohne St zung	o.W.						
SM-V1 without support	m.W.						
SM-V2 mechan.St zung	o.W.						
SM-V2 mech.support	m.W.						
SM-V3 mit Druckluft	o.W.						
SM-V3 with compressed air	m.W.						
SM-V4 Fl ssigkeitsst tzung	o.W.						
SM-V4 fluid support	m.W.						
SM-V5 Erddruck-St tzung	o.W.						
SM-v5 earth pressure balance support	m.W.						
SM-T1 ohne St tzung	o.W.						
SM-T1 without support	m.W.						
SM-T2 Teilst zung	o.W.						
SM-T2 partial su	m.W.						
SM-T3 mit Druckluft	o.W.						
SM-T3 with compressed air	m.W.						
SM-T4 Fl ssigkeitsst tzung	o.W.						
SM-T4 fluid support	m.W.						
Abbauwerkzeug / Extracion tool	V	rollend (Diskenmei§el) rolling (cutter disc)	rollend (Diskenmei§el) rolling (disc bit)	sch lend (Flachmei§el) stripping (flat bit)	sch lend (Flachmei§el) stripping (chisel)	l send/sch lend (Stichel/Flachmei§el) loosening/stripping (cutter/flat bit)	l send (Stichel) loosening (pick)
	T	ritzend (Spitzmei§el) notching (pick)	ritzend (Spitzmei§el) notching (point bit)	ritzend (Spitzmei§el) notching (point bit)	sch lend (Flachmei§el) stripping (flat bit)	sch lend (Flachmei§el) stripping (flat bit)	l send (Stichel) loosening (pick)

o.W.=ohne Grund-bzw.Schichtwasser/without groundwater or underground water — ■ Haupteinsatzbereich/Main field of appl

m.W.=mit Grund-bzw.Schichtwasser/with groundwater or underground water — ▒ Einsatz m glich/application possible

4 Einsatzbereich der Tunnelvortriebsmaschinen
Ranges of application for tunnelling machines

5.3. Pointers for special geotechnical and constructional conditions

Due to special marginal conditions, the application of a certain method and/or a tunneling machine can be considerably restricted. By use of suitable measures, however, an application can be made possible, above all, providing these special conditions only occur locally or over a limited zone. The decisive factor is then the economic feasibility.

Through lowering the groundwater, a simplified technique for the tunneling machines can be applied, which e.g. facilitates the removal of obstacles, should these be expected at very frequent intervals.

In the case of strongly fluctuating geotechnical conditions, the possibility of being in a position to adapt the operating mode of the tunneling machine brings advantages. This is, above all, purposeful when lengthy inter-connected sections are concerned (see 4.3).

When selecting the suitable tunneling machines, a critical evaluation of eventual additional equipment is advisable, which may be required to cope with any deviations from the projected geotechnical conditions within a certain range.

By means of grouting, freezing, vibrator compaction or soil replacement, the subsoil can be improved. This is suitable for the entire tunnel cross-section but most importantly for the area above the tunnel when only thin overburden is present.

Using compressed air when a thin overburden is present, e.g. below a watercourse, ballast or a waterproofing and ballasting layer should be installed.

When fluid support is used, additional measures are required in order to avoid uncontrollable suspension losses given high permeability of the soil and thin overburden.

Should there be a high frequency of coarse gravels and boulders in the sand, the utilization of a rock crusher enhances the operational safety in the case of fluid supported shield machines in addition to equipping the cutter head with cutter discs.

A tunneling machine is only in the position to head a circular cross-section, which has a constant diameter. However, it is technically possible to expand the driven circular cross-section over short stretches subsequently in such a way that other, above all larger cross-sectional forms are created, e.g. for a subterranean station, employing soil improvements should these be called for.

The greater the proportion of ultra fine material in the subsoil, the more attention has to be paid to spoil separation in the case of fluid supported shield machines.

The requirements on the water content and/or the degree of purity of the separated soil material then govern the limits of the economy of the method.

The operational safety of a method is, among other things, dependent on a tunnel's over-burden. This should generally correspond at least to the diameter of the tunnel excavated, if additional measures are to be avoided. This must be accorded special attention in the case of large diameters.

The unrestricted application of certain types of machine is not always assured as the diameter increases and is only possible in conjunction with suitable measures. In the case of tunnel boring machines with large diameters, machines with shield body and systematic placing of segments have proved themselves. As far as earth pressure balance shield machines are concerned, extremely high torques at the cutter head are necessary, which possibly cannot be attained in the case of very large diameters.

As far as earth pressure balance shield machines are concerned, cutter discs can be employed for reducing coarse gravel and boulders. The dimensions of the screw conveyor must be designed in such a fashion the coarse lumps which are present after extraction can be removed. A screw without a shaft is suitable for conveying coarse lumps.

Certain clays or rocks containing clay can cause the cutter head to stick and to form bridges over apertures for removing material. This phenomenon can be counteracted through the proper shape, flushing installations or additives, which reduce the stickiness.

Ingresses of gas require flameproof protection for the tunneling machines or a change of operational mode.

Italy

ITA WORKING GROUP No. 14
(ITA WG14)

“MÉCANISATION DE L’EXCAVATION”
“MECHANIZATION OF EXCAVATION”

Tuteur/Tutor	Animateur	Vice-Animateur
S.KUWAHARA	N.MITSUTA	M.DIETZ

PRÉPARATION DU RAPPORT “RECOMMANDATIONS POUR LE CHOIX DES MACHINES FOREUSES”

PREPARATION OF THE REPORT “GUIDELINES FOR THE SELECTION OF TBM’S”

(Contribution from the Italian Tunneling Association “Mechanized Tunneling”working group - GL14, to the ITA WG14)

World Tunnel Congress‘98
Tunnel and Metropolis
24th ITA Annual Meeting
25 - 30 April 1998
Sao Paulo - Brazil

CONTENTS

1. Aim and Scope (deleted)

2. Classification and Outline of Tunnel Excavation Machines

2.1. Classification of tunnel excavation machines

All over the world there are different classification schemes for tunnel excavation machines (TMs), based on different classification purposes.

The proposed classification scheme represented in fig. l is based on the possibility of dividing TMs on the basis of_

_ ground support system
_ excavation (method and tools)
_ reaction force tool

Following the two machine categories into which all TMs may be grouped, the next paragraphs broadly illustrate all types of TMs.

.1 - General class f cat on scheme for tunnel ng mach ne

- General classification scheme for tunneling machine

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<tr><td colspan="4">1 - General classification scheme for tunneling machine</td><td colspan="2">Excavation</td><td rowspan="3">Excavation</td><td colspan="2">Excavation</td></tr>
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<tr><td>Excavation</td><td rowspan="2">Excavation</td><td rowspan="2">Excavation</td><td>Earth Pressure Balance Shield-EPBS
Special EPBS</td></tr>
<tr><td>Earth Pressure Balance Shield-EPBS Special</td><td>Earth Pressure Balance
Shield-EPBS
Special EPBS</td><td>Earth Pressure Balance Shield-EPBS
Special EPBS</td></tr>
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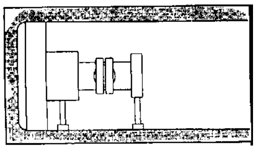
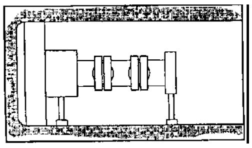
2.2. Rock tunneling machines

2.2.1. Unshielded TBMs

Function principle – A cutterhead, rotating on an axis which coincides with the axis of the tunnel being excavated, is pressed against the excavation face; the cutters (normally disc cutters) penetrate into the rock, pulverizing it locally and creating intense tensile and shear stresses. As the resistance under each disc cutter is overcome, cracks are created which intersect creating chips. Special buckets in the cutterhead allow the debris to be collected and removed to the primary mucking system. The working cycle is discontinuous and includes: 1) excavation for a length equivalent to the effective stroke; 2) regripping; 3) new - excavation.

Main components of the machine - The TBM basically consists of:
the traveling element which basically consists of the rotating cutting head and the primary mucking system
a stationary element which counters the thrust jacks of the cutterhead using one or more pairs of grippers which anchor the TBM against the tunnel walls
_a rear portion containing the driving gear and back-up elements;

Depending on the type of stationary element it is possible to divide unshielded TBMs into: main beam types or kelly types.

Main field of application - Rock masses whose characteristic range from optimal to moderate with medium to high self-supporting time.

2.2.2. Special Unshielded TBMs

2.2.2.1 Reaming Boring Machines - RBMs

Function principle - The Boring Machine is a TM which allows a tunnel made using a TBM (pilot tunnel) to be widened (reaming).

The function principle on which it is based is identical to that for the unshielded TBM; the working stages are also the same as for the unshielded TBM.

Main components of the machine - The RBM basically consists of:
_the traveling element which basically consists of the reaming head, on which the cutting tools are fitted_and the primary mucking system_
_a stationary element located inside the pilot tunnel opposite the reaming head, which counters the thrust jacks on the cutting head using two pairs of grippers;
_a rear portion containing the engines_the driving gear and back-up elements.

A special type of RBM is the Down Reaming Boring Machine_this machine is used for shaft excavation and enables the top-to-bottom reaming of a pilot tunnel dug using a Raise Borer_see below_.

Main field of application - Rock masses whose characteristics range from optimal to moderate with medium to high self-supporting time.

2.2.2.2 Raise Borer

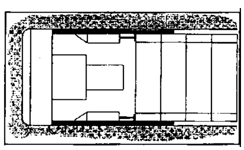
Function principle - The Raise Borer is a machine used for shaft excavation which enables the top-to-bottom reaming of a small diameter pilot tunnel created using a drilling rig.

A cutterhead, rotating on an axis, which coincides, with the axis of the tunnel being excavated, is pulled against the excavation face by a drilling rod guided through the pilot tunnel. The cutters provoke crack formation using the same mechanism illustrated for the unshielded TBMs. Debris falls to the bottom of the shaft where it is collected and removed.

Main components of the machine - The Raise Borer basically consists of 3 parts:
_ the cutterhead (discs or pin discs);
_ the drilling rod which provides torque and pull to the cutterhead;
_ a body, housed outside the shaft, which gives the drilling rod the necessary torque and pull for excavation.

Main field of an application - Rock masses with optimal to poor characteristics.

2.2.3. Single Shielded TBMs: SS-TBMs



Function principle - See the section for unshielded TBMs. In this case the working cycle is also -

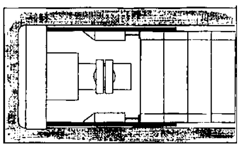
discontinuous and includes:
l) excavation for a length equivalent to the effective stroke; 2) regripping (using the longitudinal thrust jacks braced against the precast segments of the tunnel lining) and simultaneous laying of tunnel lining using precast segments; 3) new excavation.

Main components of the machine

_the cutterhead (discs), which can be connected rigidly to the shield or articulated;
_ the protective shield which is cylindrical or slightly truncated cone-shaped and contains the main components of the machine; the shield may be monolithic (the machine is guided by the thrust system and/- or cutterhead) or articulated (the machine is guided by the thrust system and/or shield articulation);
_ the thrust system which consists of a series of longitudinal/hydraulic jacks placed inside the shield which are braced against the tunnel lining.

Main field of application - Rock masses whose characteristics vary from moderate to poor.

2.2.4. Double Shielded TBMs: DS-TBMs



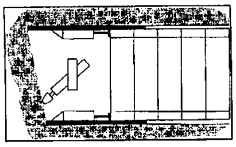
Function principle - Similar to unshielded TBMs, but offers the possibility of a - continuous work cycle owing to the double thrust system, making it more versatile since it can move forward even without laying the tunnel lining of precast - segments.

Main components of the machine:

_ the cutterhead (discs);
_ the protective shield which is cylindrical or slightly truncated cone-shaped and - articulated, and contains the main machine component;
_ the double thrust system which consists of:
1) a series of longitudinal jacks;
2) a series of grippers, positioned inside the front part of the shield which use the tunnel walls to brace against the thrust jacks.

Main field of application - Rock masses whose characteristics range from excellent to poor.

2.3. Soft Ground Tunneling Machines



2.3.1. Open Shields

Function principle - The open shield is a TM in which face excavation is - accomplished using a partial section cutterhead.

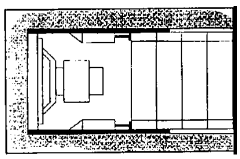
At the base of the excavating head are hand shields and partly mechanized shields in which excavation is accomplished using a roadheader or using a bucket attached to the shield, and using an automatic unloading and mucking system.

Main components of the machine

_ the face excavation system;
_ the protective shield whose shape can be altered to suit the type of section to be excavated (non-obligatory circular section);
_ the thrust system consisting of longitudinal jacks.

Main field of application - Rock masses whose characteristics vary from poor to very bad, - cohesive or self-supporting ground in general. It can also be used in ground, which lacks self-supporting capacity using appropriate preconsolidation or presupport of the - excavation face.

2.3.2. Mechanically Supported Closed Shields



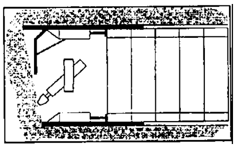
Function principle - This mechanically supported, closed shield is a TBM in which the cutterhead plays the dual role of acting as the cutterhead and supporting the face using mobile plates, integral to the cutterhead, thrust against the face by special hydraulic jacks. The debris is extracted through adjustable openings or buckets and conveyed to the primary mucking system.

Main components of the machine

_ the cutterhead (blades and teeth);
_ the protective cylindrical shield containing all the main components of the machine;
_ longitudinal thrust jacks.

Main field of application - Soft rocks, cohesive or partially cohesive ground, self-supporting ground in general. Absence of groundwater.

2.3.3. Mechanical Supported Open Shields

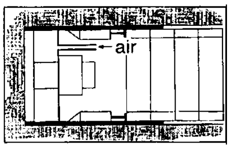


Function principle - Similar to that - described for open - Shields; face stability is achieved using metal plates which thrust alternatively against the face.

Main components of the machine - Similar to those described for open shields; the metal face support plates are located in the upper part of the section and are integral to the shield.

Main field of an application - Soft rocks, cohesive or partially cohesive ground, self-supporting ground in general. Absence of groundwater.

2.3.4. Compressed Air Closed Shields



Function principal - In compressed air closed shields the rotating - cutterhead acts as the means of excavation whereas face support is ensured by compressed air at a sufficient level to balance the - hydrostatic pressure of the ground. Debris is extracted from the pressurized excavation chamber using a ball valve-type rotary hopper and then conveyed to the primary mucking system.

Main components of the machine

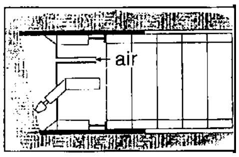
_ the cutterhead (blades and teeth);

_ the protective cylindrical shield containing all the main components of the machine; the front part is closed by a bulk head, which guarantees the separation between the excavation chamber (pressurized), housing the cutterhead, and the zone containing the machine components (unpressurized);

_ longitudinal thrust jacks.

Main field of application - Ground lacking self-supporting capacity and with medium-low permeability ($k \leq 10^{-4}$m/s). Presence of - groundwater. Higher permeability can be locally reduced by injecting bentonite slurry onto the excavation face. The operating limit of the machine is the maximum pressure applicable based on regulations for the use of compressed air in force in different countries.

2.3.5. Compressed Air Open Shields



Function principle - As in the case of open shields, face - excavation is achieved using a roadheader; face support is - provided by compressed air in sufficient quantities to balance the hydrostatic pressure of the ground.

Main components of the machine

_face excavation system (roadheader, excavator);

_ protective shield shaped to fit the type of section to be excavated; the front part, which houses the roadheader, is closed by a bulkhead separating the shield and - excavation chamber (pressurized);

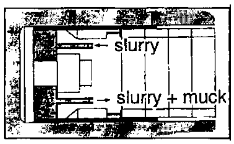
_ longitudinal thrust jacks.

Main field of application - The same as for compressed air closed shields.

2.3.6. Slurry shields

2.3.6.1.Slurry shields-SS

Function principle - The cutterhead acts as the means of - excavation whereas face support is - provided by slurry counterpressure, namely a suspension of bentonite or a clay and water mix (slurry).



This suspension is pumped into the excavation chamber where it reaches the face and penetrates into the ground forming the filter cake, or the impermeable bulkhead (fine ground) or impregnated zone (coarse ground) which guarantees the transfer of - couneterpressure to the excavation face.

Excavated debris by the tools on the rotating cutterhead consists partly of natural soil and partly of the bentonite or clay and water mixture (slurry). This mixture is pumped (hydraulic mucking) from the excavation chamber to a separation plant (which enables the bentonite/clay slurry to be recycled) normally located on the surface.

Main components of the machine

_ cutterhead (discs, blades or teeth);

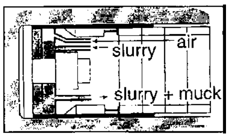
_ protective shield containing all the main components of the machine; the front part is sealed by a bulkhead which guarantees the separation between the shield and the excavation chamber (pressurized) containing the cutterhead;

_ longitudinal thrust jacks;

_ mud and debris separation system (normally located on the surface).

Main field of application - Ground with limited self-supporting capacity. In granulometric - terms, slurry shields are mainly suitable for excavation in sand and gravels with silts. The installation of a crusher in the excavation chamber allows any lumps, which would not pass through the hydraulic mucking system to be crushed. The use of disc cutters enables the machine to excavate in rock. Polymers can be used to excavate ground containing much silt and clay. Presence of groundwater.

2.3.6.2 Hydroshields HS

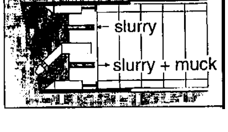


Function principle - Identical to that - described for - uncompensated slurry shields; the only - difference is the way of transferring the counterpressure to the face.

In the closed slurry shield in which the counterpressure is compensated inside the excavation chamber, in addition to the rotating head, there is always a metal buffer which creates a chamber partially filled with air connected to a compressor which can adjust the counterpressure at the face independent of the hydraulic circuit (supply of bentonite slurry and mucking of slurry and natural ground)

Main components of the machine - Similar to those described for closed slurry shields with uncompensated counterpressure.

2.3.7. Open slurry shields



Function principle - It is identical to that described for - compressed air open shields. In this case face support is provided by slurry counterpressure. Depending on the function of the cutterhead used, the following types can be identified:

Thixshield: excavation using a roadheader

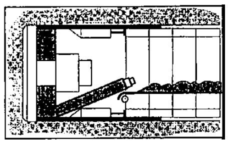
Hydrojetshield: excavation using high-pressure water jets

Main components of the machine – Similar to those described for compressed air open shields.

Main field of application - Similar to that described for the closed slurry shield.

2.3.8. Earth Pressure Balance Shields – EPBS

2.3.8.1 Earth Pressure Balance Shields - EPBS.



Function principle - The cutterhead serves as the means of excavation whereas face support is provided by the - excavated earth which is kept under pressure inside the excavation chamber by the thrust jacks on the shield (which transfer the pressure to the separation bulkhead between the shield and the excavation chamber, and hence to the excavated earth).

Excavation debris is removed from the excavation chamber by a screw conveyor which allows the gradual reduction of pressure.

Main components of the machine

_ cutterhead: rotates with cutting spokes;

_ protective shield similar to that used for closed slurry shields;

_ thrust system: longitudinal jacks which brace against the lining of precast segments.

Main field of application - Ground with limited or no self-supporting capacity. In - granulometric terms, earth pressure balance shields are mainly used for excavating in silts or clays with sand. The use of additives, such as high-density mud or foams, enables excavations in sandy-gravely soil.

2.3.8.2 .Special EPBS

DK shield - Differs from the earth pressure balance shield because of the geometry of the cutterhead whose central cutter projects further than the cutters on the spokes, thus creating a concave cavity.

Double shield (DOT shield) - These are two partially interpenetrated earth pressure balance shields which operate simultaneously on the same plane, creating a “binocular” tunnel.

Flexible Section Shield Tunneling Method -

Earth pressure balance shield in which the excavation system is based on the presence of several rotating cutterheads which enable the construction of non-cylindrical sections.
Elliptical Excavation Face Shield Method - Earth pressure balance shield in which the combined action of a circular cutterhead and additional cutters enables an elliptical section to be excavated.

Triple Circular Face Shield Tunnel - This consists of three shields, operating using earth or slurry pressure balance, which allow large excavation sections to be constructed, such as those required to house an underground railway station.
Vertical Horizontal Continuous Tunnel - This is a slurry pressure balance TM consisting of a main shield, for shaft excavation, which contains a spherical joint housing a secondary shield.
When the main shield has reached the appropriate depth, the spherical joint is rotated 90- and the secondary shield starts tunnel excavation.
Horizontal Sharp Edge Curving Tunnel - Similar to the Vertical-Horizontal Continuous Tunnel, it enables the construction of two tunnels intersecting at right angles.
Double Tube Shield Technology - This is a TM fitted with two concentric shields. The main shield excavates the tunnel with the large section; the secondary shield then excavates the tunnel with the smaller section.

2.3.9. Combined Shields: Mixshield, Polyshield

Function principle - The closed combined shield is a machine, which can be adapted to different excavating conditions mainly by altering the excavation face support system.
The following combinations have already been used:
l) air pressure balance _ _ no pressure balance,
2) slurry pressure balance _ _ no pressure balance,
3) earth pressure balance _ _ no pressure balance,
Main components of the machine
_ rotating cutterhead (rotates with cutter spokes more or less closed);
_ protective shield;
_ thrust system:longitudinal jacks.

Main field of application - The versatility of combined closed shields lends them to be used in rocks and soils under the groundwater table with limited or no self -supporting capacity.

3. Conditions for Tunnel Construction and Selection of TBM Tunneling Method

3.1. Investigations

3.1.1. Introduction

In underground works, construction induces complex and often time-dependent soil-structure interaction. Design must therefore develop both of the basic aspects, which determine the interaction: statics of the excavation and the construction method employed.

The success of a project, in terms of time and costs, strongly depends on the method of excavation employed and the timing of the various construction phases.

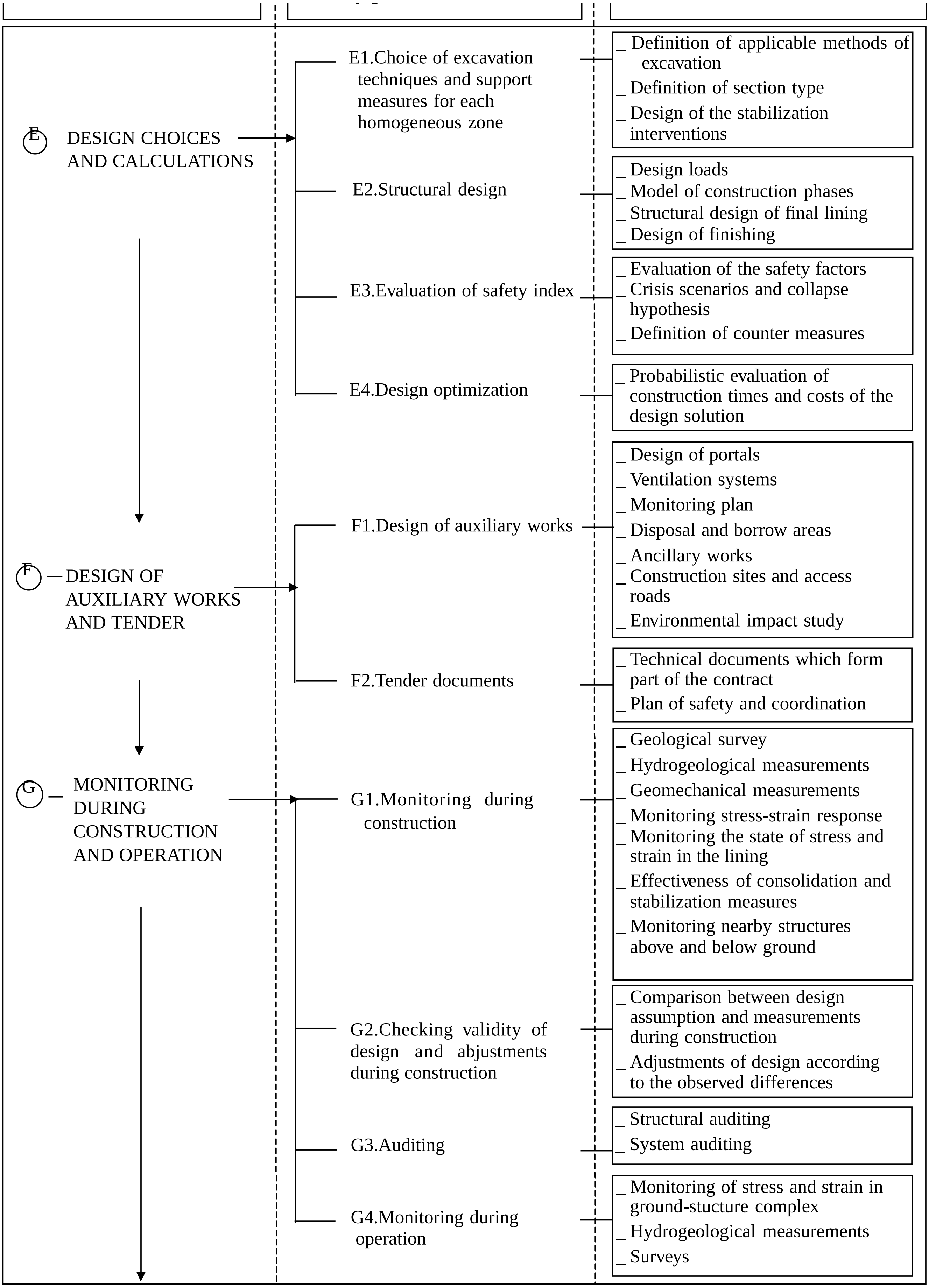
The planning of investigation and tests must take into account these considerations and must be inserted into a well-defined design planning.

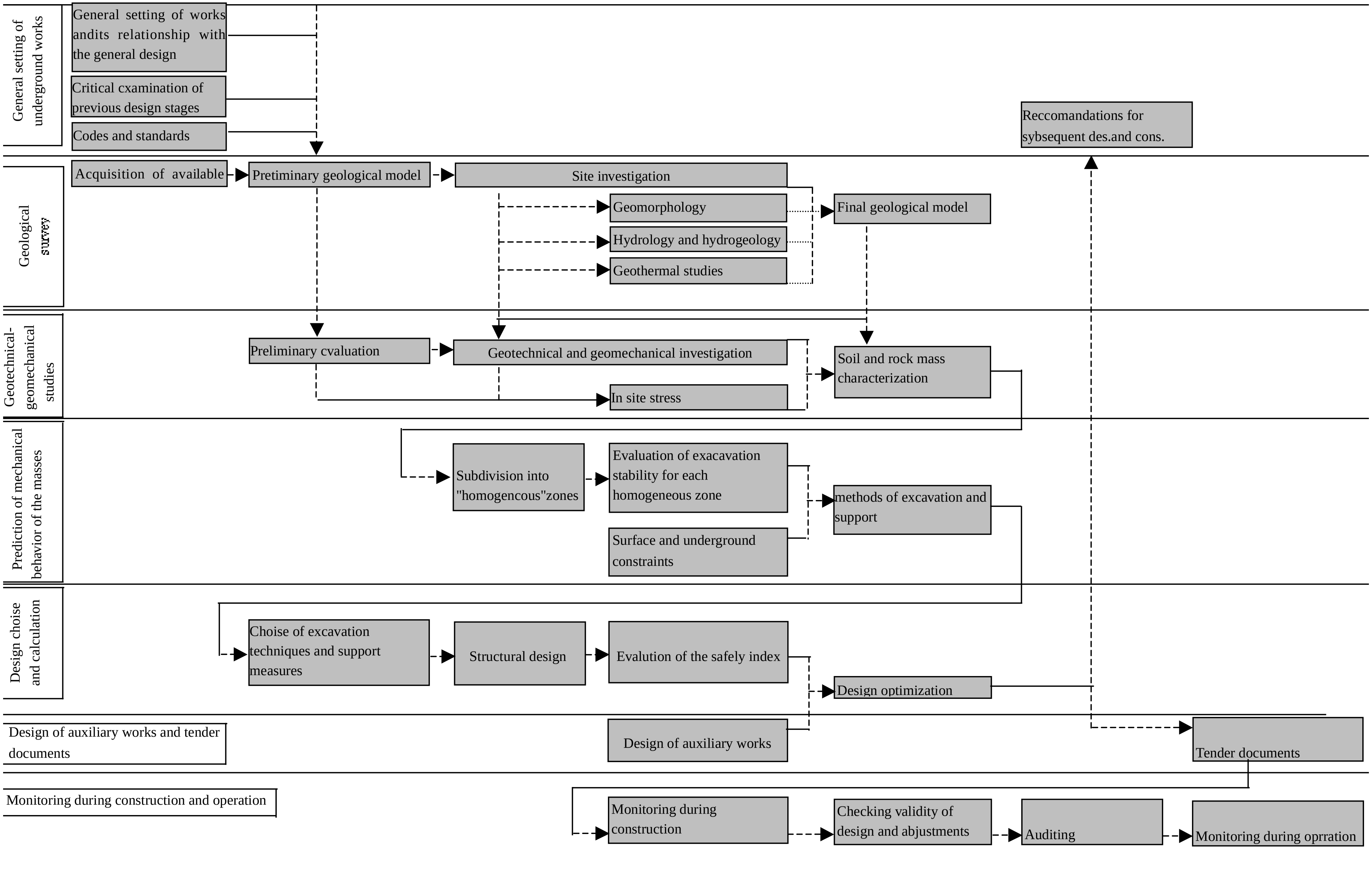
Figure 3.1 shows the schematic structure of the“Guidelines for Design, Tendering and Construction of Underground Works adopted by the main Italian Engineering Associations in relation to tunneling.
These“guidelines”are based on the identification of the“key points”and their organization into“subjects”representing the various successive aspects of the problem to be analyzed and quantified during design/ -tendering/construction. The degree of detail of each“key point”will depend on the -Peculiarities of the specific project and design stage.
In general planning for design, tendering and construction, as illustrated in Figure 3.2, the various“key points”and“subjects”are linked. The relationship between site investigations (Geological Survey and/or Geotechnical -geomechanical studies) and the Preliminary design of excavation and support (Choice of excavation techniques and support measures) is indicated.

Figure 3.1 Schematic structure of the Guidelines for Design, Tendering and Construction of Underground Works

1. Main Themes	2. Key points	3. Subjects
A — GENERAL SETTING OF THE UNDERGROUND WORK	A1.General setting of the works and its relationship with the general design	_ Functional Requirements _ Design Constraints _ Environmental Aspects _ Comparative analysis of alternative routes
	A2.Critical examination of previous design stages	_ Collection and examination of technical documents _ Assessment of the degree of completeness of the design
	A3.Codes and standards	
	A4.Reccommendations for subsequent stages of design and construction	_ General technical judgment of project and recommendations _ Indication of possible design adjustments _ Considerations regarding possible alternatives _ Special conditions for tendering and construction
B — GEOLOGICAL SURVEY	B1.Acquisition of available data	_ Literature review
	B2.Preliminary geological model	_ Structure _ Stratigraphy _ Geomorphology _ Hydrology and Hydrogeology
	B3.Site investigations	_ Evaluation of interaction with geotechnical and geomechanical investigations (C2) _ Planning of site investigations
	B4.Final geological model	_ Structural geological setting _ Meso – structural features _ Lithostratigraohic features _ Mineralogical and petrographic features _ Reliability of the qeoloqical model
	B5.Geomorphology	_ Geomorphological setting _ Interaction between morphogenetic dynamics and designed structures
	B6.Hydrology and hydrogeology	_ General Hydrology and Hydrogeology _ Water Chemistry _ Structure acquifer interaction _ Presence of other fluids
	B7.Geothermal studies	
	B8.Seismicty	_ Seismicity of the area and neotectonic aspects

4. Main Themes	5. Key points	6. Subjects
(C) — GEOTECHNICAL GEOMECHANICAL STUDIES	C1.Preliminary	_ Review of data from geological study _ Review of data from literature
	C2.Geotechnical and geomechanical investigations	_ Evaluation of the relation-ship with site investigation (B3) _ Planning of investigation and tests _ Summary of results _ Additional investigations
	C3.Soil or rock mass characterization	_ Soil and rock-mass structure _ Soil and intact rock characterization _ Mechanical characterization of discontinuities _ Hydraulic properties of soil and rock masses _ Geomechanical classification of rock masses _ Geotechnical and geomechanical models
	C4.Natural state of stress	
(D) — PREDICTION OF MECHANICAL BEHAVIOR OF THE MASSES	D1.Subdivision of the route into"homogeneous"zones	
	D2.Evaluation of the excavation stability conditions for each homogeneous zone	_ Calculation of behavior of face and profile of excavation without support
	D3.Surface and underground constraints	_ Effects of underground excavations on the surface _ Effects of excavation on the surrounding mass _ Effects of tunneling on the existing hydrogeologic equilibrium
	D4.Preliminary design of methods of excavation and support	_ Study of different methods of excavation and suppout:traditional and mechanized _ Choice of general criteria for construction


E DESIGN CHOICES AND CALCULATIONS
E1.Choice of excavation techniques and support measures for each homogeneous zone
_ Definition of applicable methods of excavation
_ Definition of section type
_ Design of the stabilization interventions
E2.Structural design
_ Design loads
_ Model of construction phases
_ Structural design of final lining
_ Design of finishing
E3.Evaluation of safety index
_ Evaluation of the safety factors
_ Crisis scenarios and collapse hypothesis
_ Definition of counter measures
E4.Design optimization
_ Probabilistic evaluation of construction times and costs of the design solution
F DESIGN OF AUXILIARY WORKS AND TENDER
F1.Design of auxiliary works
_ Design of portals
_ Ventilation systems
_ Monitoring plan
_ Disposal and borrow areas
_ Ancillary works
_ Construction sites and access roads
_ Environmental impact study
F2.Tender documents
_ Technical documents which form part of the contract
_ Plan of safety and coordination
G MONITORING DURING CONSTRUCTION AND OPERATION
G1.Monitoring during construction
_ Geological survey
_ Hydrogeological measurements
_ Geomechanical measurements
_ Monitoring stress-strain response
_ Monitoring the state of stress and strain in the lining
_ Effectiveness of consolidation and stabilization measures
_ Monitoring nearby structures above and below ground
G2.Checking validity of design and abjustments during construction
_ Comparison between design assumption and measurements during construction
_ Adjustments of design according to the observed differences
G3.Auditing
_ Structural auditing
_ System auditing
G4.Monitoring during operation
_ Monitoring of stress and strain in ground-stucture complex
_ Hydrogeological measurements
_ Surveys


General setting of underground works
General setting of works andits relationship with the general design
Critical cxamination of previous design stages
Codes and standards
Reccomandations for sybsequent des.and cons.
Geological survey
Acquisition of available
Pretiminary geological model
Site investigation
Geomorphology
Hydrology and hydrogeology
Geothermal studies
Final geological model
Geotechnical-geomechanical studies
Preliminary cvaluation
Geotechnical and geomechanical investigation
In site stress
Soil and rock mass characterization
Prediction of mechanical behavior of the masses
Subdivision into "homogencous"zones
Evaluation of exacavation stability for each homogeneous zone
Surface and underground constraints
methods of excavation and support
Design choise and calculation
Choise of excavation techniques and support measures
Structural design
Evalution of the safely index
Design optimization
Design of auxiliary works and tender documents
Design of auxiliary works
Tender documents
Monitoring during construction and operation
Monitoring during construction
Checking validity of design and abjustments
Auditing
Monitoring during opration

3.1.2. Parameter selection

In line with the general criteria discussed in the previous section the parameters to be investigated for obtaining useful information for mechanized tunnel design and construction
have been divided in two categories:

1. geological parameters;
2. geotechnical - geomechanical parameters.

In the first category, the parameters are common to all tunnel studies and/or design, not restricted to mechanized tunneling (Table 3.1).

The geotechnical - geomechanical parameters specifically to mechanized tunneling are
presented in Table 3.2.

In accordance also with the work carried out by the French Tunneling Association -
(AFTES),
the parameters have been divided in different groups:

l. state of stress,
2. physical,
3. mechanical,
4. hydrogeological,
5. other parameters.

The following information are reported for each group in Table 3.2:
a) the parameter symbol (s)

b) the relationship with TBM excavation, in terms of:
 _ tunnel face and cavity stability
 _ cutting head
 _ cutting tools
 _ mucking system

c) the stage of the work in which the parameter is required, in terms of:

 _ FS feasibility study/preliminary design
 _ DD detailed design
 _ DC construction stage

d) notes related to particular conditions.

Table 3.1: Geological parameters and investigations required for the design of mechanized tunnel excavation

No.	OBJECTIVE OF INVESTIGATION	INVESTIGATION TYPE
	GEOLOGICAL	
1 2	Regional structural setting Mesostructural and lithostratigraphic features	Topography Photogrammetry Photointerpretation Remote sensing Regional geological studies and mapping Detailed geological studies and mapping
3	Type of soils/rocks	Detailed geological studies and mapping Boreholes
4	Soils/rocks structure (fabric, stratification, fracturing)	Detailed geological studies and mapping Boreholes
5 6	Overburden and layers thickness , Degree and depth of weathering	Detailed geological studies and mapping Geophysical methods Boreholes
7	Geological structural discontinuities (faults, shear zones, crushed areas, main joints)	Detailed geological studies and mapping Geophysical methods Boreholes
8	Special formation (salt, gypsum, tale, organic deposits)	Detailed geological studies and mapping , Boreholes
9	Karst phenomena: location of cavities, degree of karstification, age and origin, infilling and karst water	Detailed geological studies and mapping Speleological studies Boreholes Micro-gravimetric survey
	GEOMORPHOLOGY	
10	General geomorphological condition	Topography Photogrammetry Photointerpretation Regional geomorphological studies and mapping Detailed geomorphological studies and mapping
11	Active or potentially active processes	Detailed geomorphological studies and mapping
	HYDROLOGY AND HYDROGEOLOGY	
12 13	Hydrologic condition Groundwater features (swampy ground areas, springs or seepage position, notes on groundwater properties)	Topography Photogrammetry Photointerpretation Regional hydrological studies and mapping Detailed hydrological studies and mapping Detailed geological studies and mapping Detailed hydrogeological studies and mapping
14	No. of groundwater bodies, groundwater levels (and potential groundwater levels)	Detailed hydrogeological studies and mapping Boreholes
15	Soil/rock masses permeability types	Detailed hydrogeological studies and mapping
	GEOTHERMALCONDITIONS	
16	Hydrotermal conditions	Regional geological studies
17	Gas emanations	Regional geological studies and mapping Detailed geological studies and mapping Boreholes
	SEISMICDONDITIONS	
18	Seismicity	Regional geological studies

Table 3.2: Geo-Parameters related to mechanized tunneling

No.	PARAMETER	Symbol	Relationship with TBM excavation				Stage of the work in which the parameter is required			NOTE
			Tunnel face and cavity stability	Excavation: Cutting head	Excavation: Cutting tools	Mucking system	FS	DD	DC	
1	**STATE OF STRESS**									
1.1	Natural stress	$\sigma_1, \sigma_2, \sigma_3,$	R					A	A-O	
1.2	Vertical stress	σ_v	S/R	S/R			A	N		
1.3	Horizontal/Vertical total stress ratio	$Kt\_(\sigma_h/\sigma_v)$	S/R				A	N		
1.4	Horizontal/Vertical effective stress ratio	K_O	S	S/R			A	N	N	
1.5	Consolidation degree (compaction, decompr.)		S				A	NO		
2	**PHYSICAL**									
2.1	Index properties									
2.11	Volumetric weight	$\gamma, \gamma d, \gamma s,$	S/R	S/R		S/R	A	N-O	A	Absolutely necessary in soft ground
2.12	Water content, saturation degree, void ratio	w, S_i, e	S/R	S/R		S/R	A	N-O	A	
2.13	Plasticity index	w_l, w_n	S	S			A	N-O	A	
2.14	Granulometric characteristics		S		S/R	S/R	A	N-O	A	
2.15	Activity			S	S	S	A	N-O	A	
2.16	Mineralogic and petrographic features				S/R		A	N-O	A	
2.2	Global evaluation quality									
2.21	General quality index		S/R	S/R			N-O			
2.22	Alteration index	A_M	R	R	R			N-O	N	
2.23	Quality index	t_O	R	R	R			A-O		
2.3	***Discontinuities***									
2.31	Discontinuities density	RQD, λ	R	R			N-O	N-O	N	
2.32	Number of sets	N_l, N_X	R	R			A	N-O	N	
2.33	Sets characteristics: _ Orientation (dip and dip direction) _ Spacing _ Persistence _ Roughness _ Aperture _ Infilling _ Seepage _ Shear strength _ Genesis (foliation, joint)		R	R			A	N-O	N	

No.	PARAMETER	Symbol	Relationship with TBM excavation				Stage of the work in which the parameter is required			NOTE
			Tunnel face and cavity stability	Excavation		Mucking system				
				Cutting head	Cutting tools		FS	DD	DC	
2.4	Weatherability									
2.41	Sensibility to water, solubility		S/R	S/R		S/R	A	N-O	A	
2.42	Sensibility to hydrometric variations		S/R	S/R		S/R	A	N-O	A	Not frequently used
2.43	Sensibility to thermic variations		S/R	S/R				A	A	
2.5	Water chemistry									
2.51	Chemical characteristics					S/R	A	N-O	A	
2.52	Waste conditions					S/R		N	N	
3	MECHANICAL									
3.1	Strength									
3.11	Shear (short time)	τ	S/R	S/R	S/R		N(S)	N-O	A	
3.12	Uniaxial compressive	s_f, σ_c	S/R	S/R	S/R		N(R)	N-O	A	
3.13	Tensile	σ_t, I_S	R	R	R		A-O	N-O	A	
3.14	Residual		S/R				A	A-O	A	
3.15	General strength index		S/R	S/R					A	
3.2	Global evaluation quality									
3.21	Anisotropy elastic constants		R	R				A	A	
3.22	Isotropic elastic constants	E, υ	S/R	S/R			N-O	N-O	A	
3.23	Visco-behavior	$E_{(t)}$, $E_{(q)}$	S/R				A	N-O	A	
3.24	Swelling		S/R	S/R			N	N-O	A	
3.3	Dynamic characteristics of soil/rock mass									
3.31	P and S waves velocity		S/R				A	N-O	A	
3.32	Deformability modules		S/R				A	N-O	A	
3.33	Liquefaction potential		S				A	N-O	A	
4	**HYDROGEOLOGICAL**									
4.1	Anisotropy permeability	k_X, k_V, k_Z,	S/R					A-O		
4.2	Isotropic permeability	k	S/R				N-O	N-O		
4.3	Piezometric level, hydraulic gradient	H, i	S/R				N	N-O	N-O	
4.4	Water flow	Q	S/R			S/R		A-O	N-O	

No.	PARAMETER	Symbol	Relationship with TBM excavation				Stage of the work in which the parameter is required			NOTE
			Tunnel face and cavity stability	Excavation		Mucking system	FS	DD	DC	
				Cutting head	Cutting tools					
5,1	Abrasivity			S/R	S/R		A-O	N-O		All these parameters are particularly required for mechanized tunneling
5,2	Hardness			R	R		A-O	N-O		
5,3	Drillability				R			N-O		
5,4	Sticky behavior			S/R	S/R	S/R	A-O	N-O	A	
5,5	Ground friction			S/R			A	A-O	A-O	

S=Soil; R=Rock
FS=Feasibility Study/Preliminary Design, DD=Detailed Design; DC=During Construction
N=Necessary; A=Advisable; O=Quantification of this parameter is done through specific tests

Table 3.3: Geo-parameters and related investigations

No.	PARAMETER	INVESTIGATION TYPE	Location S=site L==Lab.	NOTE
1	**STATE OF STRESS**			
1.1	Natural stress	Borehole Slotter stressmeter method (R)	S	In borehole test
		Overcoring methods (R)	S	In borehole test
			S	In borehole test
		Hydraulic Fracturing Technique (R) '	S	Min. 5-6 tests are required, useful also for soft rock
1.2	Vertical stress	Dilatometer (S/R)	S	Experimental method
1.3	Horizontal/Vertical total stress ratio	Aedometer test with lateral pressure control (S)	L	Not frequently used
			L	Not frequently used
		Triaxial test with lateral deformation control (S)	S	Experimental method
			S	
		Dilatometer (S/R)	S	
1.4	Horizontal/Vertical effective stress ratio	Flat jack method (R)	L	
1.5	Consolidation degree (com action, decompr.)	Marchetti dilatometer (S)	L	
		Oedometric test (S)		
		Oedometric test (S)		
2	PHYSICAL			
2.1	Index properties			
1.11	Unit weight	Density tests (S/R)	L	
		Gamma-densimeter (S)	S	
2.12	Water content, saturation degree, void	Laboratory index tests (S/R)	L	
2.13	index	Atterberg limits (S)	L	
2.14	Plasticity index	Grain-size analyses and sedimentation	L	
	Granulometric characteristics	analyses (S)	L	
2.15		Petrographic analyses (R)	L	
2.16	Activity	Mineralogic analyses (S)	L	
	Mineralogic and petrographic features	Mineralogic analyses (S/R)	L	
		Petrographic analyses (S/R)	L	
		Chemical analyses (S/R)		
2.2	Global evaluation of quality			
2.21	Genera quality index	Geophysical methods: .seismic, geoelectric, micro gravimetric, georadar (S/R)	S	
		Borehole perforation parameters: velocity, torque, pressure (S/R)	S	Qualitative evaluation
2.22	Alteration index	Visual examination of the material (R)	S	Outcrops, investigation galleries, borehole samples
		Slake durability test (R)	L	

No.	PARAMETER	INVESTIGATION TYPE	Lo_ation S=site L=Lab	NOTE
2,23	Quality index	Sonic waves test (R)	L	
		Mineralogical analysis (R)	L	
		Petrographic analysis (R)	L	
2,3	**Discontinuities**			
2,31	Discontinuities density	Site measurements (R)	S	Outcrop
		Rock Quality Designation-RQD (R)	S/L	Borehole samples
		Seismic _urvey(R)	S	Qualitative evaluation
2,32	Number of sets	Site measurements + stereographic analyses (R)	S/L	
2,33	Sets characteristics:			
	1. Orientation (dip and dip direction)	1. Site measurement	1. S	
	2. Spacing	2. Site measurement	2. S	
	3. Persistence	3. Site measurement	3. S	
	_. Roughness	4. Site measurement	4. S	
	5. Aperture	5. Site measurement	5. S	
	6. Infilling	6. Site observation and_mineralogical test	6. S	
	7. Seepage	7. Site measurement	7. S	
	8. Shear strength	8. Shear test	8. L	
	9. Genesis (foliation, bedding, joint)	9. Geological observations	9. S	
	Weatherability			
2,41	Sensitivity to water, solubility	Site observation (S/R)	S	
		Mineralogical analyses (S/R)	L	
		Swelling test (R)	L	etc.)
		Cyclic tests (wet-dry) (R)	L	
		Solubility test (R)	L	
2,42	Sensibility to hygrometric variations	Mineralogic analyses (S/R)	L	
		Site observation (S/R)	S	
2,43	Sensibility to thermic variations	Heating test (S/R)	L	
		Freezing test (S/R)	L	
	Water chemistr			
2,51	Chemical characteristics	Chemical analyses: salt content, aggressivity, hardness, pH value, temperature, etc.	L	
2,52	waste conditions	Chemical analyses	L	
3	**MECHANICAL**			
3,1	Strength			
3,11	Shear (short time)	Casagrande shear box test, undrained conditions (S)	S/L	
		Direct Shear test (R)	L	
		In situ direct shear test (R)	S	
		Triaxial test (S/R)	S/L	Soils : lower limit values ; rocks : upper limit values
		Scissometer / Vane test (S)	S/L	In clayed borehole samples

No.	PARAMETER	INVESTIGATION TYPE	Location S=site L=Lab	NOTE
3,12	Uniaxial compressive	Uniaxial compression test (S/R)	L	
		Point load test (R)	S/L	Indirect measurement of the parameter
3,13	Tensile	Direct tensile test (R)	L	
		Brazilian test (R)	L	Indirect measurement of the parameter
		Point load test (R)	S/L	_ndirect measurement of the parameter
3,14	Residual	Residual strength test (shear, triaxial tests)	S/L	
3,15	General strength index	Borehole perforation parameter measurements: velocity, torque, pressure (S/R)		
3.,2	Deformability			
3,21	Isotropic/Anisotropic el_stic constants	Plate loading test (R/S)	S	Rock surface test
		Directional dilatometer test (S/R)	S	Inside borehole tests
		Uniaxial -Triaxial compressive tests on directional samples (S/R)	L	
		P-S wave measurement (_/R)	S/L	Qualitative measurement
		Deformation measurement (S/R): _. Convergence/dilatancy 2. Extensometer 3. Inclinome_ers 4. Settlements	S	Rock surface test (i.e. experimental-gallery)
3,22	Visco-behavior	Flat jack method (R)	S	Rock surface test
		Long-time plate loading test (R)	S	Rock surface test
		Creep load test (S)	L	
		Cycle dilatometer test (S/R)	S	Inside borehole tests
		_eformation measurements (S/R)	S	Rock surface test
3,23	Swelling	Swelling test (S/R)	L/S	
3,3	Dynamic characteristics of soil/rock mass			
3,31	P and S waves velocity	Seismic survey: cross-hole, down-hole	S	Inside borehole tes_s
3,32	Deformability modules	Seismic survey: cross-hole, down-hole	S	Inside borehole tests
3,33	Liquefaction potential	Standard penetration test	S	Inside borehole tests
4	**HYDROGEOLOGICAL**			
4,1	Anisotropic permeability	Observation during borehole drilling	S	Qualitative determination
		Permeability tests: 1. Lefranc 2. Lugeon 3. Directional, constant or variable water level	S	Inside borehole teests

No.	PARAMETER	INVESTIGATION TYPE	Location S=site L=Lab	NOTE
4.2	Isotropic permeability	Pumping test	S	
		Injection test	S	
4.3	Piezometric level, hydraulic gradient	Piezometer (open type)	S	
		Piezometer (close type)	S	
4.4	Water flow	Tunnel measurements	S	
		Springs measurements	S	
	OTHER PARAMETERS			
5.1	Abrasivity	Abrasivity ISRM	L	
		Cerchar test (R)	L	
		Abrasivity (Norwegian Institure of Technology)	L	
		LCPT test (S/R)	L	
5.2	Hardness	Hardness ISRM (R)	L	
		Schmith hammer (R)	L	
		LCPT test	L	
		Knoop	S/L	
		Cone Indenter test (NCB)	L	
		Punch test (Colorado Shool of Mines)	L	
		Drop test (Norwegian Institute of Technology)	L	
		Los Angeles test (S/R)	L	
5.3	Drillability	Siever's test	L	
		Drillability tests	L	
5.4	Sticky behavior	Mineralogical analyses (S/R)	L	Indirect measurement, related with physical index properties
		Atterberg limits (S)	L	

In Table 3.3 an international standard is given for each investigation or test related to mechanized tunneling.

Table 3.3: Investigations, test methods and references

METHODS	REFERENCE
SITE INVESTIGATION	
Topography,aerotopography,photogrammetry photointerpretation	ISRM 1975
Engineering geological investigations	ISRM 1975
Geophysical methods:	
_ micro-gravity	
_ seismic	ASTM D4428-84
_ geoelectric	
_ georadar	
Drilling,borehore cameras and television	
Trenches,shafts and galleries	
SITE TEST	
Plate loading test (R)	ISRM11 / ISRM19 / ASTM D4394-84 / ASTM D4395-84
Overcoring methods (R)	
Flat jack method (R)	ASTM D4623-86
Hydraulic fracturing method (R)	ASTM D4729-87
Compression Test (R)	ASTM D4645-87
Direct shear test (R)	ASTM D4555-90
Dilatometer (S/R)	
Standard penetration test: SPT (S)	ASTM D4971-89 / ASTM D4506 -90
Cone penetration test: CPT (S)	ASTM D1586-84 / ASTM D4633 - 86
Rock Quality Designation (RQD)	ASTM D3441-86
Vane Shear Test (S)	
Discontinuities (R)	ASTM D2573-72
Deformation measurements (S/R):	ISRM07 / ISRM14 / ASTM D4554 - 90
1. Convergence/dilatancy	
2. Extensometer	
3. Inclinometers	
4. Settlements	
Long-time plate bearing test (R)	
Creep test (S/R)	
Cycle dilatometer test (S/R)	ASTM D4553-90
Deformation measurements (S/R)	
Permeability tests:	
1. Lefranc	ASTM D2434-68
2. Lugeon	
3. Directional, constant or variable, water level	
Pumping test	
Injection test	
Piezometer (open type)	
Piezometer (close type)	ASTM D4750-87
Tunnel measurements	
LABORATORY - SOIL	
Identification tests : _ Volumetric weights (natural, dry, satured) _ natural water content, saturation degree _ porosity, void ratio _ Atterberg limits _ Activity (clay)	ASTM D4318-84 / ASTM D4254-83 / ASTM D3282-88 ASTM D2487-90 / ASTM D4404-84 / ASTM D4959-89 / ASTM D854-83 ASTM D427-83 / ASTM D2210-90

METHODS	**REFERENCE**
Grain-size analyses and sedimentation analyses	ASTM D422-63 / ASTM D2487-90 / ASTM D1140-54
Gamma-densimeter	
Mineralogic analyses (diffractometer)	ISRM 1977 / ASTM D4452-85
Chemical analyses	
Permeability	ASTM D2438-90
<u>Oedometric test</u>:	ASTM D4186-90 / ASTM D2435-90 / ISRM 89
_ Natural consolidation	ASTM D2435-90 / ASTM D2166-85
_ compressibility characteristics (consolidation index, edometric compressibility index)	
_ permeability	
_ swelling pressure/ swelling index	
swelling test (Huder-Amberg)	
<u>Shear test:</u>	ASTM D3080-90 / ASTM D2435-90
_ total coesion	
_ total frictional angle	
<u>Triaxial test:</u>	ASTM D4767-88 / BS1377 / ASTM D2850-87
_ drained coesion	
_ drained frictional angle	
_ undrained coesion	
_ total coesion	
_ total frictional angle	
LABORATORY - ROCK	
<u>Index laboratory tests</u>:	ISRM09
_ Density	
_ natural water content	
_ porosity	
Slake durability test	ISRM09 / ASTM D4644-87
Petrographic analyses	ISRM01
Mineralogic analyses	
Chemical analyses	
Uniaxial compression test (R)	ISRM08 / ASTM D3148-86 / ASTM D2938-86
Point load test (R)	ISRM16 / ISRM25
Triaxial test (R)	ISRM02 / ISRM13 / ASTM D2664-86 ASTM D4767 / BS1377 / AGI 1994 ISRM05 / ASTM D2936-84 / ASTM D3967-86
Direct Shear test (R)	ISRM24
Direct tensile test (R)	
Brazilian test (R)	
Creep	ASTM D4341-84 / ASTM D4406-84 / ASTM D4405-84
Sonic waves test (R)	ASTM D2845-90 / ISRM03
Swelling test (R)	ISRM09
Cyclic tests (wet-dry) (R)	
Solubility test (R)	
Thermal expansion test (S/R)	ASTM D4611-86
Frozing test (S/R)	
Abrasivity	
_ Abrasivity	ISRM04
_ Cerchar test (CAI index)	West 1989
_ Abrasivity test (AV – AVS)	NIT 1990
Hardness	
_ Hardness	ISRM04
_ Schmith hammer	ISRM 1977
_ Knoop	
_ Cone Indenter (INCB)	National Coal Board, UK, 1964
_ Punch test	
_ Los Angeles test (S/R)	

METHODS	REFERENCE
Drillability • Sievers test • Drillability test • Resistance to crushing : Drop test	 NIT 1990 NIT 1990
LABORATORY - WATER Chemical analyses: • salt content / organic materials • aggressivity_ • hardness • pH value • temperature	 Standard Methods for examination of wa waters. American Public Health Associa

3.1.3. Monitoring during construction

The deterministic design of a tunnel is based on judgment in selecting the most probable values within the ranges of possible values of engineering properties. As construction - progresses the geotechnical - geomechanical conditions are observed, work performance is monitored and the design judgments can be evaluated or, if necessary, updated. Thus, engineering observations during tunnel works are often an integral part of the design process, and geotechnical - geomechanical - instrumentation is a tool, which assists with these observations.

From a general point of view, the scope of the monitoring scheme is to:

A. control the stability and stress - strain conditions of the structures in the new underground construction;

B. control the stability and stress-strain conditions of the existing structures which potentially interfere with the new construction; and

C. control ground movement around the new underground constructions;

D. monitor environmental aspects.

The design of the general monitoring scheme comprises the following activities:

1. identification of the significant parameters which need to be monitored in consideration of:
   - _ construction geometries and materials;
   - _ stability of existing structures (surface and/or underground) and their potential interference with the new construction;
   - _ geotechnical – geomechanical parameters of the ground and their range of variation;
   - _ geo-structural calculations and structural analysis; and
   - _ construction sequence.

2. definition of the adequate types of instruments;

3. specification of the caution and alarm values for each parameter to be monitored;

4. definition of the counter – measures in case that caution and/or alarm leveis are exceeded.

The different investigation/monitoring possibilities are
- _ from ground surface, or from underground
- _ before, during and after excavation.

In the following subsections we will only examine the underground investigation/- monitoring systems specifically related to TBM tunneling (investigation before - excavation and monitoring during excavation from underground).

3.1.4. TBM tunneling monitoring system

Monitoring systems are used to study the stress-strain behavior of the surrounding ground and lining during and after - construction.

The use of a TBM for the construction of a tunnel does not permit continuous, direct observation of the ground being excavated. Therefore, all the necessary geological- geomechanical information required both during the construction phase for evaluating the ground conditions ahead of the excavation face and subsequently for the purpose of documentation when the work is completed, are normally obtained using indirect methods.

Usually the studies on the interaction between the soil/rock mass and the TBM aim to characterize the quality of the ground mass, above all, to assess its borability.

However, the current problem is an inverse one: given that there is no question that the ground can be excavated, efforts must be focused on the characterization itself through analysis and elaboration of all construction parameters that could possibly be recorded.

Through precise and objective documentation of what the TBM encounters during excavation it is possible to derive the principal - characteristics of the soil/rock mass because variations in TBM behavior are usually correlated with changes in the geotechnical-geomechanical situations.

It is important to underline right from the outset that the prerequisites for making a correct evaluation of the ground mass using all the construction parameters that can possibly be recorded may be summed up as follows:
- _ the use of a TBM fitted with appropriate instrumentation.
- _in this approach skilled engineering geologists with experience should be employed to collect and interpret all the relevant data.
- _ The data collected should be stored in a dynamic database so that multiple-parameter correlation can be not only established but also continuously updated in quasi-real time, as well as offering the possibility to carry out ground conditions extrapolation and forecasting.

From a tunnel excavated by TBM it is possible to investigate the ground ahead of the tunnel face using the methods listed it Table 3.4.

The TBM monitoring systems which can be used to collect data during tunnel construction are listed in Table 3.5.

Table 3.4: Types of soil/rock mass investigations used ahead TBM face

INVESTIGATION TYPE	NOTE
Direct investigation	
Boreholes with core recovery	Horizontal boreholes are normally performed through the TBM cutting head; inclined boreholes are normally possible immediately behind the cutting head in open TBM, through the shield in shielded TBM. Radial boreholes are possible in all TBM types through the lining. The objective of boreholes is to: _ determine the lithological nature of the ground to be excavated through by the TBM. _ determine the presence of water _ determine thepresence of voids (karst) and/or decompressed zones; The drilling is realized with a rig positioned behind the TBM cutting head. In the case of shielded TBMs, it is also possible to utilize a“preventer”system to avoid the ingress of groundwater to the tunnel during execution of the drilling. Horizontal and/or inclined boreholes with core-recovery is not commonly used because the time and drilling diameter required.
Boreholes without core recovery	The method of no-core-recovery with registration of the following drilling parameters using a data-logger. _ drilling rate (**VA**, m/h) ; _ pressure on drill bit (**PO**,bar) ; _ pressure of the drilling fluid (**PI,** bar) ; _ torque (**CR**, bar) ; It is possible to use either a drilling hammer or a tricone bit. The diameter of the drill hole may be limited to 75mm, whereas the drilling rods may be of the aluminum type in order to reduce potential problems associated with the advance of the TBM later in the case that the drilling rods might be completely lost in the drill hole.
Geostructural mapping of the face and/or of the sidewalls	The mapping must be performed using the same methodologies adopted for the face mapping in tunnels excavated by conventional methods. This type of investigation can be performed only when the TBM stops excavation and thus it can be executed at more or less regular intervals in function of the various construction needs. The mapping involves the collection of all geological, structural and geomechanical data of the soil/rock mass. The purpose of this kind of investigation is: _ direct characterization and classification of the soil/rock mass; _calibration of all construction parameters which may permit indirect characterization of the rock mass.
Indirect investigation	
Georadar (in borehole)	
Other borehole logs	Gamma ray log Neutron logs Geoelectric logs
Seismic methods	Tunnel Seismic Prediction method (TSP) Soft Ground Sonic Probing System (SSP)

Table 3.5: TBM monitoring systems

Category		Parameter	UdM	TBMtype
Excavation	Cutting head	Power	kW	All TBM
		Torque	Knm	
		Thrust	KN	
		Rotation speed	RPM	
		Penetration rate	mm/s	
	Cutting tools	Consumption	-	
		Wedge position	mm	
Support in the excavation chamber		Air pressure	kPa	Closed slurry shield (hydroshield) Compressed air close shield
		Air discharge	m^3/h	
		Slurry pressure	kPa	Closed slurry shield
		Slurry level	mm	
		Earth pressure	kPa	Earth pressure balance shield
Mucking	Amount	Slurry discharge	m^3/h	Closed slurry shield
		Slurry density	kg/dm^3	
		Discharge	m^3/h	
		Density	kg/dm^3	
		Weight	kN	Unshielded, single-double shielded TBM, mec. supported, comp. air, closed slurry and EPB shields
		Amount	m^3	
	Charact.	Petrographic characteristics	-	All TBM (in slurry shield or EPB shield TBM is not required)
		Grain-size distribution	-	
		mechanical parameters	-	
Other parameters		Shield position (x, y, z)	m	All TBM
		Gripper thrust	kN	Open TBM and some double shielded TBM
		Gripper stroke	mm	
		Jack thrust	kN	Single-double shielded TBM, mec. suppoted, comp. air, closed slurry and EPB shields TBM
		Jacks stroke	mm	
		Injection (through the shield) pressure	kPa	Closed slurry and EPB shield
		Injection (through the shield) amount	m^3	
		Concrete injection pressure	kPa	Shielded TBM with extruded concrete lining system
		Concrete injection amount	m^3	
Construction data	Performance	Excavation cycle (min. - med. - max.)	h	All TBM
		Advance rate per shift/day/week/month	m	
		Lining rings per shift/day/week/month	N°	
	TBM stops	Planned (holidays, tools change, other ordinary maintenance)	h	
		Due to machine problem (mechanical, electrical, etc.)	h	
		Due to unpredicted rock mass behavior, (water inflow, tunnel face and /or cavity instabilities, squeezing ground, karst, etc.)	h	
		Due to lining problems	h	
		Diverse (back-up problems, others)	h	
		Due to mucking system problems (slurry circuit, screw conveyor, belt conveyor, muck cars, etc.)	h	

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NEW RECOMMENDATIONS ON

CHOOSING MECHANIZED TUNNELLING TECHNIQUES

A.F.T.E.S. will be pleased to receive any suggestions concerning these recommendations

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ACKNOWLEDGEMENTS ARE DUE TO THE FOLLOWING FOR CHECKING THIS DOCUMENT:
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INTRODUCTION

The first recommendations on mechanized tunnelling techniques issued in 1986 essentially concerned hard-rock machines.

The shape of the French market has changed a great deal since then. The development of the hydropower sector which was first a pioneer, then a big user of mechanized tunnelling methods has peaked and is now declining. In its place, tunnels now concern a range of generally urban works, i.e. sewers, metros, road and rail tunnels.

Since most of France's large urban centres are built on the flat, and often on rivers, the predominant tunnelling technique has also switched from hard rock to loose or soft ground, often below the water table.

To meet these new requirements, France has picked up on trends from the east (Germany and Japan).

Faced with France's extremely varied geology, project owners, contractors, engineers, and suppliers have adapted these foreign techniques to their new conditions at astonishing speed.

Now, this new French technical culture is being exported throughout the world (Germany, Egypt, United Kingdom, Australia, China, Italy, Spain, Venezuela, Denmark, Singapore, etc.).

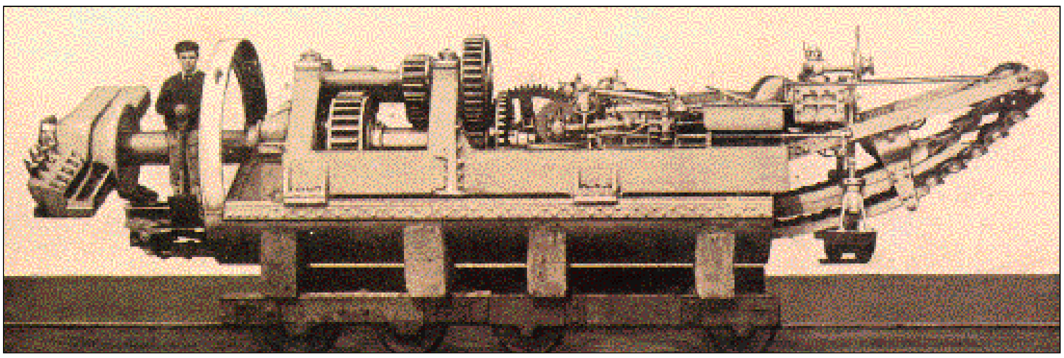
This experience forms the basis for these recommendations, drawn up by a group of 25 professionals representing the different bodies involved.

Before the large number of parameters and selection criteria, this group soon realized that it was not possible to draw up an analytical method for choosing the most appropriate mechanized tunnelling method, but rather that they could provide a document which:

1) clarifies the different techniques, describing and classifying them in different groups and categories,

2) analyzes the effect of the selection criteria (geological, project, environmental aspects, etc.),

3) highlights the special features of each technique and indicates its standard scope of application, together with the possible accompanying measures.In other words, these new recommendations do not provide ready-made answers, but guide the reader towards a reasoned choice based on a combination of technical factors.



Beaumont machine, 1882. First attempt to drive a tunnel beneath the English Channel.

CONTENTS

1 - PURPOSE OF THESE RECOMMENDATIONS

These recommendations supersede the previous version which was issued in 1986 and which dealt essentially with hard-rock or "main-beam" tunnel boring machines (TBMs).

The scope of this revised version has been broadened to include all (or nearly all) types of tunnelling machines.

The recommendations are intended to serve as a technical guide for the difficult and often irreversible choice of a tunnel boring machine consistent with the expected geological and hydrogeological conditions, the environment, and the type of the tunnel project.

To start with, the different kinds of machines are classified by group, category, and type. Since all the machines share the common characteristic of excavating tunnels mechanically, the first criterion for classification is naturally the machine's ability to provide immediate support to the excavation.

This is followed by a list of the parameters which should be analyzed in the selection process, then by details of the extent to which these parameters affect mechanized tunnelling techniques, and finally a series of fundamental comments on the different kinds of machine.

By combining these parameters, decision-makers will arrive at the optimum choice.

The principal specific features of the different groups and categories of techniques are then outlined, and the fundamental fields of application of each category are explained.

Lastly, accompanying techniques, which are often common to several techniques and vital for proper operation of the machine, are listed and detailed. It should be noted that data logging techniques have meant remarkable progress has been made in technical analysis of the problems that can be encountered.

Since health and safety are of constant concern in underground works, a special chapter is devoted to the matter.

2 - MECHANIZED TUNNELLING TECHNIQUES

2.1 - DEFINITION AND LIMITS

For the purposes of these recommendations, "mechanized tunnelling techniques" (as opposed to the so-called "conventional" techniques) are all the tunnelling techniques in which excavation is performed mechanically by means of teeth, picks, or discs. The recommendations therefore cover all (or nearly all) categories of tunnelling machines, ranging from the simplest (backhoe digger) to the most complicated (confinement-type shield TBM).

The mechanized shaft sinking techniques that are sometimes derived from tunnelling techniques are not discussed here.

For drawing up tunnelling machine supply contracts, contractors should refer to the recommendations of AFTES WG 17, "Pratiques contractuelles dans les travaux souterrains ; contrat de fourniture d'un tunnelier" (Contract practice for underground works; tunnelling machine supply contract) (TOS No. 150 November / December 1998).

2.2 - BASIC FUNCTIONS

2.2.1 - Excavation

Excavation is the primary function of all these techniques.

The two basic mechanized excavation techniques are:

• Partial-face excavation

• Full-face excavation

With partial-face excavation, the excavation equipment covers the whole sectional area of the tunnel in a succession of sweeps across the face.

With full-face excavation, a cutterhead - generally rotary - excavates the entire sectional area of the tunnel in a single operation.

2.2.2 - Support and opposition to hydrostatic pressure

Tunnel support follows excavation in the hierarchy of classification.

"Support" here means the immediate support provided directly by the machine (where applicable).

A distinction is made between the techniques providing support only for the tunnel walls, roof, and invert (peripheral support) and those which also support the tunnel face (peripheral and frontal support).

There are two types of support: passive and active. Passive or "open-face" support reacts passively against decompression of the surrounding ground. Active or "confinement-pressure" support provides active support of the excavation.

Permanent support is sometimes a direct and integral part of the mechanized tunnelling process (segmental lining for instance). This aspect has been examined in other AFTES recommendations and is not discussed further here.

Recent evolution of mechanized tunnelling techniques now enables tunnels to be driven in unstable, permeable, and water-bearing ground without improving the ground beforehand. de ceux-ci.This calls for constant opposition to the hydrostatic pressure and potential water inflow. Only confinement-pressure techniques meet this requirement.

2.2.3 - Mucking out

Mucking out of spoil from the tunnel itself is not discussed in these recommendations. However, it should be recalled that mucking out can be substantially affected by the tunnelling technique adopted. Inversely, the constraints associated with mucking operations or spoil treatment sometimes affect the choice of tunnelling techniques.

The basic mucking-out techniques are:

• haulage by dump truck or similar

• haulage by train

• hydraulic conveyance system

• pumping (less frequent)

• belt conveyors

2.3 - MAIN RISKS AND ADVANTAGES OF MECHANIZED TUNNELLING TECHNIQUES

The advantages of mechanized tunnelling are multiple. They are chiefly:

• enhanced health and safety conditions for the workforce,

• industrialization of the tunnelling process, with ensuing reductions in costs and lead-times,

• the possibility some techniques provide of crossing complex geological and hydrogeological conditions safely and economically,

• the good quality of the finished product (surrounding ground less altered, precast concrete lining segments, etc.)

However, there are still risks associated with mechanized tunnelling, for the choice of technique is often irreversible and it is often impossible to change from the technique first applied, or only at the cost of immense upheaval to the design and / or the economics of the project.

Detailed analysis of the conditions under which the project is to be carried out should substantially reduce this risk, something for which these recommendations will be of great help. The experience and technical skills of tunnelling machine operators are also an important factor in the reduction of risks.

3 - CLASSIFICATION OF MECHANIZED TUNNELLING TECHNIQUES

It was felt to be vital to have an official classification of mechanized tunnelling techniques in order to harmonize the terminology applied to the most common methods.

The following table presents this classification. The corresponding definitions are given in Chapter 4.

The table breaks the classification down into groups of machines (e.g. boom-type unit) on the basis of a preliminary division into types of immediate support (none, peripheral, peripheral and frontal) provided by the tunnelling technique.

To give more details on the different techniques, the groups are further broken down into categories and types.

4 - DEFINITION OF THE DIFFERENT MECHANIZED TUNNELLING TECHNIQUES CLASSIFIED IN CHAPTER 3

4.1 - MACHINES NOT PROVIDING IMMEDIATE SUPPORT

4.1.1 - General

Machines not providing immediate support are necessarily those working in ground not requiring immediate and continuous tunnel support.

4.1.2 - Boom-type tunnelling machine

Boom-type units (sometimes called “tunnel heading machines”) are machines with a selective excavation arm fitted with a tool of some sort. They work the face in a series of sweeps of the arm. Consequently the faces they excavate can be both varied and variable. The penetration force of the tools is resisted solely by the weight of the machineLa réaction à.

This group of machines is fitted with one of three types of tool:

- Backhoe, ripper, or hydraulic impact breaker
- In-line cutterhead (roadheader)
- Transverse cutterhead (roadheader)

AFTES data sheets: No. 8 – 14 (photo 4.1.2)

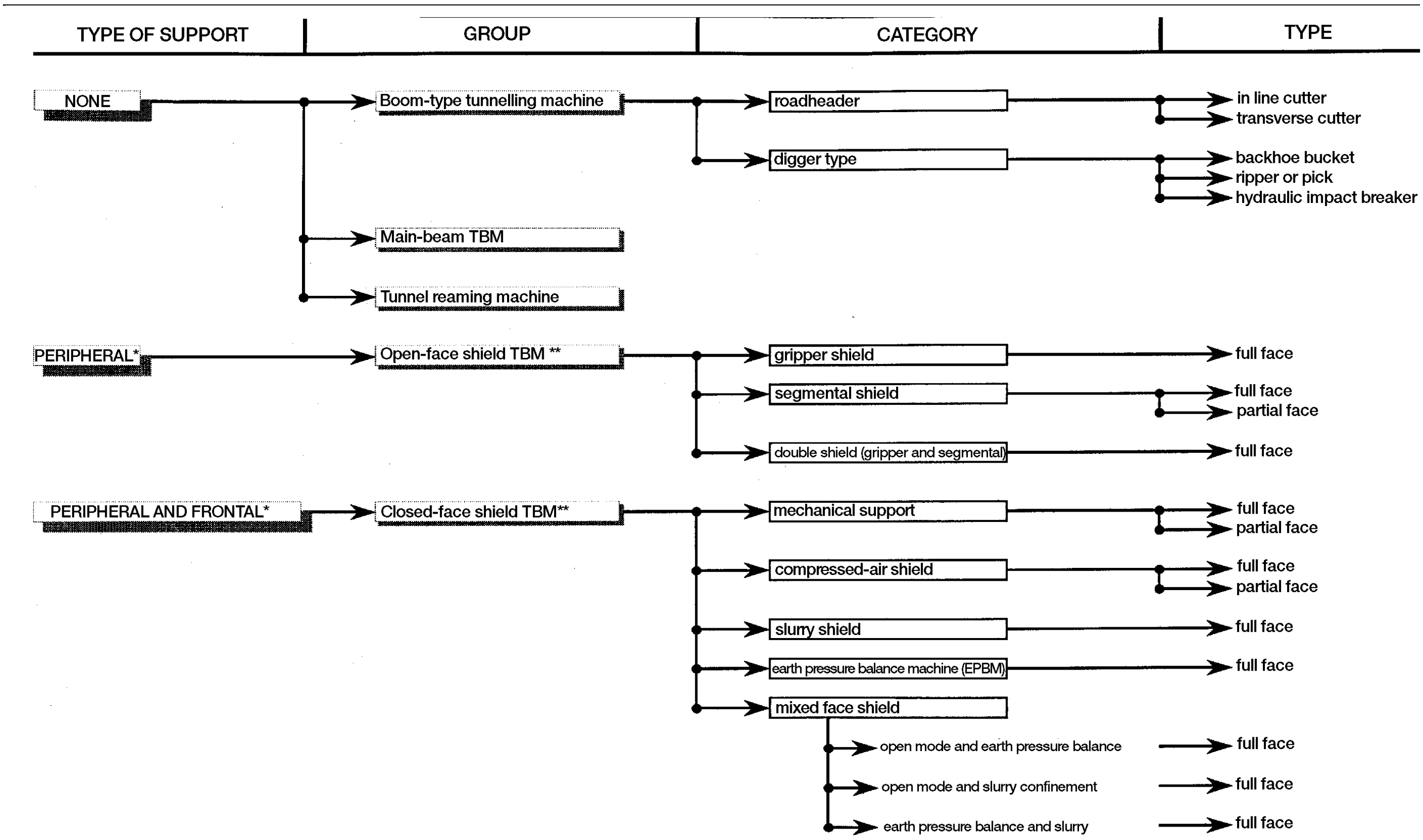
4.1.3 - Main-beam TBM

A main-beam TBM has a cutterhead that excavates the full tunnel face in a single pass.

The thrust on the cutterhead is reacted by bearing pads (or grippers) which push radially against the rock of the tunnel wall.

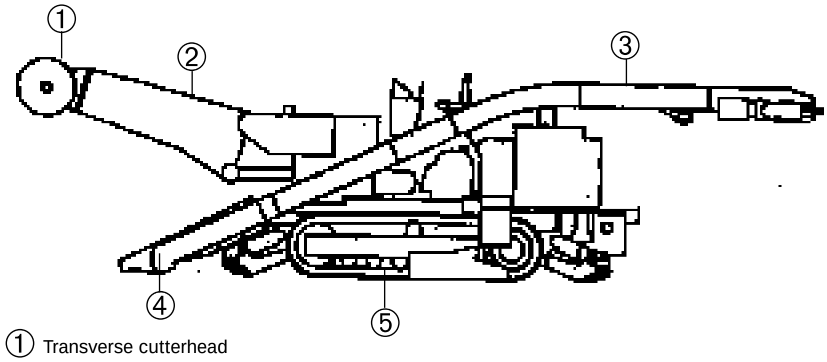
The machine advances sequentially, in two phases:

- Excavation (the gripper unit is stationary)
- Regripping



*For microtunnellers (diameter no greater than 1200 mm), refer to the work of the ISTT.
**Machines used in pipe-jacking and pipe-ramming are included in these groups.

CLASSIFICATION OF MECHANIZED TUNNELLING TECHNIQUES



① Transverse cutterhead
② Boom
③ Muck conveyor
④ Loading apron
⑤ Crawler chassis

Schéma 4.1.2

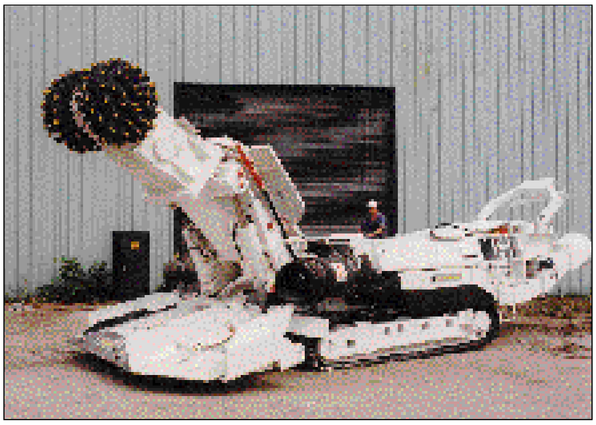


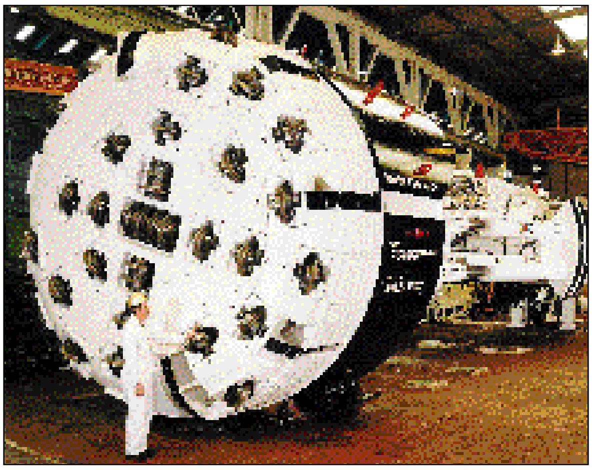
Photo 4.1.2 - Roadheader

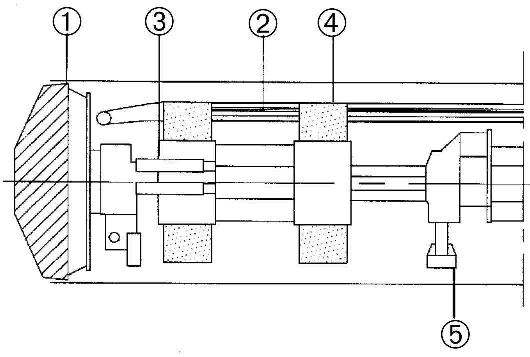
Spoil is collected and removed rearwards by the machine itself.

This type of TBM does not play an active role in immediate tunnel support.

AFTES data sheets: No. 1 to 7, 10 to 13, 15 to 24, 26 to 30, 67(photo 4.1.3)

Photo 4.1.3 - Lesotho Highlands Water Project

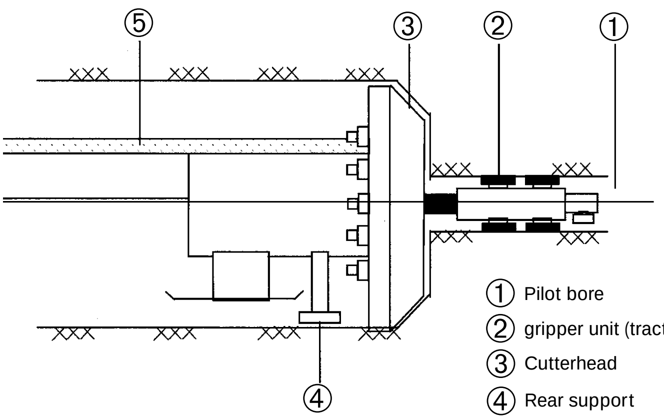




① Canopy/Hood/Roof
② Rear gripper
③ Front gripper
④ Muck conveyor
⑤ Rear lift leg

4.1.4 - Tunnel reaming machine

A tunnel reaming machine has the same basic functions as a main-beam TBM. It bores the final section from an axial tunnel (pilot bore) from which it pulls itself forward by means of a gripper unit.



① Pilot bore
② gripper unit (traction)
③ Cutterhead
④ Rear support
⑤ Muck conveyor

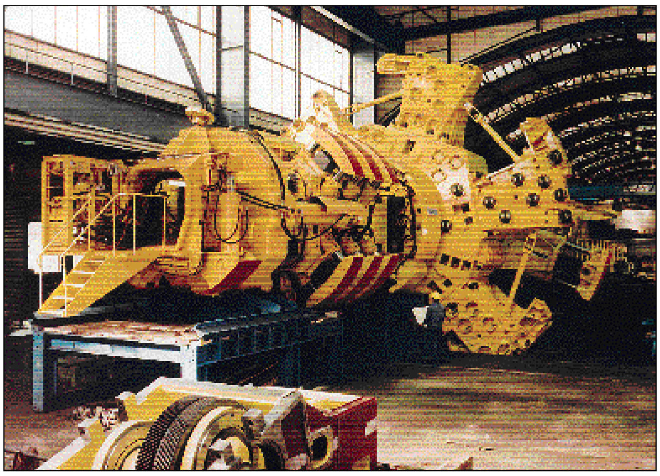


Photo 4.1.4 - Sauges tunnel (Switzerland)

4.2 - MACHINES PROVIDING IMMEDIATE PERIPHERAL SUPPORT

4.2.1 - General

Machines providing immediate peripheral support only belong to the open-face TBM group.

While they excavate they also support the sides of the tunnel. The tunnel face is not supported. d'aucune façon.

They can have two types of shield:

- one-can shield,
- shield of two or more cans connected by articulations.

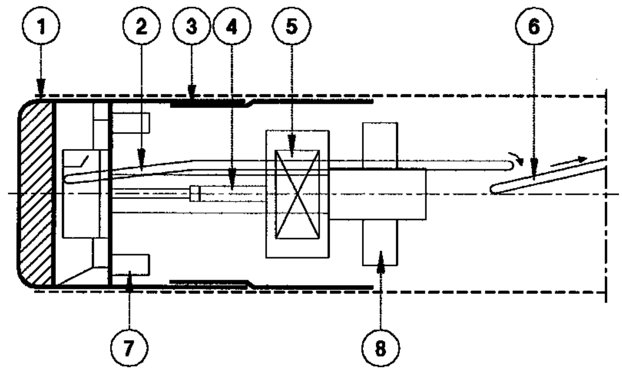
The different configurations for peripheral-support TBMs are detailed below.

4.2.2 - Open-face gripper shield TBM

A gripper shield TBM corresponds to the definition given in § 4.1.32 except that it is mounted inside a cylindrical shield incorporating grippers.

The shield provides immediate passive peripheral support to the tunnel walls.

AFTES data sheet: N° 25 ▼



1. Cutterhead
2. Muck extraction conveyor
3. Télescopic section
4. Thrust ram
5. Grippers (radial thrust)
6. Muck transfer conveyor
7. Motor
8. Segment erector

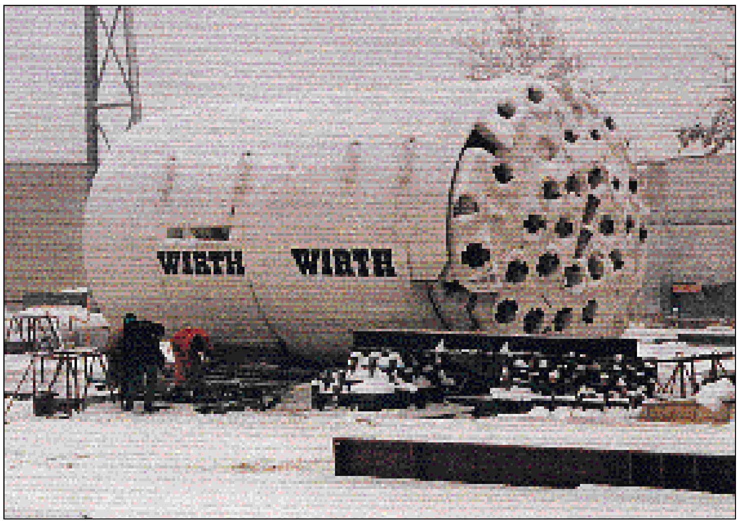
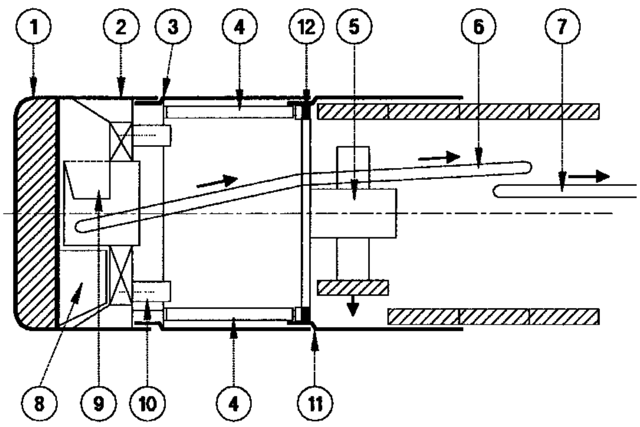


Photo 4.2.2 - Main CERN tunnel

4.2.3 - Open-face segmental shield TBM

An open-face segmental shield TBM is fitted with either a full-face cutterhead or an excavator arm like those of the different boom-type units. To advance and tunnel, the TBM's longitudinal thrust rams react against the tunnel lining erected behind it by a special erector incorporated into the TBM.

AFTES data sheets: No. 31 - 32 - 41 - 66 ▼



- a Cutterhead
- b Shield
- c Articulation (option)
- d Thrust ram
- e Segment erector
- f Muck extraction conveyor
- g Muck transfer conveyor
- h Gathering arm
- i Muck hopper
- j Motor
- k Tailskin articulation (option)
- l Thrust ring

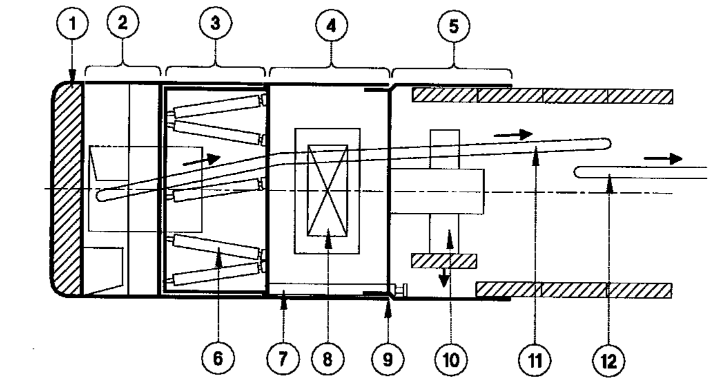
Photo 4.2.3
Athens metro

4.2.4 - Double shield

A double shield is a TBM with a full-face cutterhead and two sets of thrust rams that react against either the tunnel walls (radial grippers) or the tunnel lining. The thrust method used at any time depends on the type of ground encountered. With longitudinal thrust, segmental lining must be installed behind the machine as it advances.

The TBM has three or more cans connected by articulations and a telescopic central unit which relays thrust from the gripping/thrusting system used at the time to the front of the TBM.

AFTES data sheets: No. 65 – 68 – 71



a Cutterhead
b Front can
c Telescopic section
d Gripper unit
e Tailskin
f Main thrust rams
g Longitudinal thrust rams
h Grippers
i Tailskin articulation (option)
j Segment erector
k Muck extraction conveyor
l Muck transfer conveyor

Photo 4.2.4 - Salazie water transfer project (Reunion Island)

4.3 - MACHINES PROVIDING IMMEDIATE PERIPHERAL AND FRONTAL SUPPORT SIMULTANEOUSLY

4.3.1 - General

The TBMs that provide immediate peripheral and frontal support simultaneously belong to the closed-faced group.

They excavate and support both the tunnel walls and the face at the same time.

Except for mechanical-support TBMs, they all have what is called a cutterhead chamber at the front, isolated from the rearward part of the machine by a bulkhead, in which a confinement pressure is maintained in order to actively support the excavation and/or balance the hydrostatic pressure of the groundwater.

The face is excavated by a cutterhead working in the chamber.

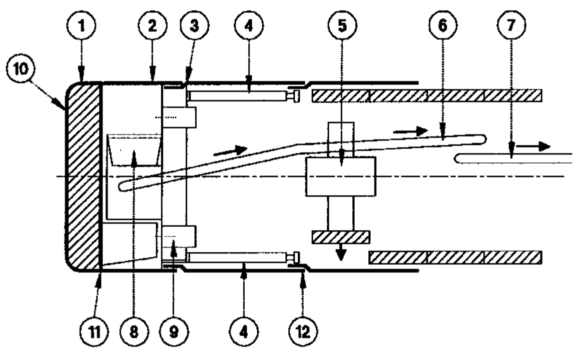
The TBM is jacked forward by rams pushing off the segmental lining erected inside the TBM tailskin, using an erector integrated into the machine.

4.3.2 - Mechanical-support TBM

A mechanical-support TBM has a full-face cutterhead which provides face support by constantly pushing the excavated material ahead of the cutterhead against the surrounding ground.

Muck is extracted by means of openings on the cutterhead fitted with adjustable gates that are controlled in real time.

AFTES data sheets: No. 38 – 39 – 40 – 51 – 58 – 64



a Cutterhead
b Shield
c Articulation (option)
d Thrust ram
e Segment erector
f Muck extraction conveyor
g Muck transfer conveyor
h Muck hopper (with optional gate)
i Cutterhead drive motor
j Gated cutterhead openings
k Peripheral seal between cutterhead and shield
l Tailskin articulation (option)

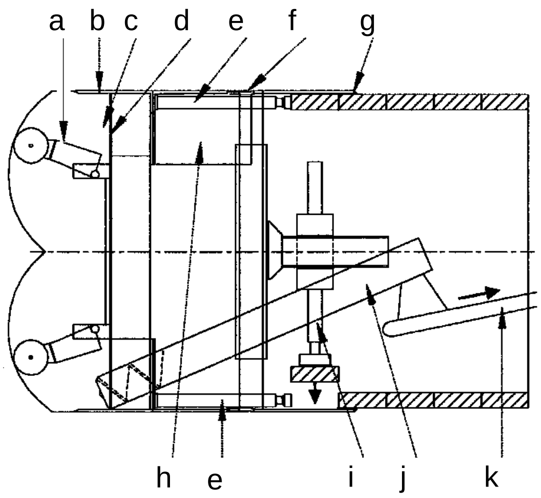
Photo 4.3.2
RER Line D (Paris)

4.3.3 - Compressed-air TBM

A compressed-air TBM can have either a full-face cutterhead or excavating arms like those of the different boom-type units. Confinement is achieved by pressurizing the air in the cutting chamber.

Muck is extracted continuously or intermittently by a pressure-relief discharge system that takes the material from the confinement pressure to the ambient pressure in the tunnel.

AFTES data sheets: No. 37 – 42 – 43 – 53 – 54 – 70



a Excavating arm
b Shield
c Cutting chamber
d Airtight bulkhead
e Thrust ram
f Articulation (option)
g Tailskin seal
h Airlock to cutting chamber
i Segment erector
j Screw conveyor (or conveyor and gate)
k Muck transfer conveyor

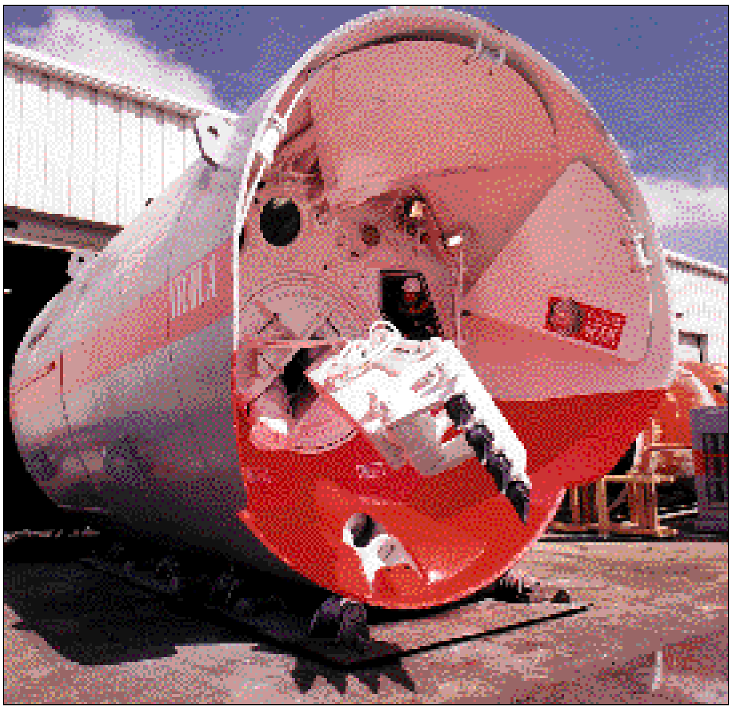
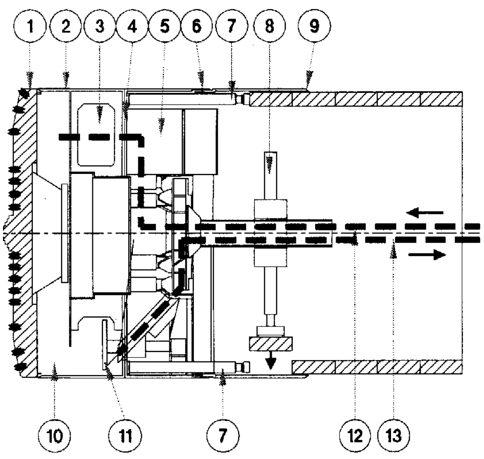


Photo 4.3.3 - Compressed air TBM - Boom type

4.3.4 - Slurry shield TBM

A slurry shield TBM has a full-face cutterhead. Confinement is achieved by pressurizing boring fluid inside the cutterhead chamber. Circulation of the fluid in the chamber flushes out the muck, with a regular pressure being maintained by directly or indirectly controlling discharge rates.

AFTES data sheets:
No. 33 – 34 – 35 – 36 – 44 – 50 – 52 – 56 – 57 - 60 – 62 – 63 – 69 – 76 – Cairo – Sydney



a Cutterhead
b Shield
c Air bubble
d Watertight bulkhead
e Airlock to cutterhead chamber
f Tailskin articulation (option)
g Thrust ram
h Segment erector
i Tailskin seal
j Cutterhead chamber
k Agitator (option)
l Slurry supply line
m Slurry return line

Photo 4.3.4 - Cairo metro

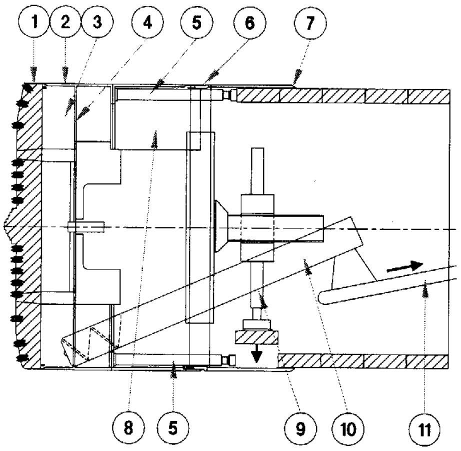
4.3.5 - Earth pressure balance machine

An earth pressure balance machine (EPBM) has a full-face cutterhead. Confinement is achieved by pressurizing the excavated material in the cutterhead chamber. Muck is extracted from the chamber continuously or intermittently by a pressure-relief discharge system that takes it from the confinement pressure to the ambient pressure in the tunnel.

EPBMs can also operate in open mode or with compressed-air confinement if specially equipped.

AFTES data sheets: No. 45 – 46 - 47 – 48 – 49 – 55 – 59 – 61 – 72 – 73 – 74* - 77 to 85

*TBMs also working with compressed-air confinement



a Cutterhead
b Shield
c Cutterhead chamber
d Airtight
e Thrust ram
f Articulation (option)
g Tailskin seal
h Airlock to cutterheau chamber
i Segment erector
j Screw conveyor
k Muck transfer conveyor

Photo 4.3.5 - CaluireTunnel, Lyons (France)

4.3.6 - Mixed-face shield TBM

Mixed-face shield TBMs have full-face cutterheads and can work in closed or open mode and with different confinement techniques.

Changeover from one work mode to another requires mechanical intervention to change the machine configuration.

Different means of muck extraction are used for each work mode.

There are three main categories of machine:

• Machines capable of working in open mode, with a belt conveyor extracting the muck, and, after a change in configuration, in closed mode, with earth pressure balance confinement provided by a screw conveyor;

• Machines capable of working in open mode, with a belt conveyor extracting the muck, and, after a change in configuration, in closed mode, with slurry confinement provided by means of a hydraulic mucking out system (after isolation of the belt conveyor);

• Machines capable of providing earth pressure balance and slurry confinement.

TBMs of this type are generally restricted to large-diameter bores because of the space required for the special equipment required for each confinement method.

AFTES data sheets: A86 Ouest (Socatop), Madrid metro packages 2 & 4, KCR 320 (Hong Kong)

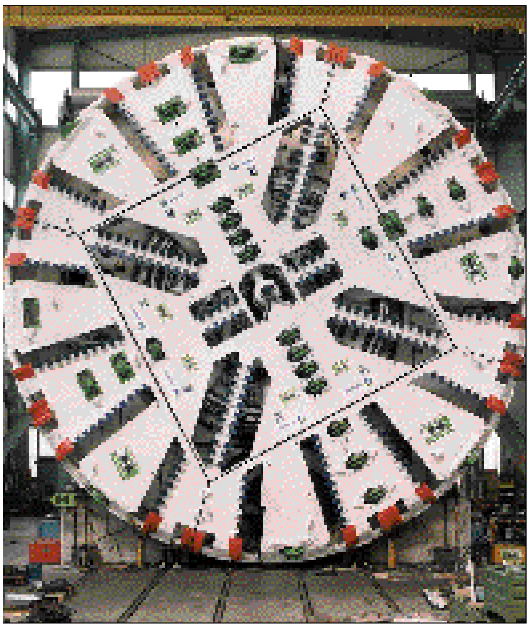


Photo 4.3.6a
A86 Oues tunnel (Socatop)

Photo 4.3.6b - A86 Ouest tunnel (Socatop)
Madrid metro

5 - EVALUATION OF PARAMETERS FOR CHOICE OF MECHANIZED TUNNELLING TECHNIQUES

5.1. GENERAL

It was felt useful to assess the degree to which elementary parameters of all kinds affect the decision-making process for choosing between the different mechanized tunnelling techniques.

The objectives of this evaluation are:

- to rank the importance of the elementary selection parameters, with some indication of the basic functions concerned.
- to enable project designers envisaging a mechanized tunnelling solution to check that all the factors affecting the choice have been examined.
- to enable contractors taking on construction of a project for which mechanized tunnelling is envisaged to check that they are in possession of all the relevant information in order to validate the solution chosen.

This evaluation is presented in the form of two tables (Tables 1 and 2).

Table 1 (§ 5.2.) indicates the degree to which each of the elementary selection parameters affects each of the basic functions of mechanized tunnelling techniques (all techniques combined).

Table 2 (§ 5.3) indicates the degree to which each of the elementary selection parameters affects each individual mechanized tunnelling technique.

These evaluation tables are complemented by comments in the appendix.

The list of parameters is based on that drawn up by AFTES recommendations work group No. 7 in its very useful document "Choix des paramètres et essais géotechniques utiles à la conception, au dimensionnement et à l'exécution des ouvrages creusés en souterrain" (Choice of geotechnical parameters and tests of relevance to the design and construction of underground works). This initial list has been complemented by factors other than geotechnical ones.

5.2 - EVALUATION OF THE EFFECT OF ELEMENTARY SELECTION PARAMETERS ON THE BASIC FUNCTIONS OF MECHANIZED TUNNELLING TECHNIQUES

Elementary parameters / Basic function	SUPPORT		OPPOSITION TO HYDROSTATIC PRESSURE	EXCAVATION	MUCKING OUT, EXTRACTION, TRANSPORT STOCKPILING
	Frontal	Peripherical			
	A	B	C	D	E
1. NATURAL CONTRAINTS	2	2	SO	1	0
2. PHYSICAL PARAMETERS					
2.1 Identification	2	1	2	2	1
2.2 Global appreciation of quality	2	2	0	1	0
2.3 Discontinuities	2	2	2	1	0
2.4 Alterability	1	1	SO	1	1
2.5 Water chemistry	1	0	SO	0	1
3. MECHANICAL PARAMETERS					
3.1 Strength — Soft ground	2	2	SO	1	0
Hard rock	1	1	SO	2	0
3.2 Deformability	2	2	SO	0	0
3.3 Liquefaction potential	0	0	0	0	0
4. HYDROGEOLOGICAL PARAMETERS	2	2	2	1	0
5. OTHER PARAMETERS					
5.1 Abrasiveness - Hardness	0	0	0	2	1
5.2 Propensity to stick	0	0	0	2	2
5.3 Ground/machine friction	0	1	0	0	0
5.4 Présence of gas	0	0	0	0	0
6. PROJECT CHARACTERISTICS					
6.1 Dimensions, shape	2	2	2	1	2
6.2 Vertical alignment	0	0	0	0	2
6.3 Horizontal alignment	0	0	0	0	1
6.4 Environment					
6.4.1 Sensitivity to settlement	2	2	2	0	0
6.4.2 Sensitivity to disturbance and work constraints	0	0	0	0	2
6.5 Anomalies in ground					
6.5.1 Heterogeneity of ground in tunnel section	1	1	0	2	0
6.5.2 Natural/artificial obstacles	0	0	0	1	0
6.5.3 Voids	2	2	2	0	0

2 : Decisive 1 : Has effect 0: No effect SO: Not applicable

See comments on this table in Appendix 1

Table 1

5.3 - EVALUATION DE L'INFLUENCE DES PARAMETRES ELEMENTAIRES DE CHOIX POUR LES SOLUTIONS DE TECHNIQUES D'EXCAVATION MECANISEE

Solution / Paramètres élémentaires	Machine n'assurant pas de soutènement		Machine assurant un soutènement laté-			Machine assurant un soutènement latéralet frontal				
	Machine à attaque ponctuelle	Aléseur et tunnelier à appui latéral	Bouclier mécanisé ouvert à appui radial	Bouclier mécanisé ouvert à appui longitudinal		Bouclier mécanisé à soutène-ment méca-	Bouclier mécanisé à confinement d'air comprimé		Bouclier mécanisé à confinement de boue	Bouclier mécanisé à confinement de terre
				A attaque pleine face	A attaque ponctuelle		A attaque pleine face	A attaque ponctuelle		
1. NATURAL CONTRAINTS	0	2	2	2	2	2	1	2	1	1
2. PHYSICAL PARAMETERS										
2.1 Identification	2	2	2	2	2	2	2	2	2	2
2.2 Global appreciation of quality	1	1	1	0	0	1	1	1	1	1
2.3 Soft ground/hard rock discontinuities	1/2	1/2	SO/2	1/2	1/2	1/2	2/2	2/2	2/2	1/2
2.4 Alterability	1	1	1	1	1	1	1	1	1	1
2.5 Water chemistry	0	0	0	0	0	0	1	0	1	1
3. MECHANICAL PARAMETERS										
3.1 Strength										
For soft ground	2	2	SO	2	2	2	1	2	1	1
For hard ground	2	1	1	SO	2	1	1	2	1	1
3.2 Deformability	0	1	1	1	1	1	1	1	1	1
3.3 Liquefaction potential	0	0	0	0	0	0	0	0	0	0
4. HYDROGEOLOGICAL PARAMETERS	2	2	2	2	2	2	2	2	2	2
5. OTHER PARAMETERS										
5.1 Abrasiveness - Hardness	2	1	1	1	1	1	1	1	1	1 ou 2
5.2 Propensity to stick	1	1	0	1	1	1	1	1	1	1
5.3 Ground/machine friction	0	1	1	1	1	1	1	1	0	1
5.4 Presence of gas	0	0	0	0	0	0	1	1	0	1
6. PROJECT CHARACTERISTICS										
6.1 Dimensions, shape	0	2	2	2	2	2	2	2	1	1
6.2 Vertical alignment	1	1	1	1	1	1	1	1	0	0
6.3 Horizontal alignment	0	1	1	1	1	1	0	0	0	0
6.4 Environment										
6.4.1 Sensitivity to settlement	2	2	2	2	2	2	2	2	1	1
6.4.2 Sensitivity to disturbance and work constraints	1	1	1	1	1	1	1	1	1 à 2	1
6.5 Anomalies in ground										
6.5.1 Heterogeneity of ground	1	1	1	1	1	1	2	2	1	1 à 2
6.5.2 Natural/artificial obstacles	0	1	1	1	1	1	1	1	1	1 à 2
6.5.3 Voids	1	1	1	1	1	1	1	1	1	1

2: Decisive 1: Has effect 0: No effect SO: Not applicable

See comments on this table in Appendix 2

Table 2

6 - SPECIFIC FEATURES OF THE DIFFERENT TUNNELLING TECHNIQUES

6.1 - MACHINES PROVIDING NO IMMEDIATE SUPPORT

6.1.1 - Specific features of boom-type tunnelling machines

a) General

Boom-type tunnelling machines are generally suited to highly cohesive soils and soft rock. They consist of an excavating arm or boom mounted on a self-propelling chassis. There is no direct relationship between the machine and the shape of the tunnel to be driven; the tunnel cross-sections excavated can be varied and variable. The face can be accessed directly at all times. Since these machines react directly against the tunnel floor, the floor must have a certain bearing capacity.

b) Excavation

The arms or booms of these machines are generally fitted with a cutting or milling head which excavates the face in a series of sweeps. These machines are called roadheaders. The maximum thrust on the roadheader cutterhead is directly related to the mass of the machine. The cutters work either transversally (perpendicular to the boom) or in-line (axially, about the boom axis). In most cases the spoil falling from the face is gathered by a loading apron fitted to the front of the machine and transported to the back of the machine by belt conveyor. This excavation method generates a lot of dust which has to be controlled (extraction, water spray, filtering, etc.).

In some cases the cutterhead can be replaced by a backhoe bucket, ripper, or hydraulic impact breaker.

c) Support and opposition to hydrostatic pressure

There is no tunnel support associated with this type of machine. It must be accompanied by a support method consistent with the shape of the tunnel and the ground conditions encountered (steel ribs, rockbolts, shotcrete, etc.).

This type of machine cannot oppose hydrostatic pressure, so accompanying measures (ground improvement, groundwater lowering, etc.) may be necessary.

d) Mucking out

Mucking out can be associated with this kind of machine or handled separately. It can be done directly from the face.

6.1.2 - Specific features of main-beam TBMs

a) General

The thrust at the cutterhead is reacted to one or two rows of radial thrust pads or grippers which take purchase directly on the tunnel walls. As with shield TBMs, a trailing backup behind the machine carries all the equipment it needs to operate and the associated logistics. Forward probe drilling equipment is generally fitted to this type of TBM. The face can be accessed by retracting the cutterhead from the face when the TBM is stopped.

The machine advances sequentially (bore, regrip, bore again).

b) Excavation

These full-face TBMs generally have a rotary cutterhead dressed with different cutters (disc cutters, drag bits, etc.). Muck is generally removed by a series of scrapers and a bucket chain which delivers it onto a conveyor transferring it to the back of the machine. Water spray is generally required at the face both to keep dust down and to limit the temperature rise of the cutters.

c) Support and opposition to hydrostatic pressure

Tunnel support is independent of the machine (steel ribs, rockbolts, shotcrete, etc.) but can be erected by auxiliary equipment mounted on the beam and/or backup. If support is erected from the main beam, it must take account of TBM movement and the gripper advance stroke. The cutterhead is not generally designed to hold up the face. A canopy or full can is sometimes provided to protect operators from falling blocks.

This kind of TBM cannot oppose hydrostatic pressure. Accompanying measures (groundwater lowering, drainage, ground improvement, etc.) are required if the expected pressures or inflows are high.

d) Mucking out

Mucking out is generally done with wagons or by belt conveyor. It is directly linked to the TBM advance cycle.

6.1.3. Specific features of tunnel reaming machines

a) General

Tunnel reaming machines work in much the same way as main-beam TBMs, except that the cutterhead is pulled rather than pushed. This is done by a traction unit with grippers in a pilot bore. As with all main-beam and shield machines, the cutterhead is rotated by a series of hydraulic or electric motors. The tunnel can be reamed in a single pass with a single cutterhead or in several passes with cutterheads of increasing diameter.

b) Excavation

See Chapter 6.1.2 § b) (main-beam TBM).

c) Support and opposition to hydrostatic pressure

The support in the pilot bore must be destructible (glass-fibre rockbolts) or removable (steel ribs) so that the cutterhead is not damaged. The final support is independent of the reaming machine, but can be erected from its backup.

For details on opposition to the hydrostatic pressure, see Chapter 6.1.2 § c (main-beam TBM).

d) Mucking out

See Chapter 6.1.2.§ d) (main-beam TBM).

6.2 - SPECIFIC FEATURES OF MACHINES PROVIDING IMMEDIATE PERIPHERAL SUPPORT

6.2.1 - Specific features of open-face gripper shield TBMs

a) General

An open-face gripper shield TBM is the same as a main-beam TBM except that it has a cylindrical shield.

The thrust of the cutterhead is reacted against the tunnel walls by means of radial pads (or grippers) taking purchase through openings in the shield or immediately behind it. As with other TBM types, a backup trailing behind the TBM carries all the equipment it needs to operate, together with the associated logistics.

The TBM does not thrust against the tunnel lining or support.

b) Excavation

See Chapter 6.1.2 § b) (main-beam TBM).

c) Support and opposition to hydrostatic pressure

The TBM provides immediate passive peripheral support. It also protects workers from the risk of falling blocks. If permanent tunnel support is required, it consists either of segments (installed by an erector on the TBM) or of support erected independently.

This type of machine cannot oppose hydrostatic pressure, so accompanying measures (ground improvement, groundwater lowering, etc.) may be necessary when working in water-bearing or unstable terrain.

d) Mucking out

See Chapter 6.1.2 § d) (main-beam TBM).

6.2.2 - Specific features of open-face segmental shield TBMs

a) General

An open-face shield segmental TBM has either a full-face cutterhead or an excavating arm like those of the different boom-type tunnelling machines. The TBM is thrust forward by rams reacting longitudinally against the tunnel lining erected behind it.

b) Excavation

TBM advance is generally sequential:

1) boring under thrust from longitudinal rams reacting against the tunnel lining

2) retraction of thrust rams and erection of new ring of lining.

c) Support and opposition to hydrostatic pressure

The TBM provides passive peripheral support and also protects workers from the risk of falling blocks.

The tunnel face must be self-supporting. Even a full-face cutterhead can only hold up the face under exceptional conditions (e.g. limitation of collapse when the TBM is stopped).

Temporary or final lining is erected behind the TBM by an erector mounted on it. It is against this lining that the rams thrust to push the machine forward.

This type of machine cannot oppose hydrostatic pressure, so accompanying measures (ground improvement, groundwater lowering, etc.) may be necessary when working in water-bearing or unstable terrain.

d) Mucking out

Muck is generally removed by mine cars or belt conveyors. Mucking out is directly linked to the TBM advance cycle.

6.2.3 - Specific features of double shield TBMs

Double shield TBMs combine radial purchase by means of grippers with longitudinal purchase by means of thrust rams reacting against the lining. A telescopic section at the centre of the TBM makes it possible for excavation to continue while lining segments are being erected.

Excavation proceeds as follows: with the rear section of the TBM secured by the grippers, the front section thrusts against it by means of the main rams between the two sections, and tunnels forward. A ring of segmental lining segments is erected at the same time. The grippers are then released and the longitudinal rams thrust against the tunnel lining to shove the rear section forward. The rear section regrips and the cycle is repeated.

6.3 - SPECIFIC FEATURES OF TBMS PROVIDING IMMEDIATE FRONTAL AND PERIPHERAL SUPPORT

6.3.1 - Specific features of mechanical-support shield TBMs

a) General

Mechanical-support shield TBMs ensure the stability of the excavation by retaining excavated material ahead of the cutterhead. This is done by partially closing gates on openings in the head.

b) Excavation

The face is excavated by a full-face cutterhead.

c) Support and opposition to hydrostatic pressure

Real-time adjustment of the openings in the cutterhead holds spoil against the face.

Frontal support is achieved by holding spoil against the face (in front of the cutterhead).

The shield provides immediate passive peripheral support.

The tunnel lining is erected:

• either inside the TBM tailskin, in which case it is sealed against the tailskin (tail seal) and back grout is injected into the annular space around it,

• or behind the TBM tailskin (expanded lining, segments with pea-gravel backfill and grout).

This type of machine cannot oppose hydrostatic pressure as a rule, so accompanying measures (ground improvement, groundwater lowering, etc.) may be necessary when working in water-bearing or unstable terrain.

d) Mucking out

Mucking out is generally by means of mine cars or belt conveyors.

6.3.2 - Specific features of compressed-air TBMs

a) General

With compressed-air TBMs, only pressurization of the air in the cutter chamber opposes the hydrostatic pressure at the face.

Compressed-air confinement pressure is practically uniform over the full height of the face. On the other hand, the pressure diagram for thrust due to water and ground at the face is trapezoidal. This means there are differences in the balancing of pressures at the face. The solution generally adopted involves compressing the air to balance the water pressure at the lowest point of the face. The greater the diameter, the greater the resulting pressure differential; for this reason the use of compressed-air confinement in large-diameter tunnels must be studied very attentively.

Compressed-air TBMs are generally used with moderate hydrostatic pressures (less than 0.1 MPa).

b) Excavation

The face can be excavated by a variety of equipment (from diggers to full-face cutterheads dressed with an array of tools). In the case of rotating cutterheads, the size of the spoil discharged is controlled by the openings in the cutterheadla roue.

Muck can be extracted from the face by a screw conveyor (low hydrostatic pressure) or by an enclosed conveyor with an airlock.

c) Support and opposition to hydrostatic pressure

Mechanical immediate support of the tunnel face and walls excavation is provided by the cutterhead and shield respectively.

The hydrostatic pressure in the ground is opposed by compressed air.

d) Mucking out

Muck is generally removed by conveyor or by wheeled vehicles (trains, trucks, etc.).

6.3.3 - Specific features of slurry shield TBMs

a) General

The principle of slurry shield TBM operation is that the tunnel excavation is held up by means of a pressurized slurry in the cutterhead. The slurry entrains spoil which is removed through the slurry return line.

The tunnel lining is erected inside the TBM tailskin where a special seal (tailskin seal) prevents leakage.

Back grout is injected behind the lining as the TBM advances.

b) Excavation

The face is excavated by a full-face cutterhead dressed with an array of cutter tools. Openings in the cutterhead (plus possibly a crusher upline of the first slurry return line suction pump) control the size of spoil removed before it reaches the pumps.

c) Support and opposition to hydrostatic pressure

Frontal and peripheral support of the tunnel excavation are the same, i.e. by means of the slurry pressure generated by the hydraulic mucking out system.

In permeable ground (K ≥ 5 x 10-5 m/s) it is possible to pressurize the chamber by creating a 'cake' of thixotropic slurry (bentonite, polymer, etc.), generally with relative density of between 1.05 and 1.15, on a tunnel face and walls.

With such a 'cake' in place it is possible for workers to enter the pressurized cutterhead (via an airlock).

The TBM can be converted to open mode, but the task is complex.

As for tunnel support, the hydrostatic pressure is withstood by forming a 'cake' to help form a hydraulic gradient between the hydrostatic pressure in the ground and the slurry pressure in the cutterhead chamber.

Together with control of the stability of the excavation and of settlement, opposition to hydrostatic pressure is a design consideration for the confinement pressure; the confinement pressure is regulated either by direct adjustment of the slurry supply and return pumps or by means of an "air bubble" whose level and pressure are controlled by a compressor and relief valves. With an "air bubble" in the cutterhead chamber the confinement pressure can be measured and regulated within a very narrow range of variation.

d) Mucking out

Muck is removed by pumping it through the pipes connecting the TBM to the slurry separation and recycling plant.

In most cases the muck is often treated outside the tunnel, in a slurry separation plant. This does introduce some risks associated with the type of spoil to be treated (clogging of plant, difficulties for disposal of residual sludge).

The pump flowrate and the treatment capacity of the separation plant determine TBM progress.

6.3.4 - Specific features of earth pressure balance machines

a) General

The principle of EPBM operation is that the excavation is held up by pressurizing the spoil held in the cutterhead chamber to balance the earth pressure exerted. If necessary, the bulked spoil can be made more plastic by injecting additives from the openings in the cutterhead chamber, the pressure bulkhead, and the muck-extraction screw conveyor. By reducing friction, the additives reduce the torque required to churn the spoil, thus liberating more torque to work on the face. They also help maintain a constant confinement pressure at the face.

Muck is extracted by a screw conveyor, possibly together with other pressure-relief devices.

The tunnel lining is erected inside the TBM tailskin, with a tailskin seal ensuring there are no leaks. Back grout is injected behind the lining as the TBM advances.

b) Excavation

The tunnel is excavated by a full-face cutterhead dressed with an array of tools. The size of spoil removed is controlled by openings in the cutterhead which are in turn determined by the dimensional capacity of the screw conveyor.

The power at the cutterhead has to be high because spoil is constantly churned in the cutterhead chamber.

c) Support and opposition to hydrostatic pressure

Face support is uniform. It is obtained by means of the excavated spoil and additives which generally maintain its relative density at between 1 and 2. Peripheral support can be enhanced by injecting products through the shield.

For manual work to proceed in the cutterhead chamber, it may be necessary to create a sealing cake at the face through controlled substitution (without loss of confinement pressure) of the spoil in the chamber with bentonite slurry.

L'architecture de ce type de tunnelier permet un passage rapide du mode fermé en mode ouvert.

The hydrostatic pressure is withstood by forming a plug of confined earth in the chamber and screw conveyor; the pressure gradient between the face and the spoil discharge point is balanced by pressure losses in the extraction and pressure-relief device.

Care must be take over the type and location of sensors in order to achieve proper measurement and control of the pressure in the cutterhead chamber.

d) Mucking out

After the muck-extraction screw conveyor, spoil is generally transported by conveyors or by wheeled vehicles (trains, trucks).

The muck is generally "diggable", enabling it to be disposed of without additional treatment; however, it may be necessary to study the biodegradability of the additives if the disposal site is in a sensitive environment.

The architecture of this type of TBM allows for rapid changeover from closed to open mode and vice versa.

7 - APPLICATION OF MECHANIZED TUNNELLING TECHNIQUES

7.1 - MACHINES NOT PROVIDING IMMEDIATE SUPPORT

7.1.1 - Boom-type tunnelling machines

Boom-type units are generally suitable for highly cohesive soils and soft rock. They reach their limits in soils with compressive strength in excess of 30 to 40 MPa, which corresponds to class R3 to R5 in the classification given in Appendix 3 (depending on the degree of cracking or foliation). The effective power of these machines is directly related to their weight.

When these machines are used in water-bearing ground, some form of ground improvement must be carried out beforehand to overcome the problem of significant water inflow.

When excavating clayey soils in water, the cutters of roadheaders may become clogged or balled; in such terrain, a special study of the cutters must carried out to overcome the problem. It may be advisable to use a backhoe instead.

These techniques are particularly suitable for excavating tunnels with short lengths of different cross-sections, or where the tunnel is to be driven in successive headings.

The tunnel support accompanying this method of excavation is independent of the machine used. It will be adapted to the conditions encountered (ground, environment, etc.) and the shape of the excavation.

7.1.2 - Main-beam TBMs

Main-beam TBMs are particularly suited to tunnels of constant cross-section in rock of strength classes R1 to R4 (see rock classification in Appendix 3).

For the lower strength classes (R3b-R4), the bearing surface of the grippers is generally increased in order to prevent them punching into the ground. If there is a risk of alteration of the tunnel floor due to water, laying a concrete invert behind the machine will facilitate movement of the backup. To provide short-term stabilization of the excavation, it will be necessary to have rapid support-erection systems that will be independent of but nevertheless compatible with the TBM.

For the higher strength classes (R1-R2a), all the boreability parameters must be taken into account in the TBM design.

In hard and abrasive ground in particular, it is recommended that every precaution be taken to allow for cutters to be replaced in perfect safety.

A system for spraying water on the tunnel face will cool the cutters and keep dust down. It can be complemented by dust screens, extraction, and filters.

Main-beam TBMs are generally fitted with destructive drilling rigs for forward probe drilling, together with drill data-logging equipment. The probe holes are drilled when the TBM is not working.

The design of these machines does not allow them to support non-cohesive soils as they advance, or to oppose hydrostatic pressure. For this reason accompanying measures such as drainage and/or consolidation of the ground are necessary before the machines traverse a geological accident. Consequently the TBM must be equipped to detect such features and to treat the ground ahead of the face when necessary.

7.1.3 - Tunnel reaming machines

Tunnel reamers are suitable for excavating large horizontal or inclined tunnels (upwards of 8 m in diameter) in rock (R1 to R3, sometimes R4 and R5).

The advantages of reaming a tunnel from a pilot bore are as follows:

• The ground is investigated as the pilot bore is driven

• Any low-strength ground encountered can be consolidated from the pilot bore before full-diameter excavation

• The ground to be excavated is drained

• The pilot bore can be used for dewatering and ventilation

• Temporary support can be erected independently of the machine.

7.2 - MACHINES PROVIDING IMMEDIATE PERIPHERAL SUPPORT

7.2.1 - Open-face gripper shield TBMs

Open-face gripper shield TBMs are particularly suitable for tunnelling in rock of strength classes between R1 and R3

The shield provides immediate support for the tunnel and/or protects the workforce from falling blocks.

The shield can help get through certain geological difficulties by avoiding the need for support immediately behind the cutterhead.

Application of this technique can be limited by the ability of the ground to withstand the radial gripper thrust.

The general considerations outlined in § 7.1.2 also apply here.

7.2.2 - Open-face segmental shield TBMs

An open-face segmental shield TBM requires full lining or support along the length of the tunnel against which it can thrust to advance.

Its field of application is soft rock (strength classes R4 and R5) and soft ground requiring support but in which the tunnel face holds up.

The general considerations outlined in § 7.1.2 also apply here.

This type of TBM can traverse certain types of heterogeneity in the ground. It also enables the tunnel support to be industrialized to some extent. On the other hand, the presence of the lining and shield can give rise to difficulties when crossing obstacles such as geological accidents, since they hinder access to the face for treatment or consolidation of the ground.

7.2.3 - Open-face double shield TBMs

Open-face double shield TBMs combine the advantages and disadvantages associated with radial grippers and longitudinal thrust rams pushing off tunnel lining: they need either a lining or ground of sufficient strength to withstand gripper thrust.

This greater technical complexity is sometimes chosen when lining is required so that boring can proceed (with gripper purchase) while the lining ring is being erected.

7.3 - MACHINES PROVIDING IMMEDIATE FRONTAL AND PERIPHERAL SUPPORT

7.3.1 - Mechanical-support shield TBMs

The difference between mechanical-support shield TBMs and open-face segmental shield TBMs lies in the nature of the cutterhead. Mechanical-support TBMs have:

• openings with adjustable gates

• a peripheral seal between the cutterhead and the shield.

Face support is achieved by holding spoil ahead of the cutterhead by adjusting the openings. It does not provide ‘genuine’ confinement, merely passive support of the face.

Its specific field of application is therefore in soft rock and consolidated soft ground with little or no water pressure

7.3.2 - Compressed-air TBMs

Compressed-air TBMs are particularly suitable for ground of low permeability with no major discontinuities (i.e. no risk of sudden loss of air pressure).

The ground tunnelled must necessarily have an impermeable layer in the overburden.

Compressed-air TBMs tend to be used to excavate small-diameter tunnels.

Their use is not recommended in circumstances where the ground at the face is heterogeneous (unstable ground in the roof which could cave in). They should be prohibited in organic soil where there is a risk of fire.

In the case of small-diameter tunnels, it may be possible to have compressed air in all or part of the finished tunnel.

7.3.3 - Slurry shield TBMs

Slurry shield TBMs are particularly suitable for use in granular soil (sand, gravel, etc.) and heterogeneous soft ground, though they can also be used in other terrain, even if it includes hard-rock sections.

There might be clogging and difficulty separating the spoil from the slurry if there is clay in the soil.

These TBMs can be used in ground with high permeability (up to 10-2 m/s), but if there is high water pressure a special slurry has to be used to form a watertight cake on the excavation walls. However, their use is usually restricted to hydrostatic pressures of a few dozen MPa.

Generally speaking, good control of slurry quality and of the regularity of confinement pressure ensures that surface settlement is kept to the very minimum.

Contaminated ground (or highly aggressive water) may cause problems and require special adaptation of the slurry mix design.

The presence of methane in the ground is not a problem for this kind of TBM.

If the tunnel alignment runs through contrasting heterogeneous ground, there may be difficulties extracting and processing the spoil.

7.3.4 - Earth pressure balance machines

EPBMs are particularly suitable for soils which, after churning, are likely to be of a consistency capable of transmitting the pressure in the cutterhead chamber and forming a plug in the muck-extraction screw conveyor (clayey soil, silt, fine clayey sand, soft chalk, marl, clayey schist).

They can handle ground of quite high permeability (10–3 to 10-4 m/s), and are also capable of working in ground with occasional discontinuities requiring localized confinement.en l'absence

In hard and abrasive ground it may be necessary to use additives or to take special measures such as installing hard-facing or wearplates on the cutterhead and screw conveyor.a vitesse de progression de l'usure par

In permeable ground, maintenance in the cutterhead chamber is made complex because of the need to establish a watertight cake at the face beforehand, without losing confinement pressure.

8 - TECHNIQUES ACCOMPANYING MECHANIZED TUNNELLING

8.1 - PRELIMINARY INVESTIGATIONS FROM THE SURFACE

8.1.1 - Environmental impact assessment

At the preliminary design stage an environmental impact assessment should be carried out in order to properly assess the dimensional characteristics proposed for the tunnel, particularly its cross-section, sectional area, and overburden.

In addition, the effect and sensitivity of settlement-especially in built-up areas-should be given special attention. This is a decisive factor in choosing the tunnelling and support methods, the tunnel alignment, and the cross-section.

The environmental impact assessment should be thorough, taking account of the density of existing works and the diversity of their behaviours.

For existing underground works, the compatibility of the proposed tunnelling and support methods or the adaptations required (special treatment or accompanying measures) should be assessed through special analysis.

8.1.2 - Ground conditions

The purpose of preliminary investigations is not just for design of the temporary and permanent works, but also to check the feasibility of the project in constructional terms, i.e. with respect to excavation, mucking out, and short- and long-term stability.

Design of the works involves determining shape, geological cross-sections, the physical and mechanical characteristics of the ground encountered by the tunnel, and the hydrogeological context of the project as a whole.

Project feasibility is determined by the potential reactions of the ground, including details of both the formations traversed and of the terrain as a whole, with respect to the loadings generated by the works, i.e. with respect to the excavation/confinement method adopted.

Depending on the context and the specific requirements of the project, the synopsis of investigation results should therefore deal with each of the topics detailed in the AFTES recommendations on the choice of geotechnical tests and parameters, irrespective of the geological context (cf.: T.O.S No. 28, 1978, re-issued 05/93 – review in progress; and T.O.S No. 123, 1994).

If the excavation/confinement method is only chosen at the tender stage, and depending on the confinement method chosen by the Contractor, additional investigations may have to be carried out to validate the various options adopted.

8.1.3 - Resources used

Depending on the magnitude and complexity of the project, preliminary investigations - traditionally based on boreholes and borehole tests - may be extended to “large-scale” observation of the behaviour of the ground by means of test adits and shafts.

Advantage can be taken of the investigation period to proceed with tests of the tunnelling and support methods as well as any associated treatments.

If there are to be forward probe investigations, matching of the boring and investigation methods should be envisaged at the preliminary investigation stage.

In the event of exceptional overburden conditions and difficult access from the surface, directional drilling investigation (mining and/or petroleum industry techniques) of long distances (one kilometre or more) along the tunnel alignment may be justified, especially if it is associated with geophysical investigations and appropriate in situ testing.

8.2 - FORWARD PROBING

The concept of forward probing must be set against the risk involved. This type of investigation is cumbersome and costly, for it penalizes tunnelling progress since—in the case of full-face and shield TBMs—the machine has to be stopped during probing (with current-day technology). It should therefore be used only in response to an explicit and absolute requirement to raise any uncertainty over the conditions to be expected when crossing areas where site safety, preservation of existing works, or the durability of the project might be at risk.

Irrespective of the methodology selected, it must give the specialists implementing it real possibilities for avoiding difficulties by implementing corrective action in good time.

The first condition that forward probing must meet in order to achieve this objective is that it give sufficiently clear and objective information about the situation ahead of the face (between 1 and 5 times the tunnel diameter ahead), with a leadtime consistent with the rate of tunnel progress.

The second condition is that in terms of quality it must be adapted to the specific requirements of the project (identification of clear voids, of decompressed areas, faults, etc.). These criteria should be determined jointly by the Designer, Engineer, and Contractor and should be clearly featured in specifications issued to the persons carrying out the investigations.

During tunnelling, analysis of results is generally the responsibility of the investigations contractor, but the interpretation of data, in correlation with TBM advance parameters (monitoring), should in principle be the responsibility of the contractor operating the TBM.

8.3 - GROUND IMPROVEMENT

Prior ground improvement is sometimes necessary, particularly in order to cross:

• singular features such as break-ins and breakouts, including on works along the route (shafts, stations, etc.)

• discontinuities and fault zones identified beforehand

• permeable water-bearing ground.

If the problem areas are of limited extent, ground improvement will sometimes enable a less sophisticated - and therefore less costly - tunnelling technique to be adopted.

Since ground improvement is long and costly to carry out from the tunnel (especially when the alignment is below the water table), the work is generally done from the surface (in the case of shallow overburden).

These days, however, there is a trend for TBMs to be fitted with the basic equipment (such as penetrations in the bulkhead and / or cans) enabling ground improvement to be carried out from the machine should water-bearing ground not compatible with the tunnelling technique adopted be encountered unexpectedly. This can also be the case when local conditions prohibit treatment from the surface.

When confinement-type TBMs are used, geological and hydrogeological conditions often require special treatment for break-ins and breakouts. This point should not be overlooked, neither at the preliminary design stage (surface occupation, ground and network investigations, works schedule) nor during the construction phase, for this is one of the most difficult phases of tunnelling.

Special attention should be given to the compatibility of ground treatment with the tunnelling process (foaming, reaction with slurry and additives, etc.)

The most commonly used ground improvement techniques are:

• permeation-grouted plug of bentonite-cement and / or gel

• diaphragm-wall box

• total replacement of soil by bentonite-cement

• jet-grouted plug

8.4 - GUIDANCE

Guidance of full-face TBMs is vital. The performance of the guidance system used must be consistent with the type of TBM and lining, and with the purpose of the tunnel.

The development of shield TBMs incorporating simultaneous erection of precast segmental lining has led to the design of highly sophisticated guidance systems, because with tunnel lining it is impossible to remedy deviation from the correct course. Consequently, the operator (or automatic operating system) must be given real-time information on the position of the face and the tunnelling trend relative to the theoretical alignment. However, when considering the construction tolerance it must be remembered that the lining will not necessarily be centred in the excavation, and that it may be subject to its own deformation (offset, ovalization, etc.). The generally accepted tolerance is an envelope forming a circle about 20 cm larger in diameter than the theoretical diameter.

Whatever the degree of sophistication of the guidance system, it is necessary to:

• reliably transfer a traverse into the tunnel and close it as soon as possible (breakout into shaft, station, etc.)

• carry out regular and precise topographical checks of the position of the TBM and of the tunnel

• know how quickly (speed and distance) the TBM can react to modifications to the trajectory it is on.

8.5 - ADDITIVES

a) General

Mechanized tunnelling techniques make use of products of widely differing physical and chemical natures that can all be labelled “conditioning fluids and slurries”. Before any chemical additives are used, it should be checked that they present no danger for the environment (they will be mixed in with the muck and could present problems when it is disposed of) or for the workforce (particularly during pressurized work in the cutterhead chamber where the temperature can be high).

b) Water

Water will be present in the ground in varying quantities, and will determine the soil's consistency, as can be seen from different geotechnical characterization tests or concrete tests (Atterberg limits for clayey soils and slump or Abrams cone test for granular soils). It can be used alone, with clay (bentonite), with hydrosoluble polymers, or with surfactants to form a conditioning fluid (slurry or foam).

c) Air

By itself air cannot be considered to be a boring additive in the same way as water or other products; its conditioning action is very limited. When used in pressurized TBMs - if the permeability of the ground does not prohibit it - air helps support the tunnel. As a compressible fluid, air helps damp confinement-pressure variations in the techniques using slurry machines with “air bubbles” and EPB machines with foam. As a constituent of foam, air also helps fluidify and reduce the density of muck, and helps regulate the confinement pressure in the earth-pressure-balance process.

d) Bentonite

Of the many kinds of clay, bentonite is most certainly the best-known drilling or boring mud. It has extremely high swell, due to the presence of its specific clayey constituent, montmorillonite, which gives it very interesting colloidal and sealing qualities.

In the slurry-confinement technique, the rheological qualities of bentonite (thixotropy) make it possible to establish a confinement pressure in a permeable medium by sealing the walls of the excavation through pressurized filtration of the slurry into the soil (formation of a sealing cake through a combination of permeation and membrane), and to transport muck by pumping.

Bentonite slurry can also be used with an EPB machine, to improve the consistency of the granular material excavated (homogenization, plastification, lubrication, etc.).

In permeable ground, the EPB technique uses the same principle of cake formation before work is carried out in the pressurized cutterhead chamber.

e) Polymers

Of the multitude of products on the market, only hydrosoluble or dispersible compounds are of any interest as tunnelling additives. Most of these are well known products in the drilling industry whose rheological properties have been enhanced to meet the specific requirements of mechanized tunnelling.

These modifications essentially concern enhanced viscosifying power in order to better homogenize coarse granular materials, and enhanced lubrifying qualities in order to limit sticking or clogging of the cutterhead and mucking out system when boring in certain types of soil.

Polymers may be of three types:

• natural polymers (starch, guar gum, xanthan gum, etc.)

• modified natural or semi-synthetic polymers (CMC [carboxymethylcellulose], etc.)

• synthetic polymers (polyacrylamides, polyacrylates, etc.)

f) Foams (surfactants)

Foams are two-phase systems (a gas phase and a liquid phase containing the foaming agent) which are characterized physically by their expansion factor (volume occupied by the air in the foam relative to the volume of liquid).

Foams are easy to use. They are similar to aerated slurries, combining the advantages of a gas (compressibility, practically zero density, etc.) and of a slurry (fluidification, lubrication, pore filling, etc.). With EPB machines they are used to facilitate confinement and sometimes excavation and mucking out as well.

8.6 - DATA LOGGING

The acquisition and restitution of TBM operating parameters is undoubtedly the biggest factor in the technical progress of mechanized tunnelling in the last ten years.

It makes for objective analysis of the operating status and dysfunctions of the machine and its auxiliaries.

The status of the machine at any given time is short-lived and changes rapidly. Without data logging, this gave rise to varied and often erroneous interpretations in the past.

Logging gives a “true” technical analysis that is indispensable for smooth operation on projects in difficult or sensitive sites.

Data logging also provides a basis for computerized control of TBM operation and automation of its functions (guidance, mucking out, confinement pressure regulation, etc.).

Data logging also provides an exact record of operating statuses and their durations (cf. recommendation on analysis of TBM operating time and coefficients, TOS No. 148, July 98).

They also constitute operating feedback that can be used to optimize TBM use.

8.7 - TUNNEL LINING AND BACKGROUTING

8.7.1 - General

In the case of segmental TBMs, the lining and its backgrouting are inseparable from the operation of the machine.

Without any transition and in perfectly controlled fashion, the lining and backgrout must balance the hydrostatic pressure, support the excavation peripherally, and limit surface settlement.

Because of their interfaces with the machine, they must be designed in parallel and in interdependence with the TBM.

8.7.2 - Lining

The lining behind a shield TBM generally consists of reinforced concrete segments. Sometimes (for small-diameter tunnels) cast-iron segments are used. More exceptionally the lining is slipcast behind a sliding form.

Reinforced concrete segments are by far the most commonly used. The other techniques are gradually being phased out for economic or technical reasons.

The segments are erected by a machine incorporated into the TBM which grips them either mechanically or by means of suction.

The following AFTES recommendations examine tunnel lining:

• Recommandations sur les revêtements préfabriqués des tunnels circulaires au tunnelier (Recommendations on precast lining of bored circular tunnels), TOS No. 86

• Recommandation sur les joints d’étanchéité entre voussoirs (Recommendations on gaskets between lining segments), TOS No. 116, March/April 1993

• Recommandations “pour la conception et le dimensionnement des revêtements en voussoirs préfabriqués en béton armé installés à l’arrière d’un tunnelier” (Recommendations “on the design of precast reinforced concrete lining segments installed behind TBMs”) drawn up by AFTES work group No. 18, published in TOS No. 147, May/June 1998.

8.7.3 - Backgrouting

This section concerns only mechanized tunnelling techniques involving segmental lining.

Experience shows the extreme importance of controlling the grouting pressure and filling of the annular space in order to control and restrict settlement at the surface and to securely block the lining ring in position, given that in the short term the lining is subject to its selfweight, TBM thrust, and possibly flotational forces.

Grouting should be carried out continuously, with constant control, as the machine advances, before a gap appears behind the TBM tailskin.

In the early days backfilling consisted of either pea gravel or fast-setting or fast-hardening cement slurry or mortar that was injected intermittently through holes in the segments.

Since management of the grout and its hardening between mixing and injection is a very complex task, there has been a constant trend to drop cement-based products in favour of products with retarded set (pozzolanic reaction) and low compressive strength. Such products are injected continuously and directly into the annular space directly behind the TBM tailskin by means of grout pipes routed through the tailskin.

9 - HEALTH AND SAFETY

Mechanization of tunnelling has very substantially improved the health and safety conditions of tunnellers. However, it has also induced or magnified certain specific risks that should not be overlooked. These include:

• risk of electrical fire or spread of fire to hydraulic oils

• risk of electrocution

• risks during or subsequent to compressed-air work

• risks inherent to handling of heavy parts (lining segments)

• mechanical risks

• risk of falls and slips (walkways, ladders, etc.)

9.1 - DESIGN OF TUNNELLING MACHINES

Tunnelling machines are work items that must comply with the regulations of the Machinery Directive of the European Committee for Standardization (CEN).

These regulations are aimed primarily at designers—with a view to obtaining equipment compliant with the Directive—but also at users.

The standards give the minimum safety measures and requirements for the specific risks associated with the different kinds of tunnelling machines. Primarily they apply to machines manufactured after the date of approval of the European standard.

• At the time of writing only one standard had been homologated:

- NF EN 815 “Safety of unshielded tunnel boring machines and rodless shaft boring machines for rock” (December 1996)

• Three are in the approval process:

- Pr EN 12111 “Tunnelling machines - Roadheaders, continuous miners and impact rippers – Safety requirements”

- Pr EN 12336 “Tunnelling machines – Shield machines, horizontal thrust boring machines, lining erection equipment - Safety requirements ”

- Pr EN 12110 “Tunnelling machines – Airlocks – Safety requirements ”

9.2 - USE OF TUNNELLING MACHINES

Machine excavation of underground works involves specific risks linked essentially to atmospheric pollution (gas, toxic gases, noise, temperature), flammable gases and other flammable products in the ground, electrical equipment (low and high voltage), hydraulic equipment (power or control devices), and compressed-air work (work in large-diameter cutterhead chambers under compressed air, pressurization of whole sections of small-diameter tunnels).

A variety of bodies dealing with safety on public works projects have drawn up texts and recommendations on safety. In France, these include OPPBTP, CRAM, and INRS, for example.

All their requirements should be incorporated into the General Co-Ordination Plan and Health and Safety Plan at the start of works.

APPENDICES 1, 2, 3, AND 4

1. Comments on Table No. 1 in Chapter 5
2. Comments on Table No. 2 in Chapter 5
3. Ground classification table
4. Mechanized tunnelling project data sheets

APPENDIX 1

COMMENTS ON TABLE NO. 1 IN CHAPTER 5.

1 - Natural constraints

Support (columns A and B)

With knowledge of natural constraints:

• a choice can be made from among the tunnelling technique groups (from boom-type units to confinement-type TBMs)

• relaxation of stresses can be managed (from simple deformation-convergence to failure).

2 - PHYSICAL PARAMETERS

2.1 - Identification

❑ Face support (column A)

With knowledge of physical parameters:

• the support method can be assessed, and the tunnelling technique group chosen

• the requirement for face support can be assessed.

❑ Peripheral support (column B)

With knowledge of physical parameters the requirement for peripheral support around the machine can be assessed.

❑ Opposition to hydrostatic pressure (column C)

With knowledge of physical parameters and of grain and block sizes, the permeability of the terrain can be assessed, leading to a proposal for the way hydrostatic pressure could be controlled.

❑ Excavation (column D)

Of the parameters concerned, grain and block size are decisive for assessing the excavation method (design of cutter head, cutters, etc.).

2.2 - Global appreciation of quality

❑ Support (columns A and B)

Global appreciation of quality provides additional information for identification that concerns only the sample. This data defines more global information at the scale of the soil horizon concerned.

2.3 - Discontinuities

❑ Support (columns A and B)

This data concerns rock and coherent soft ground. With knowledge of discontinuities a choice can be made among the tunnel technique groups (from boom-type units to confinement-type TBMs).

❑ Opposition to hydrostatic pressure (column C)

With knowledge of discontinuities the crack permeability and water pressure to be taken into account for the project can be assessed. This enables the type of technique to be chosen.

❑ Excavation (column D)

In conjunction with knowledge of block sizes, knowledge of discontinuities (nature, size, and frequency) can be decisive or merely have an effect on the excavation method to be adopted.

3 - MECHANICAL PARAMETERS

3.1 - Strength

❑ Support (columns A and B)

With knowledge of mechanical parameters a preliminary choice can be made from among the tunnelling technique groups (from boom-type units to confinement-type TBMs).

❑ Excavation (hard rock)(column D)

Knowledge of mechanical parameters is particularly important for defining the architecture of the machine and helps determine its technical characteristics (torque, power, etc.) and the choice of cutting tools.

3.2 - Deformability

❑ Support (columns A and B)

With knowledge of deformability the relaxation of stresses can be assessed and taken into account (from simple deformation or convergence to failure).

3.3 - Liquefaction potential

❑ Support and mucking out (columns A, B and E)

Knowledge of the liquefaction potential has an effect in seismic zones and in cases where the technique chosen might set up vibrations in the ground (blasting, etc.).

4 - HYDROGEOLOGICAL PARAMETERS

❑ Support, opposition to hydrostatic pressure, and excavation (Columns A, B, C and D)

Knowledge of these parameters is decisive in appreciating control of the stability of the tunnel, both at the face and peripherally, and therefore in choosing the method from the various tunnelling techniques. In the case of tunnels beneath deep overburden it is not easy to obtain these parameters. They should be estimated with the greatest care and analyzed with caution.

5 - OTHER PARAMETERS

❑ Excavation and mucking out (Columns D and E)

The parameters of abrasiveness and hardness are decisive or have an effect in appreciation of the excavation and mucking-out methods to be used. These parameters should be

APPENDIX 2

Comments on Table No. 1 in Chapter 5

1 - NATURAL CONSTRAINTS

The stress pattern in the ground is very important in deep tunnels or in cases of high anisotropy. If the rate of stress release is high, with main-beam TBMs, shield TBMs, and reaming machines, it may cause:

- jamming of the machine (jamming of the cutter head or body)
- rockburst at the face or in tunnel walls, roof, or invert.

With slurry-shield TBMs or EPBMs it is rare for the natural stress pattern to be decisive in the choice of machine type since they are generally used for shallow tunnels.

2 - PHYSICAL PARAMETERS

2.1 - Identification

The type of ground plays a decisive role in the choice and design of a shield TBM. Consequently the parameters characterizing the identification of the ground must be examined carefully when choosing the excavation/support method.

The most important of the identification parameters are **plasticity** and - for hardness, clogging potential, and abrasiveness - **mineralogy** which are particularly decisive in the selection of shield TBM components.

Chemical analysis of the soil can be decisive in the case of confinement-type shield TBMs because of the effect soil might have on the additives used in these techniques.

2.2 - Global appreciation of quality

Global appreciation of quality results from combining parameters which are easy to measure in the laboratory or in situ (borehole logs, RQD) and visual approaches.

Weathered zones and zones with contrasting hardness can cause specific difficulties for the different tunnelling techniques, e.g. face instability, insufficient strength for grippers, confinement difficulties.

The degree of weathering of rock has an effect but is not generally decisive for slurry shields and EPBMs. In all cases it has an effect for cutter head design.

2.3 - Discontinuities

For rock, knowledge of the situation regarding discontinuities is decisive (orientation and density of the network), for it will affect the choice of the tunnelling and support technique as well as the tunnelling speed.

With open-face main-beam TBMs and shields and mechanical-support TBMs, attention should be given to the risk of jamming of the machine induced by the density of a network of discontinuities which could quite rapidly lead to doubtful stability of the terrain. The existence of unconsolidated infilling material can aggravate the resulting instability.

The presence of major discontinuities can have a major effect on the choice of tunnelling technique.

Slurry shields and compressed-air TBMs are generally more sensitive to the presence of discontinuities than EPBMs. If there are major discontinuities (high density of fracturation), the compressed-air confinement TBM may have to be eliminated from the possible range.

In general the overall permeability of the terrain should be examined in conjunction with its discontinuities before selecting the type of confinement.

2.4 - Alterability

Alterability characteristics concern terrain that is sensitive to water. Alterability data should be obtained at the identification stage.

Special attention should be given to alterability when mechanized tunnelling is to take place in water-sensitive ground such as certain molasses, marls, certain schists, active clays, indurated clays, etc.

Alterability has an effect on confinement-type TBMs; it can result in changes being made to the design of the machine and the choice of additives.

2.5 - Water chemistry

Problems related to the aggressivity or the degree of pollution of water may arise in very specific cases and have to be dealt with regardless of the tunnelling principles adopted.

With confinement-type TBMs this parameter may be decisive because of its effect on the quality of the slurry or additives.

3 - MECHANICAL PARAMETERS

3.1 - Strength

In the case of rock, the essential mechanical criteria are the compressive and tensile strength of the terrain, for they condition the efficacy of excavation.

In soft ground, the essential criteria are cohesion and the angle of friction, for they condition the hold-up of the face and of the excavation as a whole.

The very high strengths of some rocks exclude the use of boom-type tunnelling machines (unless they are highly cracked). Gripper-type tunnel boring and reaming machines are very sensitive to low-strength ground and may require special adaptation of the gripper pads. For main-beam and shield TBMs alike, the machine architecture, the installed power at the cutter head, and the choice and design of cutting tools and cutter head are conditioned by the strength of the ground.

If there is any chance of tunnel bearing capacity being insufficient, special treatment may be necessary for the machine to advance.

3.2 - Deformability

Deformability of the terrain may cause jamming of the TBM, especially in the event of convergence resulting from high stresses (see paragraph 1, "Natural constraints").

In the case of tunnel reamers and open-face or mechanical-support TBMs, this criterion affects the appreciation of the risks of cutter head or shield jamming.

In the case of excessively deformable material, the design of TBM gripper pads will have to be studied carefully. The deformability of the surrounding ground also affects TBM guidance. If the tunnel lining is erected to the rear of the tailskin, attention should be paid to the risk of deferred deformation.

In ground that swells in contact with water, the resulting difficulties for advancing the machine are comparable for both slurry shield and EPB machines, in so far as the swelling is due to the diffusion and absorption of water within the decompressed ground around the tunnel. Compressed-air TBMs are less sensitive to this phenomenon.

3.3 - Liquefaction potential

Not applicable, except if there is a risk of earthquake or if the ground is particularly sensitive (saturated sand, etc.).

4 - HYDROGEOLOGICAL PARAMETERS

The purpose of examining the hydrogeological parameters of the terrain is to ensure that it will remain stable in the short term. The presence of high water pressures and/or potential inflow rates entraining material will prohibit the use of boom-type machines and open-face or mechanical-support machines unless accompanying measures such as ground improvement, groundwater lowering, etc. are carried out.

Water pressure is also decisive when geological accidents (e.g. mylonite) have to be crossed, irrespective of whether or not they are infilled with loose soil.

Ground permeability and hydrostatic pressure are decisive for TBMs using compressed-air, slurry, or EPB confinement. Compressed-air machines may even be rejected because of these factors, and they are particularly decisive for EPBMs when there are likely to be sudden variations in permeability. For slurry shield TBMs, the effects of these parameters are attenuated by the fact that a fluid is used for mucking out.

5 - OTHER PARAMETERS

5.1 - Abrasiveness - Hardness

Excessively high abrasiveness and hardness make it impossible or uneconomic to use boom-type tunnelling machines.

Abrasiveness and hardness can be decisive with respect to tool wear, the structure of the cutter head, and extraction systems (screw conveyor, slurry pipes, etc.). However, the expected wear can be countered by using boring and/or extraction additives and/or protection or reinforcement on sensitive parts.

5.2 - Sticking - Clogging

When the potential the material to be excavated has to stick or clog is known, the cutters of boom-type units, tunnel reamers, or shield TBMs can be adapted or use of an additive envisaged.

This parameter alone cannot exclude a type of shield TBM; it is therefore not decisive for face-confinement shields. However, the trend for the ground to stick must be examined with respect to the development of additives (foam, admixtures, etc.) and the design of the equipment for churning and mixing the sticky spoil (agitators, jetting, etc.).

The transport of muck by trains and/or conveyors is particularly sensitive to this parameter.

5.3 - Ground/machine friction

For shield TBMs the problem of ground friction on the shield can be critical in ground where convergence is high.

Where there is a real risk of TBM jamming (convergence, swelling, dilitancy, etc.) this parameter has a particularly important effect on the design of the shield.

The lubrication provided by their bentonite slurry makes slurry shield TBMs less susceptible to the problems of ground/machine friction.

5.4 - Presence of gas

The presence of gas in the ground can determine the equipment fitted to the machine.

6 - PROJECT CHARACTERISTICS

6.1 - Dimensions and sections

Boom-type units can excavate tunnels of any shape and sectional area. Shield TBMs, main-beam machines, and reamers can excavate tunnels of constant shape only. The sectional area that can be excavated is related to the stability of the face.

The sectional area of tunnels is decisive for large-diameter EPBMs (power required at the cutter head).

The length of the project can have an effect on slurry shield TBMs (pumping distance).

6.2 - Vertical alignment

The limits imposed on tunnelling machines by the vertical profile are generally those of the associated logistics. Main-beam tunnel boring and reaming machines can be adapted to bore inclined tunnels, but the requirement for special equipment takes them beyond the scope of these recommendations.

With boom-type units and open-face or mechanical-support TBMs, water inflow can cause problems in downgrade drives.

6.3 - Horizontal alignment

❑ The use of boom-type units imposes no particular constraints.

❑ The use of main-beam tunnel boring and reaming machines and of shield TBMs is limited to certain radii of curvature (even with articulations on the machines).

❑ With shield TBMs the alignment after/before break-ins and breakouts should be straight for at least twice the length of the shield (since it is impossible to steer the machine when it is on its slide cradle).

6.4 - Environment

6.4.1 - Sensitivity to settlement

Since boom-type units, tunnel reamers, main-beam TBMs, and open-face shield TBMs do not generally provide any immediate support, they can engender settlement at the surface. Settlement will be particularly decisive in urban or sensitive zones (transits below routes of communication such as railways, pipelines, etc.).

Sensitivity to settlement is generally decisive for all TBM types and can lead to exclusion of a given technique.

Open-face or mechanical-support shield TBMs are not suitable for use in very deformable ground. If the tunnel lining is erected to the rear of the tailskin, attention should be paid to the risk of deferred deformation of the surrounding ground.

With confinement-type TBMs, control of settlement is closely linked to that of confinement pressure.

With compressed-air shields the risk of settlement lies in loss of air (sudden or gradual).

With slurry shield TBMs the risk lies in the quality of the cake and in the regulation of the pressure. In relation to this, the "air bubble" confinement pressure regulation system performs particularly well.

With EPBMs the risk lies in less precise regulation of the confinement pressure. Moreover, the annular space around the shield is not properly confined, unless arrangements are made to inject slurry through the cans.

6.4.2 - Sensitivity to disturbance and work constraints

Slurry shield machines require a large area at the surface for the slurry separation plant. This constraint can have an effect on the choice of TBM type or even be decisive in intensively built-up zones.

The additives introduced into the cutterhead chamber of shield TBMs (bentonite, polymer, surfactant, etc.) may imply constraints on disposal of spoil.

6.5 - Anomalies in ground

6.5.1 - Ground/accident heterogeneity

Mixed hard rock/soft ground generally implies face-stability and gripping problems for tunnelling techniques with no confinement, and also introduces a risk of caving-in of the roof where the ground is softest.

6.5.2 - Natural and artificial obstacles

For "open" techniques it is essential to be able to detect geological accidents. For confinement techniques attention should be paid to the presence of obstacles, whether natural or artificial. Obstacles can have an effect on the choice of machine, depending on the difficulties encountered in overcoming the obstacle and the need to work from the cutterhead chamber.

Compressed-air work necessary for detecting and dealing with obstacles requires replacement of the products in the cutterhead chamber (products depending on the confinement method) with compressed air.

The work required for replacing them is:

- ❑ faster and simpler with a compressed-air TBM (in principle)
- ❑ easy with a slurry shield TBM
- ❑ longer and more difficult with an earth pressure balance machine (extraction of the earth and substitution with slurry to form a sealing film, followed by removal of the bulk of the slurry and replacement with compressed air).

6.5.3 - Voids

Depending on their size, the presence of voids can engender very substantial deviation from the design trajectory, especially vertically. They can also be a source of disturbance to the confinement pressure, particularly with compressed-air or slurry shield TBMs.

APPENDIX 3

Ground classification table (cf. GT7)

Catégory	Description	Examples	RC (Mpa)
R1	Very strong rock	Strong quartzite and basalt	>200
R2a	Strong rock	Very strong granite, porphyry, very strong sandstone and limestone	200 à 120
R2b		Granite, very resistant or slightly dolomitized sandstone and limestone, marble, dolomite, compact conglomerate	120 à 60
R3a	Moderately strong rock	Ordinary sandstone, siliceous schist or schistose sandstone, gneiss	60 à 40
R3b		Clayey schist, moderately strong sandstone and limestone, compact marl, poorly cemented conglomerate	40 à 20
R4	Low strength rock	Schist or soft or highly cracked limestone, gypsum, highly cracked or marly sandstone, puddingstone, chalk	20 à 6
R5 a	Very low strength rock and consolidated cohesive soils	Sandy or clayey marls, marly sand, gypsum or weathered chalk	6 à 0,5
R5b		Gravelly alluvium, normally consolidated clayey sand	<0,5
R6a	Plastic or slightly consolidated soils	Weathered marl, plain clay, clayey sand, fine loam	
R6b		Peat, silt and little consolidated mud, fine non-cohesive sand	

APPENDIX 3

Mechanized tunnelling data sheets (up to 31/12/99).

N° fiche	Project	*Project type (AITES)	TOS issue**	Date	Bored length (m)	Bored diameter (m)	Geology	
1	Echaillon	D	68	1972-1973	4362	5.80	Gneiss, flysch, limestone	Wirth
2	La Coche	D	77	1972-1973	5287	3.00	Limestone, sandstone, breccia	Robbins
3	CERN SPS	H	64	1973-1974	6551	4.80	Molasse	Robbins
4	RER Châtelet-Gare de Lyon	C	64	1973-1975	5100	7.00	Limestone	Robbins
5	Belledonne	D	64	1974-1978	9998	5.88	Schist, sedimentary granite	Wirth
6	Bramefarine	D	67	1975-1977	3700	8.10	Limestone, schist	Robbins
7	Lyons metro - Crémaillère	C	64	1976	220	3.08	Gneiss, granite	Wirth
8	Galerie du Bourget	C	67	1976-1978	4845	6 m2	Limestone, molasse	Alpine
9	Monaco - Service tunnel	H	64	1977	913	3.30	Limestone, marne	Robbins
10	Grand Maison - Eau Dolle	D	64	1978	839	3.60	Gneiss, schist, dolomite	Wirth
11	Western Oslofjord	G	77	1978-1984	10500	3.00	Slate, limestone, igneous rock	Bouygues
12	Brevon	D	66	1979-1981	4150	3.00	Limestone, dolomite, other calcareous rock (malm)	Bouygues
13	Grand Maison (penstocks and service shaft)	D	75	1979-1982	5466	3.60	Gneiss, schist	Wirth
14	Marignan aqueduct	F	66	1979-1980	480	5.52 m2	Limestone	Alpine
15	Super Bissorte	D	73	1980-1981	2975	3.60	Schist, sandstone	Wirth
16	Pouget	D	66	1980-1981	3999	5.05	Gneiss	Wirth
17	Grand Maison - Vaujany	D	75	1981-1983	5400	7.70	Liptinite, gneiss, amphibolite	Robbins
18	Vieux Pré	D	68	1981-1982	1257	2.90	Sandstone, conglomeratee	Bouygues
19	Haute Romanche Tunnel	D	73	1981-1982	2860	3.60	Limestone, schist, crystalline sandstone	Wirth
20	Cilaos	F	80	1982-1984	5701	3.00	Basalt, tuff	Wirth
21	Monaco - tunnel No. 6	A	66	1982	183	5.05	Limestone, dolomite	Wirth
22	Ferrières	D	79	1982-1985	4313	5.90	Schist, gneiss	Wirth
23	Durolle	D	79	1983-1984	2139	3.40	Granite, quartz, microgranite	Wirth
24	Montfermy	D	80	1983-1985	5040	3.55	Gneiss, anatexite, granite	Robbins
25	CERN LEP (machines 1 and 2)	H	82	1985-1986	14680	4.50	Molasse	Wirth
26	CERN LEP (machine 3)	H	82	1985-1987	4706	4.50	Molasse	Wirth
27	Val d'Isère funicular	B	97	1986	1689	4.20	Limestone, dolomite, cargneule (cellular dolomite)	Wirth
28	Calavon and Luberon	F	97	1987-1988	2787	3.40	Limestone	Wirth
29	Takamaka II	D	101	1985-1987	4803	3.20	Basalt, tuff, agglomerates	Bouygues
30	Oued Lakhdar	D	101	1986-1987	6394	4.56 / 4.80	Limestone, sandstone, marl	Wirth
31	Paluel nuclear power plant	E	105	1980-1982	2427	5.00	Chalk	Zokor
32	Penly nuclear power plant	E	105	1986-1988	2510	5.15	Clay	Zokor
33	Lyons river crossing - metro line D	C	106	1984-1987	2 x 1230	6.50	Recent alluvium and granitic sand	Bade
34	Lille metro, line 1b - Package 8	C	106	1986-1987	1000	7.65	White chalk and flint	FCB/Kawasaki
35	Lille metro, Line 1b - Package 3	C	106	1986-1988	3259	7.70	Clayey sand and silt	Herrenknecht
36	Villejust tunnel	B	106	1986-1988	4805 + 4798	9.25	Fontainebleau sand	Bade/Theelen (2 machines)
37	Bordeaux: Cauderan-Naujac	G	106	1986-1988	1936	5.02	Sand, marl and limestone	Bessac
38	Caracas metro: package PS 01	C	107	1986-1987	2 x 1564	5.70	Silty-sandy alluvium, gravel, and clay	Lovat
39	Caracas metro: package CP 03	C	107	1987	2 x 2131	5.70	Weathered micaschist and silty sand	Lovat
40	Caracas metro: package CP 04	C	107	1987-1988	2 x 714	5.70	Micaschist	Lovat
41	Singapore metro: package 106	C	107	1985-1986	2600	5.89	Sandstone, marl and clay	Grosvenor
42	Bordeaux: "boulevards" main sewers Ø3800	G	113	1989-1990	1461	4.36	Karstic limestone and alluvium	Bessac
43	Bordeaux: Avenue de la Libération Ø2200	G	113	1988-1989	918	2.95	Karstic limestone and alluvium	Bessac
44	St Maur-Créteil, section 2	G	113	1988-1990	1530	3.35	Old alluvium and boulders	FCB
45	Crosne-Villeneuve St Georges	G	113	1988-1990	911	2.58	Weathered marl and indurated limestone	Howden
46	Channel Tunnel T1	B	114	1988-1990	15618	5.77	Blue chalk	Robbins
47	Channel Tunnel T2-T3	B	114	1988-1991	20009 +18860	8.78	Blue chalk	Robbins/ Kawasaki

*AITES classification of project types

A road tunnels - B rail tunnels - C metros - D hydropower tunnels - E nuclear and fossil-fuel power plant tunnels - F water tunnels - G sewers- H service tunnels - I access inclines - J underground storage facilities - K mines -

(APPENDIX 3)

N° fiche	Project	*Project type (AITES)	TOS issue**	Date	Bored length (m)	Bored diameter (m)	Geology	
48	Channel Tunnel T4	B	114	1988-1989	3162	5.61	Grey and white chalk	Mitsubishi
49	Channel Tunnel T5-T6	B	114	1988 -1990	2 x 3265	8.64	Grey and white chalk	Mitsubishi
50	Sèvres - Achères: Package 3	G	121	1989 -1991	3550	4.05	Coarse limestone, sand, upper Landenian clay (fausses glaises), plastic clay, Montian marl, chalk	Herrenknecht
51	Sèvres - Achères: Packages 4 and 5	G	121	1988 -1990	3312	4.8	Sand, upper Landenian clay (fausses glaises), plastic clay, Montian marl and limestone, chalk	Lovat
53	Orly Val: Package 2	C	124	1989 - 1990	1160	7.64	Marl with beds of gypsum	Howden
54	Bordeaux Caudéran - Naujac Rue de la Liberté	G	126	1991	150	3.84	Karstic limestone	Bessac
55	Bordeaux Amont Taudin	G	126	1991	500	2.88	Alluvium and karstic limestone	Howden
56	Rouen "Métrobus"	C	126	1993	800	8.33	Black clay, middle Albian sand and Gault clay	Herrenknecht
57	Toulouse metro: Package 3	C	131	1989 -1991	3150	7.65	Clayey-sandy molasse and beds of sandstone	FCB / Kawasaki
58	Toulouse metro: Packages 4 and 5	C	131	1990 -1991	1587 +1487	5.6	Molasse	Lovat
59	Lille metro: Line 2 Package 1	C	132	1992 - 1994	5043	7.65	Flanders clay	FCB
60	Lille metro: Line 2 Section b	C	132	1992 - 1993	1473	7.65	Chalk, clay, and sandy chalk	FCB
61	St Maur: VL3c main sewer	G	133	1992 - 1994	1350	3.5	Very heterogenous plastic clay, sand, coarse limestone,andupper Landenian clay	Herrenknecht
62	Lyons metro: Line D Vaise - Gorge de Loup	C	133	1993 - 1995	2 x 875	6.27	Sand, gravel, and clayey silt	Herrenknecht
63	METEOR Line 14	C	142	1993 - 1995	4500	8.61	Sand, limestone, marl,upper Lutetian marl/limestone (caillasses)	HDW
64	RER Line D Chatelet / Gare de Lyon	C	142	1993 - 1994	2 x 1600	7.08	Coarse limestone	Lovat
65	Cleuson Dixence Package D Inclined shaft	D	142	1994 - 1996	2300	4.77	Limestone, quartzites, schist, sandstone	Robbins
66	Cleuson Dixence Inclined shaft	D	142	1994 - 1996	400	4.4	Limestone, schist, sandstone	Lovat
67	Cleuson Dixence Package B Headrace tunnel	D	153	1994 - 1996	7400	5.6	Schist and gneiss	Wirth
68	Cleuson Dixence Package C Headrace tunnel	D	152	1994 - 1996	7400	5.8	Schist, micachist, gneiss, and quartzite	Robbins
69	EOLE	B	146	1993 - 1996	2 x 1700	7.4	Sands, marl and 'caillasse' marl/limestone, sandstone and limestone	Voest Alpine
70	South-east plateau outfall sewer (EPSE)	G	146	1994 - 1997	3925	4.42	Molasse sand, moraine, alluvium	NFM
71	Cadiz: Galerie Guadiaro Majaceite	F	148	1995 - 1997	12200	4.88	Limestone, consolidated clay	NFM/MHI
72	Lille metro Line 2 Package 2	C	148	1995 - 1997	3962	7.68	Flanders clay	FCB
73	North Lyons bypass, Caluire tunnel, North tube	A	150	1994 - 1996	3252	11.02	Gneiss, molasse, sands and conglomerate	NFM
74	North Lyons bypass, Caluire tunnel, South tube	A	150	1997 - 1998	3250	11.02	Gneiss, molasse, sand, and conglomerate	NFM
75	Storebaelt rail tunnels	B	150	1990 - 1995	14824	8.78	Clay and marl	Howden
76	Strasbourg tram line	C	150	1992 - 1993	1198	8.3	Sands and graviers	Herrenknecht
77	Thiais main sewer Package 1	G	154	1987 - 1989	4404	2.84	Marl and clay	Lovat
78	Antony urban area main sewer	G	154	1989	1483	2.84	Alluvium, limestone, marl	Lovat
79	Fresnes transit	G	154	1991	280	2.84	Marl and alluvium	Lovat
80	Main sewer beneath CD 67 road in Antony	G	154	1991	670	2.84	Marl	Lovat
81	Duplication of main sewer, Rue de la Barre in Enghien	G	154	1992 - 1993	807	2.84	Sand, marly limestone, marl	Lovat
82	Bièvre interceptor	G	154	1993	1000	2.84	Marl and alluvium	Lovat
83	Duplication of main sewer, Ru des Espérances - 8th tranche	G	156	1993 - 1994	1387	2.54	Limestone, sand	Lovat
84	Duplication of main sewer, Ru des Espérances - 9th tranche	G	156	1995 - 1996	1200	2.54	Coarse limestone, marly limestone	Lovat
85	Duplication of main sewer, Ru des Espérances - 10th tranche	G	156	1996 - 1997	469	2.54	Marly limestone	Lovat

Drilling and Blasting – An Overview

(Includes: Operation, Optimization, Production & Productivity, Safety Measures and Emerging Techniques)

Introduction

Blasting is an important phenomenon for extraction of minerals where massive amount of explosives are detonated in shot holes to break the surrounding rock and minerals. The undesirable side-effects of blasting are ground vibration, noise/air overpressure, flyrock, dust, fumes etc., which cause annoyance to the nearby residents. The annoying circumstances as a result of these impacts, at many times, lead to unfavorable mining operations, confrontations and sometimes even litigations. In order to maintain a balance between the safety and production, a blast design has to take into account these undesirable side-effects. This paper deals with the optimization of various blast design parameters for improved production and productivity with greater safety suitable for Indian geo-mining conditions.

Dominating Parameters for Optimum Blasting

In blasting operations, huge amount of high-strength explosives are used to break the surrounding rocks and minerals. The interaction of explosive energy with the surrounding rock mass is a complex phenomenon. Before making any fruitful blast design applicable for particular rock strength, the following dominating parameters should be kept in mind.

- Rock properties
- Explosive properties
- Blasthole geometry
- Drilling accuracy
- Blasting techniques
- Production requirements
- Safety

From operational point of view, an optimum blasting may further be classified into following sub-items for their specific inclusion in any kind of model or design analysis (Figure-1). These parameters may be used, as a whole or part, to define any vibration, fragmentation, noise, blast design, blast failure and cost analyses.

Deign parameters for optimum blasting

Rock parameters	*Explosive parameters*	*Design parameters*
1. P-wave velocity (m/s)	1. Explosive brand and type	1. Burden (m)
2. Density (g/cc)	2. Density (g/cc)	2. Spacing (m)
3. Joint spacing (m)	3. RWS/RVS	3. Hole depth (m)
4. Dip/Azimuth (degree)	4. Explosive amount (kg)	4. Hole diameter (mm)
5. Compressive st. (MPa)	5. Charge diameter (mm)	5. Charge diameter (mm)
6. Tensile st. (MPa)	6. VOD (m/s)	6. No. of rows
		7. Initiation pattern
		8. Delay sequence
		9. Delay interval (ms)

Fig. 1: Design parameters of operational blasting

Blast Induced Hazards

Increased mineral consumption and environmental concern have caused great attention to safe blasting practices in mining sector with a view to avoiding the continuous confrontation, compensation, litigation and political embarrassment. There are four different possible causes of annoyances, which may create uneven post-blast disturbances. These are as follows-

- Ground vibration
- Flyrock
- Noise/air overpressure
- Human response

The psychological response, particularly the fear (real or imaginary), for structural damage to the persons residing in the vicinity of any mining operation, is undoubtedly an important additional factor.

Optimum Blast Design Parameters

Blasthole Diameter

The blasthole diameter should be determined in view of the bench height of the mine, geological conditions of the rock, explosive types to be used, capacity of shovels or other heavy earth moving machinery and nearness of the residential area to the proposed blasting site. In environmentally sensitive are, it is advisable to use small diameter hole. 100 mm diameter is preferred.

Burden

Burden is defined as the shortest distance to relief at the time of detonation of holes. In any blasting operation, it is the most critical parameter. If burden is very less, the rock will be thrown to a considerable distance from the blast face, airblast level will be very high and excessive fines will be produced. Too much burden will produce severe backbreak and shattering on the back wall. Hence, the burden should be optimized correctly. The following two different formulae (Konya and Walter, 1990; Pal Roy, 2005) may be used to estimate the burden.

$$B = [(2\ SG_e/SG_r + 1.5)] \times D_e \qquad \text{....(1)}$$

Where, B = Burden in ft.

SG_e = specific gravity of explosives

SG_r = specific gravity of rock

D_e = Dia. of explosive in inch.

$$B = Hx\frac{D_e}{D_h}x\frac{5.93}{RQD} + 0.37\left[\frac{L}{C}\right]^{0.5} \qquad \text{....(2)}$$

Where, B = Burden (m), H = Bench height (m); D_e = Diameter of explosive (mm); D_h = Diameter of blasthole (mm); RQD = Rock Quality Designation; L = Loading density of explosive (kg/m) and C = Charge factor (kg/m^3).

For third and subsequent rows, the correction factor of 0.9 must be applied.

Spacing

The spacing may be defined as the distance between the boreholes or charges in the same row. Spacing (S in m) should be determined in relation to burden (B in m), joints of rocks and hole depth.

$$\text{Spacing, } S = (1.2 - 1.5)\,B \qquad \text{....(3)}$$

In some opencast mines, it has been noticed that S/B ratio up to 2 also worked well.

Stemming

The following relations between stemming and burden are found to be most suitable for fruitful blasting operations.

(a) St/B > 0.6 (for controlling flyrock)

(b) St = B (for symmetrical stress balance)

Generally, drill cuttings are used as stemming material but it should be free from the pebbles and gravels. Stone chips of 3-4 mm sizes are also used to arrest flying fragments from the blasting face.

Hole Inclination and Drilling

In sloping and fractured benches, it is always advisable to drill holes making some angles (usually 5^0 to 10^0) with vertical and as much as to avoid some clearly visible fractured lines. However, the following guidelines may be used for better results.

(a) The holes should be as far as possible away from the fractured zone to avoid any venting of gaseous energy.
(b) In slope faces, the holes should be angular in accordance with the face.

Sub-drilling

Sub-drilling means to drill the blastholes beyond the planned grade lines or below the floor level. When sub-drilling is used there will be a larger zone of maximum tension and it will occur closure to the floor level, which must be sheared for good rock breakage.

The sub-drilling may be used as per the following norm.

Sub-drilling (J in m) = 0.1 – 0.25 times of the burden (m)

Delay Interval between Consecutive Rows

The delay blasting reduces a large blast to a series of smaller blasts by using millisecond delay caps. The following guidelines should be observed for fruitful blast results.

(a) The delay interval should be such that the burden from previously-fired holes has enough time to move out to make next row to have adequate relief. Insufficient relief will result in flyrock and back breaks. As a general guideline, the delay interval between consecutive rows may be chosen as 6-12 ms/m of the burden.
(b) Delay neither should be too short nor be too long which may lead to bad fragmentation. But, it should be optimum to get the desired results.

In the vicinity of residential structures where controlled blasting is required, the delay interval plays a very predominant role. With the optimum use of delay timings, the undesirable side effects of blasting can be controlled in these areas.

Blasting Side-effects and its Control Measures

Vibration, noise and flyrock problems create great socio-economic problems for the mine management as well as for the people residing in the vicinity of the mine. Such hazardous circumstances can be efficiently handled through pre-occupational planning and their subsequent implementation in order to enhance efficiency, economy and environmental safety. Modern opencast mines are in general larger and deeper than the earlier operations, giving rise to a higher percentage of overburden requiring greater blasting in the area. Solutions to these problems call for in-depth understanding of the basic parameters involved in the process of blasting, to ascertain the basic features of generation, propagation and prediction of ground movements.

Monitoring of Blast Vibrations

A provocative environmental approach is needed to ensure that cracking and other damages do not result from blast vibrations and to reduce complaints and damage claims. Recommended, international guidelines include the following-

1. Design for a safe level, either an industry standard or a regulatory limit.
2. Prepare a site map.
3. Conduct an information and site-relation programmme for neighbours, prior to blast.
4. Perform pre-blast inspections and document the condition of nearby properties.
5. Monitor with seismographs or use an alternative such as scaled distance.
6. Develop a site-programme plot and update regularly.
7. Using the propagation measurements, track trends in PPVs and frequencies.
8. Conduct post-blast inspections of structures to document any changes.

Table-1 compiles worldwide blast vibration limits. While framing such standards the concept of resonant frequency was given due consideration. It is also an entrenched fact that the ground vibration while propagating through a structure, its value is amplified, which is proportional to the height of the structure.

Table-1: Typical blasting limits by country (source: Pal Roy, 2005)

Measure	*Units*	*Comments*
Human perception	0.15 - 1.5 mm/s	
Visible damage	50 mm/s	Values in excess cause appreciable structural damage
British Standard BS 64722.1992	8.5 - 12.7 mm/s	90 per cent confidence limit- permissible impulsive vibration at residential property
Leicestershire County Council (UK)	6 mm/s	95 per cent confidence level- part of conditions covering blasting within modern planning permissions
Australian Standard Explosives Code AS2187-1993	5 mm/s	Common environmental limit (EPA)- depends on administering authority
(AS2187.2-2006)	0.2 mm/s	Historical buildings and monuments- displacement for frequencies less than 15 Hz
	19 mm/s	Houses and low rise residential buildings- resultant PPV for frequencies greater than 15 Hz
	25 mm/s	Commercial limits AS 2187.3
India (DGMS)	5 mm/s	Domestic houses/structures- frequencies less than 8 Hz
	10 mm/s	Domestic houses/structures (Kuchcha, brick & cement): frequencies lie between 8-25 Hz
German Standard DIN 4150 (GIS.1986)	5 mm/s	Domestic houses/structures- frequencies less than 10 Hz
	5 - 15 mm/s	Domestic houses/structures- frequencies 20 to 40 Hz
	15 - 20 mm/s	Domestic houses/structures- frequencies 50 to 100 Hz
Hungarian Standard	5 mm/s	Panel houses
Swiss Standard	8 - 12 mm/s	Objects of historic interest or other sensitive structure- frequency bandwidth: 60 -90 Hz
Swedish Standard National Museums	25 mm/s	Building structure
	5 mm/s	Sensitive exhibits

The Directorate General of Mines Safety (DGMS), India has stipulated the threshold values of vibrations for the safety of residential and other structures for different frequency levels, which is given in **Table-2**. When the chosen "safe limit" is an appropriate one and the other steps are carried out properly, it should be sufficient to ensure protection of homes and important establishments near mining operations.

Table-2: DGMS prescribed permissible limit of ground vibrations (in mm/s)
(Technical Circular No. 07, 1997)

Type of structure	*Dominant excitation frequency, Hz*		
	< 8 Hz	*8-25 Hz*	*>25 Hz*
(A) Buildings/structures not belonging to the owner			
Domestic houses/structures (Kuchcha, brick & cement)	5	10	15
Industrial buildings	10	20	25
Objects of historical importance and sensitive structures	2	5	10
(B) Buildings/structures belonging to owner with limited span of life			
Domestic houses/structures	10	15	25
Industrial buildings	15	25	50

Low Frequency Response

USBM studies led to the conclusion that there was little additional risk to residences from low frequencies. At exact resonance frequencies, a few of the highest amplifications exceeded the highest in RI 8507 (4.3x), which could justify some caution. Almost similar results were obtained by Indian researchers. Below the natural or resonant frequencies, structure responses decreased rapidly, particularly when based on different displacements. It is noted that the differential displacements are the vector differences between high and low responses which in turn determine strains.

However, that is not to say low frequencies are always harmless. For example, the September 19, 1985, earthquake waves that caused extensive damage in Mexico City were of amplitude 0.2 g at 0.5 Hz. At high frequencies, 0.2 g would only be marginally risky (e.g. 15 mm/s PPV at 20 Hz.). However, because the dominant wave frequency was 0.5 Hz, that vibration had a peak velocity of 624 mm/s and, even worse, a peak displacement of 20 cm.

DGMS Regulations

1. **Use of Ammonium Nitrate-Fuel Oil:** Where in any opencast workings a mixture of Ammonium Nitrate-Fuel Oil is used as explosive;

(a) The mixing or impregnating of ammonium nitrate or a non-explosive mixture of ammonium nitrate with other substances with fuel oil shall be done in a special shed in the vicinity of the mine workings constructed from incombustible material and equipped with remote control water sprays to combat a fire in an emergency.
(b) If mechanical mixer is used, the machine in which the mixing takes place, shall be of a type approved by the licensing authority under the Explosives Act, 1884.
(c) Smoking, open flames or any other source of fire shall not be allowed when mixing, carrying or handling operations are being carried on.
(d) Crude oil or crank case oil shall not be used for mixing with ammonium nitrate.

2. **Deep-hole blasting** - The blast holes made more than 3 m in depth may be called deep-holes and where deep-holes are blasted in an opencast mine, the following provisions shall apply:

(a) The position of every deep-hole to be drilled, shall be distinctly marked by the overman so as to be readily identified by the driller.
(b) As far as practicable, the shotfiring shall be carried out either between the shifts or during rest interval or at the end of the work for the day.
(c) All holes charged on any one day, shall be fired on the same day.
(d) During the approach and progress of an electric storm, no explosive or detonators shall be handled. If loading or charging operations have begun, the work shall be discontinued until the storm has passed.
(e) If the shots are to be fired electrically, all exposed wires shall be coiled up and if practicable, placed in the mouths of the holes or kept covered with non-metallic objects.

3. **Electric Shotfiring** – Where shots are fired electrically, the following provisions shall have effect, namely:

(a) No shot shall be fired except by means of a suitable shotfiring apparatus of a type approved by the Chief Inspector of Mines; and the number of shots fired at any one time by the apparatus shall not exceed the number for which it is designed.
(b) No current from a signaling, lighting or power circuit shall be used for firing shots.
(c) The shotfirer shall retain the key of the firing apparatus in his possession throughout his shift.
(d) After firing the shots and before entering the place of firing, disconnect the cable from the firing apparatus.
(e) The cable to the shotfiring apparatus shall be connected last; and detonators of the same electrical resistance shall only be used.

4. **Danger Zone:** No firing of shot-holes (i.e. blasting) should take place, unless sufficient warnings by efficient signals or other means approved by the manager, is given over the entire area falling within a radius of 500 m from the place of firing and also he has ensured that all persons within such area have taken proper shelter.

If any part of a public road or railway lies within the danger zone, unless two persons are posted, one in either direction at the two extreme points of such road or railway which fall within the danger zone, no blasting should be carried out before receiving proper clearance from the deputed persons.

In case of any permanent building or structure lies within the danger zone, the aggregate maximum charge per delay and per round shall be fixed by a scientific study and permission in writing for such blasting shall be obtained from the Chief Inspector of Mines. Protection against flying fragments or missiles, adequate shelter or other protection shall be taken during blasting operations.

Impact of Noise and Air Overpressure on Environment

Air overpressure (*AOP*) or air blast overpressure is the energy transmitted from the blast site within the atmosphere in the form of a series of pressure waves. This series of pressure waves is similar to a series of gusts of wind condensed into a very short period of time. As these waves pass a given position, the pressure of the air rises very rapidly then falls more slowly then returns to the ambient value after a number of oscillations. The pressure wave consists of both audible (noise) and inaudible (concussion) energy. The maximum excess pressure in this wave is known as the peak air overpressure, generally measured in decibels (dB) using the linear frequency weighting.

The different impacts of air overpressure and noise can be summarized as below.

1. *AOP* can cause direct damage at fairly high levels. 150 dB (L) will break a window. The Internationally accepted damage levels due to blast-induced air overpressure are given in **Table-3**.
2. The air overpressure may interact with structure causing the structure to vibrate with associated worries about damage.
3. Human response is the main important factor associated with blast-induced air overpressure.
4. Outside- *AOP* can be felt as a pressure change on the chest and Inside- *AOP* may cause windows and other loose objects to rattle that can startle the occupants.

Table-3: Internationally accepted threshold values of air overpressure

Overpressure (dB)	*Overpressure (KPa)*	*Air Blast Effects*
177	14.00	All windows break
170	6.00	Most windows break
150	0.63	Some windows break
140	0.20	Some plate glass windows may break and rattle
136	0.13	USBM interim limit for allowable air blast
126	0.05	Complaints likely

Factors Affecting Generation of Noise and Air Overpressure

Factors affecting the generation and propagation of air overpressure induced by blasting operations in mines and quarries are summarized below-

- The use of detonating cord which can produce high frequency and hence audible energy within the air overpressure spectrum.

- Stemming release, seen as a spout of material from the boreholes, gives rise to high frequency air overpressure.
- Gas venting through an excess of explosives leading to the escape of high-velocity gases, give rise to high frequency air overpressure.
- Reflection of stress waves at a free face without breakage or movement of the rock mass. In this case, the vertical component of the ground-vibration wave gives rise to a high frequency source.
- Physical movement of the rock mass, both around the boreholes and at any other free faces, which gives rise to both low and high frequency air overpressure.

Flyrock- Occurrence and Control Measures

The primary means of controlling flyrock is through proper blast design and delay timing. The possible causes of flyrock, which are commonly encountered in any bench blasting in surface mine are depicted in Figures-2A to 2G. The following points are very important to contain flyrock within permissible limits in any bench blasting.

(i) The consistency of burden, specially the front burden (distance between the first row to free face) must be maintained.

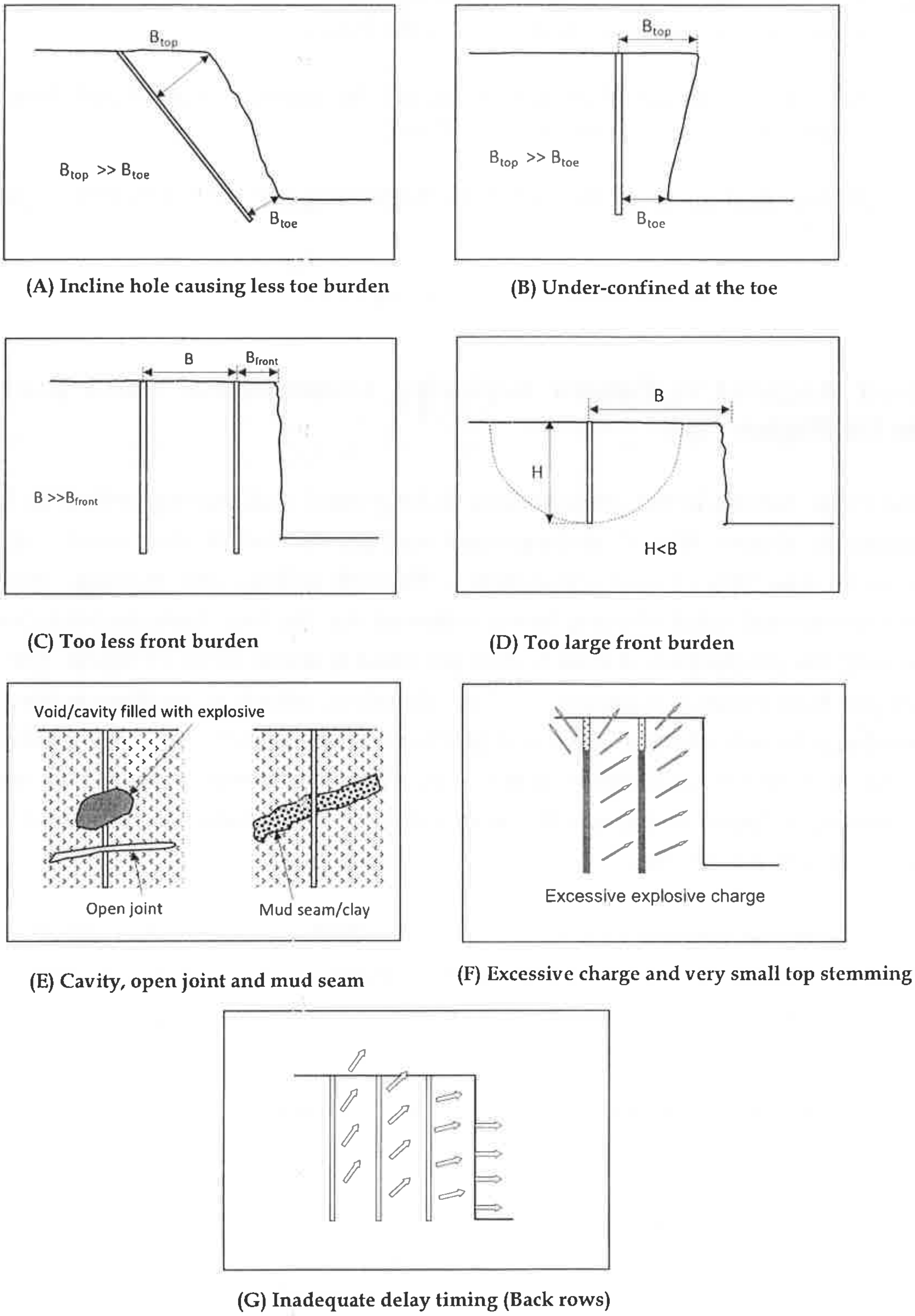
(ii) Bench height to burden ratio less than 1.5 should be avoided. The spacing of about 1.5 to a maximum of 3 times the burden is suggested to reduce the flyrock.

(iii) While loading a shot, the blaster must be aware of his true powder factor in terms of the amount of explosive to be charged for the quantity of rock to be fragmented. Charging of excessive explosive quantity must be avoided.

(iv) When firing more than one row of holes, sufficient delays should be used between two sequences of rows or holes. Based on the observations, delays of at least 42 ms should be used between two consecutive rows.

(v) Length of stemming column should be greater than or at least equal to 20-25 times the hole diameter.

(vi) The blasting site should always be inspected before marking the holes. If any open joints and bedding planes are present in the bench, an adjustment should be made in the drilling pattern.



(A) Incline hole causing less toe burden

(B) Under-confined at the toe

(C) Too less front burden

(D) Too large front burden

(E) Cavity, open joint and mud seam

(F) Excessive charge and very small top stemming

(G) Inadequate delay timing (Back rows)

Fig. 2: Common causes of flyrock in bench blasting in surface mine

(vii) Before loading, blasting officials should always check the hole depths and ensure that the holes are drilled as per the blast design.

(viii) Any change in the blast design should be carefully considered from the standpoint of its potential effect on flyrock.

(ix) All loosened pieces of the rock from the blasting site should be cleared before charging.

(x) Statutory provisions should be strictly implemented.

Modified Angled-Cut Pattern Replacing Conventional Solid Blasting Pattern for Higher Pull

Bord and Pillar mining is the predominant underground coal mining system in India, contributing to almost 98% of underground coal production in the country. In this system, more than 96% of coal production is through drilling and blasting. With the present conventional solid blasting being followed for the last three decades (wedge cut/fan cut); the production of coal (yield) per blast is about 10 to 16 tonne. The pull obtained per blast remains stagnant to 1.2 m. Therefore, efforts to mechanize bord and pillar workings by introducing SDLs and LHDs could not achieve expected production targets due to poor coal availability at the faces. Therefore, it was a great challenge for the coal mining industry to increase the availability of coal at faces for optimized use of mechanized loading machines.

Using Pentadyne-HP explosive, which has a gap sensitivity of 15 cm, extensive field trials were conducted at GDK-5 Incline of M/s Singareni Collieries Companies Ltd to achieve around 40 tonne of coal per blast (**Table-4**).

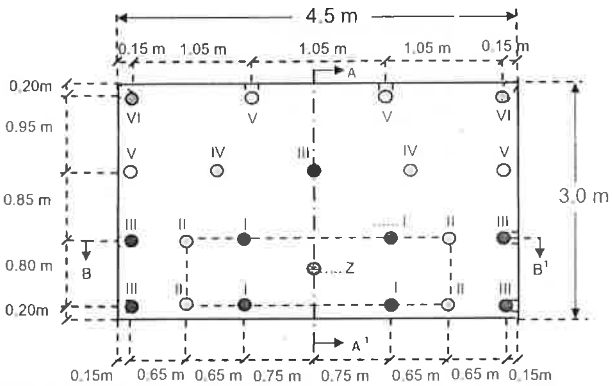
Table-4: GOCL-prescribed characteristics of Pentadyne-HP explosive*

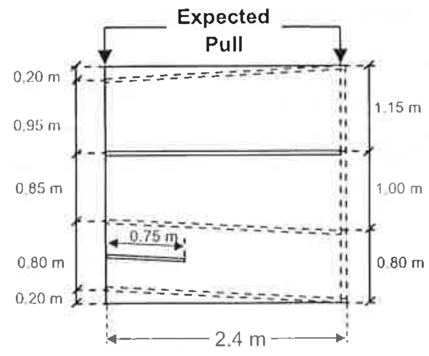
Sl. No.	*Weight of cartridge*	*200 gm*
1	i) Air Gap Sensitivity in Open ii)Air Gap Sensitivity (AGS) in 4 mm PVC pipe	16 cm 20 cm
2	Incendivity	Passed
3	Critical Diameter	22 mm
4	Density	1.08± 0.05 g/cc
5	Energy level with respect to ANFO	60-70%

6	i) Continuity of Detonation (COD) ii)COD with multiple spacer	Passed 16 cm
7	Velocity of Detonation	3700±500 m/s
8	Sensitivity	No. 6 strength detonator
9	Post-detonation Fumes (PDF)	Passed
10	Relative Weight Strength	51%
11	Shelf life	> 6 months with 15 cm AGS in open

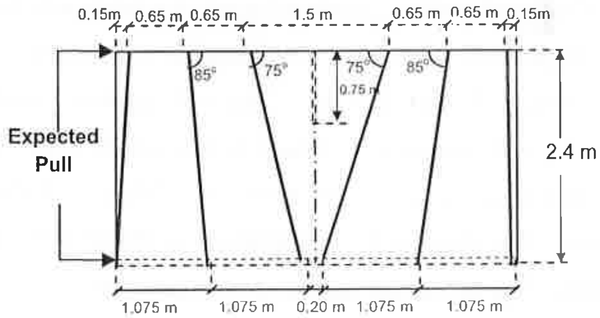
* Sarathy & Mishra (2009), GOCL - Personal Communications

Different patterns like Modified Gronlund Cut, Swedish Cut, and Sarrois Cut were tried and it was established that the blast results obtained with modified angled cut (wide V-cut pattern) were better than any other cuts (**Figure-3**). The pull obtained in modified V-cut with 2.4 m blasthole depth varied between 1.7 and 2.2 m. The calculated yield achieved in depillaring panel (4.0 m × 3.8 m face dimention) varied between 39 and 50 tonne of coal. In the development faces, smaller face dimensions were available and the yield per blast varied between 27 and 35 tonne. Powder factors obtained in the blasts of depillaring panel varied from 1.92 to 2.34 tonne/kg whereas in the development faces it varied between 1.84 and 1.91 tonne/kg. Detonator factors obtained in the blasts of depillaring panel varied between 1.62 and 2.13 tonne/detonator whereas in blasts of development faces lesser values were achieved i.e. 1.4 and 1.7 tonne/detonator.





Section- AA¹



Section- BB¹

Delay No.	*No. of hole*	*Hole depth*	*Charge/hole*	*Charge per delay*
Z	1	0.75 m	0.20 kg	0.20 kg
I	4	2.4 m	1.00 kg	4.00 kg
II	4	2.4 m	1.00 kg	4.00 kg
III	5	2.4 m	0.80 & 1.00 kg	4.40 kg
IV	2	2.4 m	0.80 kg	1.60 kg
V	4	2.4 m	0.80 kg	3.20 kg
VI	2	2.4 m	0.80 kg	1.60 kg
Total hole no. : 22			Total Charge = 19.00 kg	

Fig. 3: New blast design pattern with 2.4 m blasthole depth for development faces

It is anticipated that this particular development would fulfill the long-awaited requirement of the coal mining industry in India for mechanization of underground coal mines for improved production and productivity.

Line Drilling

Line drilling is the earliest controlled blasting method used. This is a method of controlling blast damage or backbreak by drilling a line of small diameter closely spaced holes at reduced burden back from the last row of main production blastholes in a blast. The shock energy from the main blast will be sufficient to cause inter-blasthole splitting between the individual holes which have been drilled in line. The purpose of line drilling is to create a plane of weakness by drilling closely spaced, small diameter holes along the perimeter of the excavation to which the blast can break. Contour holes are normally drilled with a centre to centre distance of 0.1 - 0.2 m or 3 - 4 blasthole diameter apart. Line drill holes are usually not over 75 mm in diameter and the spacing is 2 to 4 times the diameter of the hole. The hole depth should not be more than 12 m, since deviation in longer holes may produce adverse results. These holes are not charged.

Presplitting

This is the most successful and widely adopted controlled blasting method and creates a plane of shear on the desired line of break, exposing the half barrel of the blasthole after excavation. The holes are usually 50 mm to 100 mm in diameter in civil engineering applications whereas larger diameter holes (sometimes more than 300 mm in diameter) are proving to be successful in surface mining operations. The spacing between holes varies between ten to twenty times the hole diameter. The presplit shots transmit compressive shock waves, which, at their point of meeting between the holes, create a zone of tension, which fractures and shears the rock. This method has limited application in underground work.

Case Study at Midumkham Area of Mizoram State

At some locations in Midumkham area, rocks were soft as well as friable in nature. This often resulted in overbreak and it was difficult to achieve more than 60% half-cast factor. Half-cast factor is the percentage of the total casts (half of each borehole on the final wall) that are visible after the rock has been excavated. It is used to measure the quality of controlled blasting techniques. To obtain more half-cast factors as well as to minimize overbreak in softer rock formation, bamboo spacers were used in the perimeter holes. Bamboo spacers created air-vacuum inside the blastholes, distributing the explosion pressure more uniformly along the blastholes. Since blasthole diameter was 32 mm, smaller bamboos having diameter less than 25 mm were used. They were cut into pieces with length varying from one foot (30 cm) to three feet (90 cm) (**Figure 4**). Charging patterns of perimeter holes using bamboo spacers for 1.5 m and 2.44 m hole depth are shown in **Figure-5** whereas drilling and charging patterns of holes (5 ft) used for controlled blasting are shown in **Figures-6 & 7**.

Fig. 4: Bamboo spacers used for perimeter holes

Fig. 5: Charging of blastholes with bamboo spacers

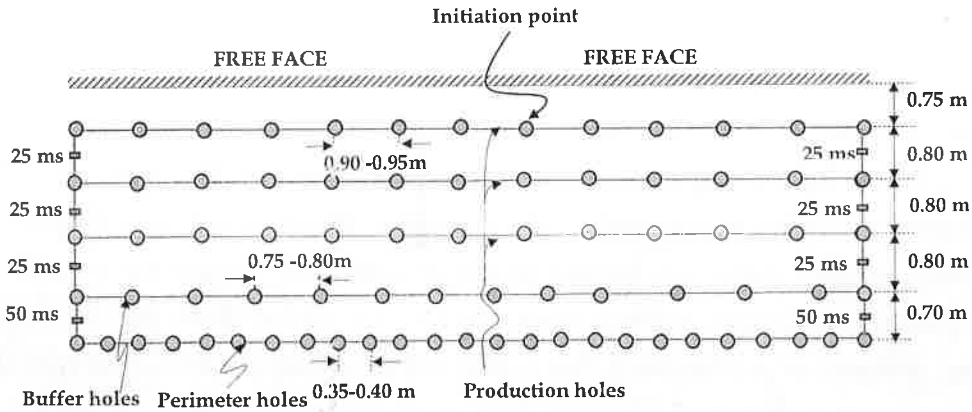


Fig. 6: Drilling and initiation pattern of holes (5 ft) used for controlled blasting at Midumkham Area

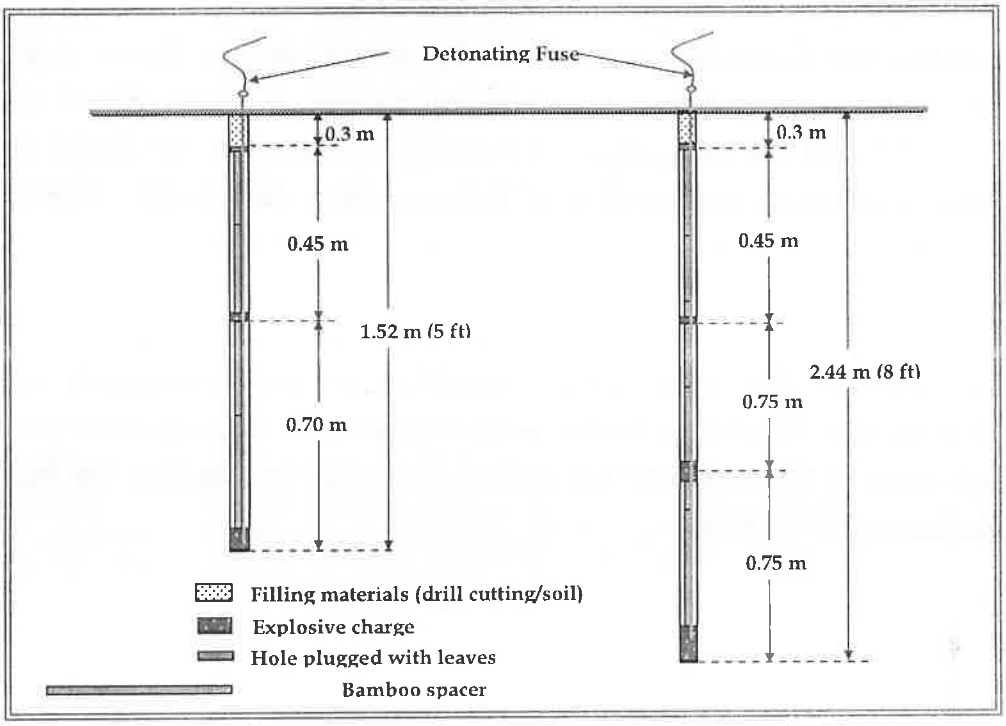


Fig. 7: Charging pattern of perimeter holes using bamboo

Bamboo spacers resulted in very good control on overbreak where more than 90% half-cast factors were achieved. The use of bamboo spacers also reduced the explosive charging time, thereby the overall progress was faster. These bamboos were easily available near the excavation site. The introduction of bamboo spacers resulted in beautiful final walls, ensuring the stability of the final walls (**Figures- 8 & 9**).

Fig. 8: Half-cast factors obtained with bamboo spacers

Fig. 9: Final wall obtained through controlled blasting using bamboo spacers

Perimeter Blasting

This type of blasting is generally used for underground blasting. However in surface blasting the same technique is known as 'smooth blasting'. In underground operations the perimeter holes of the backs (roof) of headings and tunnels are drilled along the design profile parallel to the direction of the excavation (**Figure-10**). Generally the spacing between the final lines of holes is less than 1.5 times the burden.

In surface mining the decoupled perimeter holes are drilled on a closer pattern than the production blastholes. These holes are detonated last, in order to maximize the relief of burden. This reduces overbreak. Unlike normal blasting in underground, the spacing (S) between holes in the same row for surface excavation is less than the burden (B). The usual relation is S = 0.8 x B.

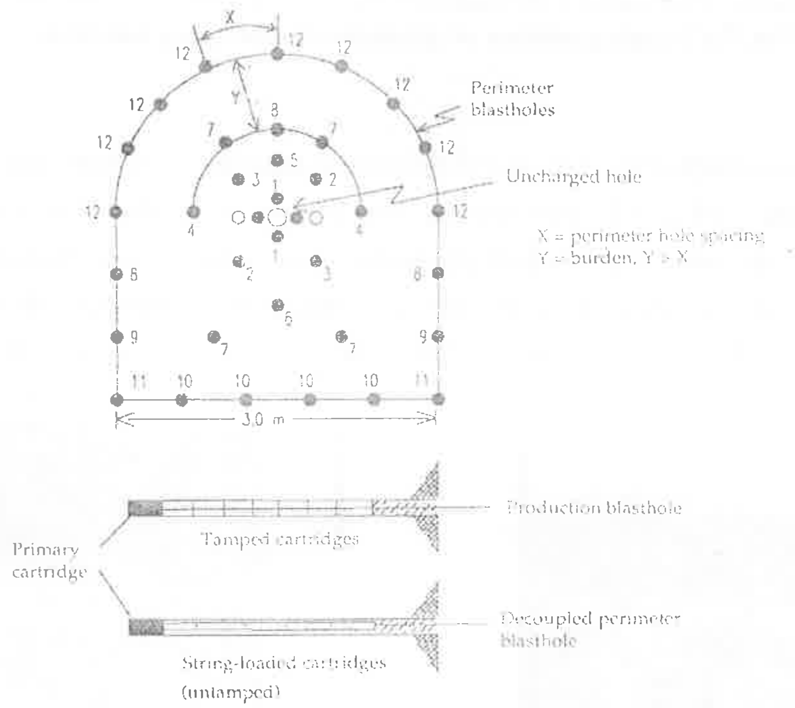


Fig. 10: View of a standard perimeter blasting arrangement

Cushion Blasting

Cushion blasting is applicable in surface mining where the object is to trim the excess material from the final highwall to improve stability. A single row of holes is drilled along the perimeter of the excavation. The size of the drillholes varies between 50 mm and 164 mm.

Cushion blastholes are charged with small, well distributed charges in completely stemmed holes, which are fired after the main blast is excavated. The charges are fired with no delay, or minimum delay between holes.

Effects of Opencast Blasting in U/G Workings

The junctions of the underground mine workings are more susceptible to produce cracking than the galleries away from the junctions. The threshold values of vibration at the junctions in terms of peak particle velocity are given in **Table-5** for different RMR (Rock Mass Rating) of roof rock for the safety of underground coal mine workings. The limiting values in the pillars are given in **Table-6** (DGMS Technical Circular No. 06 of 2007, Dated 28/05/2007).

Table-5: Threshold values of vibration (measured on roof) for the safety of roof in the underground workings for different RMR

RMR of roof rock	*Threshold values of vibration in terms of peak particle velocity (mm/s)*
20-30	50
30-40	50-70
40-50	70-100
50-60	100-120
60-80	120

Table-6: Threshold values of vibration (measured on pillars) for the safety of roof in the underground workings for different RMR

RMR of roof rock	*Threshold values of vibration in terms of peak particle velocity (mm/s)*
20-30	20
30-40	20-30
40-50	30-40
50-60	40-50
60-80	50

Summary Analysis of Vibration Risks in U/G Works

(*International Standard*)

Although, there is much variation between the structure and geologic conditions represented by several authors worldwide, the general observation is that major failure such as roof collapse and pillar failure would require vibrations greater than 300 mm/s. In some cases, loose pieces were dislodged even at lower vibration levels of about 12 to 30 mm/s. Low-level vibrations, certainly below 25 mm/s, have been found to be totally harmless to underground workings, even active ones where rockfalls are a personnel hazard (Siskind, 2000).

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Blasting Techniques to Reduce Insitu Rock Damage & Improving Highwall Stability

Blasting can be defined as conversion of solid rock by use of explosives to several smaller pieces capable of being moved by material handling equipment. The process of blasting rock, to facilitate its removal or to obtain crushed mineral/stone involves a series of steps in preparing the blast and a complicated sequence of events during and following detonation of an explosive. The process comprises of fragmentation and displacement or throw and both of these operations are controlled by the energy liberated from an explosive charge, the transfer of energy to the rock, and the rock properties

Preparation of blast begins with drilling of holes into the rock for placing explosives. The size and depth of holes, spacing of holes, their burden/relief (spacing with existing air/rock interface or free face that will subsequently be created by blasting of holes that have detonated earlier) against which the rock shall be broken, and their initiation have a bearing onto the results. Choice of explosives involves consideration of properties of the explosive as well as that of the rock being blasted. Explosives differ in density, rate of detonation, water resistivity and in energy they apply to the rock mass, and rocks' response to the explosives varies on a number of factors/inherit properties like joints, bedding planes, foliation, density, hardness, compressive & tensile strengths, and likewise.

An explosive has three basic characteristics – it is a chemical compound or mixture ignited by heat, shock, impact, friction or combination of these conditions, upon ignition, it decomposes very rapidly in detonation, and upon detonation, there is rapid release of heat (4500^{0}C) and large quantity of gases at high pressure (2,50,000 bars), which expand rapidly in the confined surrounding rock formation.

When an explosive charge is detonated, in a hole or number of holes, **only 20% of the energy is utilized in a properly designed and executed blast**. The energy of the explosives gets roughly distributed as -

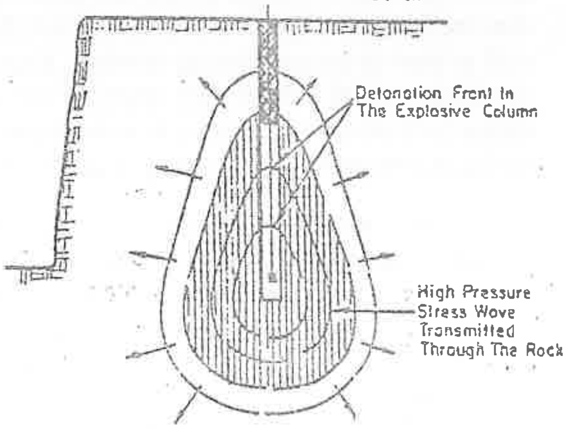
Crushing in the hole's vicinity	1.5-2.0%
Fracture insitu	<1%
Breakage	15%
Displacement	4%
Deformation of solid rock behind the hole	<1%
Fly rock	<1%
Ground vibration	40%
Air blast	38%

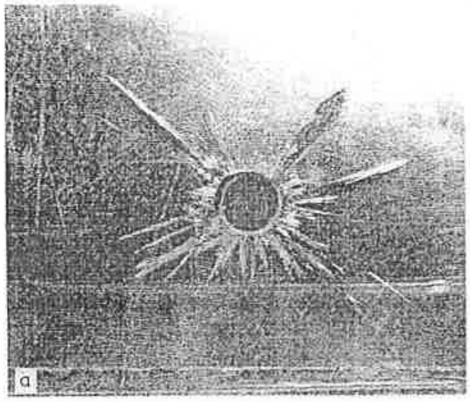
A blasting engineer's job is thus to ensure that the explosive energy is utilized to its fullest in properly fragmenting the rock and in its desired displacement, without causing damage to the in-situ rock or to the high-wall, or causing fly-rock, excessive ground vibrations and air-blast. One must remember that the undesirable outcomes of a blast, whether it is excessive ground vibration, back-break, fly-rock, air blast or damage to in-situ rock, are all inter-linked to each other Prevention of damage to in-situ rock gains utmost importance if the purpose of blasting is to make an excavation, whether in superjacent ground or on surface, as in case of adits, tunnels, pressure shafts, excavations to house generating/ transmitting equipment, dams etc., where the resultant rock-face is put to some end-use. The damage to in-situ rock can be classified into 3 zones – "***severe or over-break***" or complete detachment/ dislodgement from parent rock where the rock breakage is beyond the designed limit; "***crack widening zone***" where aperture of the existing cracks get widened due to expanding gases of explosive (rock mass may slab off in time), and "***incipient crack growth zone***" where fresh cracks are generated due to blast induced strain (not dislodging the rock mass immediately). In-bye of the incipient crack zone lies the "***intact zone***" where the rock mass strength is not reduced significantly.

It would be prudent to understand how explosive causes fragmentation. Theory of breakage involves two basic processes, viz., radial cracking and flexural (bending) rupture – the rock breaks in tension. The rock is far-far stronger in compression than in tension

When a cylindrical charge of explosives is fired, a sudden and quick release of high pressure takes place on detonation of an explosive charge. The rock over a small area immediate around the blast-hole gets pulverized. The quick release of high pressure also sends a compressive shock wave through the rock mass.

The radial compression induces tensile component in the tangential planes of the wave front. When the tangential strain exceeds dynamic tensile strength of the rock, radial cracking around the crushed zone is initiated. As rock is not homogeneous, the short lived compression waves do not develop much of radial cracks, but rather help in expansion of the existing cracks and directs the energy that follows. The wave also enlarges short radial cracks that radiate out from the blast-hole. The radial cracking propagates behind the compressive shock wave, and the number and length of cracks depend on the intensity of the shock wave, dynamic tensile strength of the rock and attenuation characteristics of the strain waves. The shock wave speed is a function of the density of the rock mass, i.e., the wave speed is faster in denser rocks. However, density is not the sole criteria for the velocity of wave propagation in the rock mass.

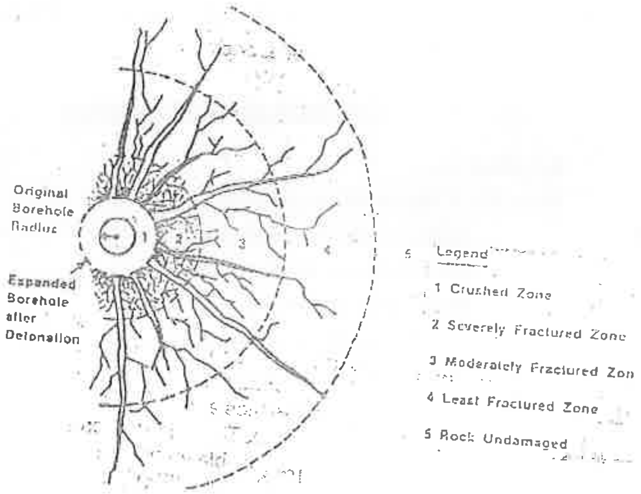


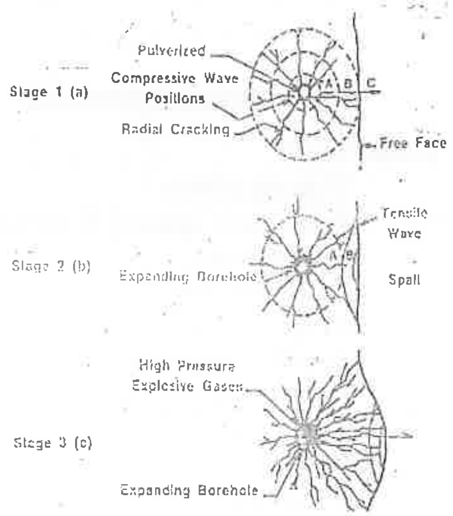


The compressive wave travels towards the nearest avail-able free face and is then reflected back towards the borehole as a tensile wave. The varying density of the rock mass and fissures/ cracks/joints in it also act as a free faces, returning a portion of the compressive shock wave to the blast hole, whist the remainder continues to travel towards the free face. For proper blasting and fragmentation results, the compressive shock wave must have sufficient intensity to travel to the free face and back, overcoming the dynamic tensile strength of the rock-mass through which it is propagating.

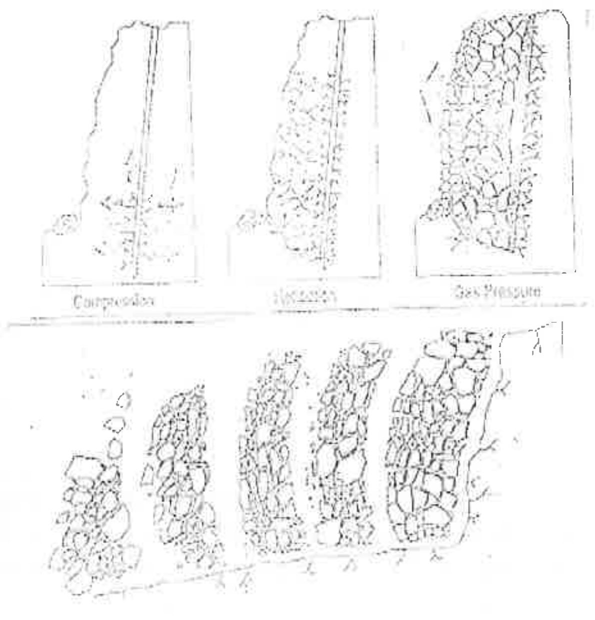
When the strain wave reaches the free face (interface between the rock & air), two waves are generated on reflection of the shock wave – tensile wave and a shear wave. The outermost edge of the rock mass cracks and breaks due to the reflected tensile wave. Phenomenon of spalling will occur if the tensile wave is strong enough to exceed the dynamic tensile strength of rock.

After formation of the radial cracks, the gases generated by detonation of explosive begin to act. The rapid expansion of gases in the blast-hole exerts pressure against the blast-hole walls, causing flexure or bending. The gas pressure drives the radial cracks through the burden to the free face and cause the rock to displace in the direction of least resistance. The gaseous pressure is thus responsible for fracture of rock in a direction perpendicular to the blast-hole axis.





It must be remembered that compressive strength of intact rock is 10-12 times greater than its tensile strength. Tensile failure of rock is thus the mechanism by which most new fractures are created during blasting.



The explosive column also makes the rock mass to act like a beam of thickness equal to burden, embedded at the bottom of blast hole on one end and in the stemming column on the other. Displacement is required in addition to cracking to break the beam. The gaseous pressure which starts exerting pressure after the explosive charge has been fired and radial cracks have been formed, as with a beam, produces fracturing and deformation as in the phenomenon of flexion.

The beam will have least resistance to bending at the centre of its span, but because the action starts at the location of the primer, the largest movement will be towards the primer side of the explosives column. The beam bends with the expansive force of gases, creating tensile stresses at the free face. The rock breaks at locations of weakness-planes either existing in the rock, or created due to radial cracks caused by the initial compressive wave.

The rock also breaks due to in-flight collision. As the rock fragments are projected towards the free face, they collide with each other, producing additional fracturing.

As also stated in preceding text, the nature of the rock, the type of explosive being used for blasting, the blast design, and its initiation have a definite bearing onto the blast results – fragmentation & displacement of rock, blast-induced in-situ rock damage and ground vibrations and fly rock. Whereas, parameters concerning the rock are fixed and are site specific, i.e., beyond the purview of control, the choice of explosive, blast geometry, and initiation of the blast, are partially or totally controllable, and have definite inter-relationship with each other. Again, rock massif not being homogenous, it is most difficult to be defined. The parameters of importance for rock blasting can be listed as under –

Fixed Parameters

Rock Massif
- Bedding planes
- Joints, their spacing & direction
- Joint fillings
- Discontinuities
- Crushed zones
- Weathering degree
- In-situ rock stresses

Rock Type
- Compressive strength
- Tensile Strength
- Young's Modulus
- Density
- Poisson's ratio
- Water content
- Internal friction & porosity
- P wave velocity

Controllable parameters

Explosives

Specific Charge/Energy content
- Impedence of explosive to rock
- Distribution in borehole
- Bore hole pressure
  - Explosive density
  - VOD of explosives
  - Charge diameter
  - Coupling with rock
  - Hole diameter
  - Explosive diameter
  - Type of explosive

Blast Geometry
- Alignment of blast-hole to free face
- Burden
- Spacing
- Bench Height
- Hole Inclination
- Number of rows in a round
- Pattern of holes (straight/staggered)
- Confinement
- Sub-drilling
- Stemming
- Initiation device & initiation pattern
- Delay time & number of delays
- Time scatter of delay caps
- Decking

We shall discuss the role of each parameter and their inter-relationship in blasting and as to how the explosive energy can be best utilized to its fullest in properly fragmenting the rock and give the desired displacement, without causing damage to the in-situ rock or to the high-wall, or causing fly-rock, excessive ground vibrations and air-blast. One must bear in mind that blast induced in-situ rock damage cannot be looked into isolation.

Rock & Rock Massif Parameters

Compressive strength & tensile strength of rock – Resistance of intact rock to failure when subjected to compressive or tensile stresses is indicated by this ratio. Normally tensile strength of rocks is $1/8^{th}$ to $1/13^{th}$ of compressive strength. Ratio of compressive strength of rock to its tensile strength is termed as "***blastability index/blastabilty coefficient***", and indicates the relative ease with which the rock can be blasted. *Higher "blastability index" rocks require less explosive or lower specific charge (kg/tonne). Lesser blastability index/blastabilty coefficient indicates that rock is difficult to blast.*

Young's Modulus – It is defined as ratio of axial stress to axial strain in uniaxial compression or tension, and is a measure of deformation which a rock can sustain before failure. It is measure of resistance offered by rock subjected to dynamic loading. *A high Young's Modulus indicates stiffer rock, less susceptible to deformation before failure, and requires greater specific charge and explosive having higher VOD.*

Density – Density and strength of rock can be correlated. *Rocks will low density can be deformed easily and require low explosive energy/lesser specific charge. Dense rocks require high energy input, explosives with higher heave energy, increase in blast-hole diameter or reduction in blast-hole spacing & burden especially if they are massive.*

Typical Rock properties

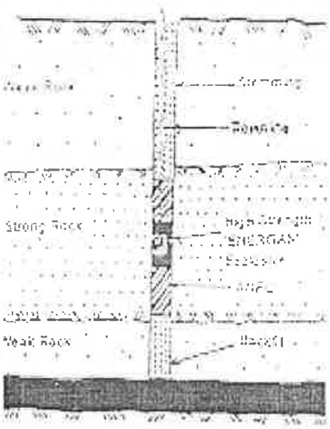
Rock Type	Density gm/cc	Young's Modulus GPa	Compr. Strength MPa	Tensile strength MPa
Basalt	2.4 – 2.9	35 – 60	50 – 300	6 – 30
Sandstone	2.2 – 2.7	10 – 40	40 – 150	2 – 15
Siltstone	2.0 – 2.8	10 – 15	30 – 130	2 – 12
Coal	1.2 – 1.5	2 - 8	4 – 40	

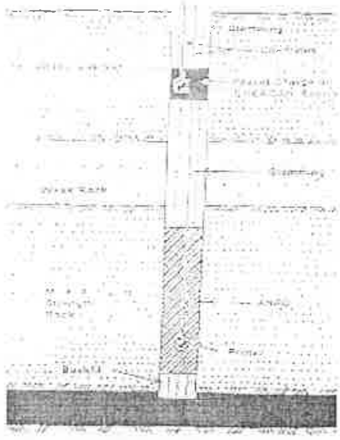
Water Content – The amount and nature of water present in the rock mass influences selection of explosives. *Water resistant explosives need to be used which require multiple priming if ground water is flowing dynamically.* Water saturation increases velocity of propagation of blast generated stress waves and reduces their attenuation. Its presence promotes heave by reducing expansion of gases into pre-existing cracks and voids in the rock mass.

Internal friction and porosity – Internal friction or damping capacity is a relative measure of rock's ability to attenuate stress waves. The internal friction differs with rock type and sedimentary rocks have higher internal damping capacity than igneous or metamorphic rocks.

Compression/Primary/P wave velocity – It is velocity of propagation of compressive wave in the rock/rock mass consequent to detonation of an explosive charge, and is high if the rock is compact and free of discontinuities. A fractured rock mass has invariably low P wave velocity due to longer time taken for travelling across the fractures. A low P wave velocity of rock mass is indicative of poor strength of rock mass quality. P wave velocity has direct relation with Young's Modulus. *Rock mass having high P wave velocity requires higher specific charge of explosives having high shock energy and high gas energy for blasting. Rocks having high P wave velocity are less susceptible to blast induced in-situ rock damage.*

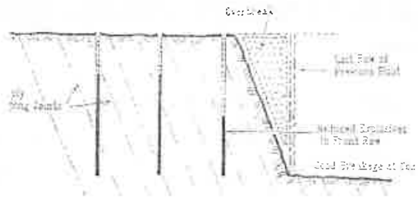
Rock Massif diversity – Rock massif is not homogenous and layers of different rock types of varying thickness (few mm to 10s or 100s of metre) or same rock of different degree of weathering are commonly encountered. Even in small area, a particular rock, despite mineralogical similarities, may have different physical properties. The straight-down/vertical variability of the rock massif can be inferred by critically observing the drilling rate and examining the drill cuttings as the drilling progresses. The lateral variability of the rock mass is once again unpredictable and calls for design changes as the blasting progresses. Figs below depict some examples of desirable interaction between explosive and rock.



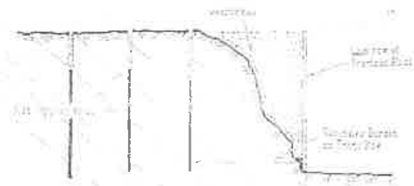


Discontinuities – All rocks contain discontinuities like bedding planes and joints. They get opened up directly by explosion gases or indirectly by ground movement during blasting. These discontinuities also prevent propagation of blast induced fractures, barring for those which lie close to the blast-hole. They bear more influence than the properties of intact rock substance. The bedding planes and joints tend to dominate fragmentation mechanism. The blast induced rock damage and severity of damage is largely governed by these discontinuities.

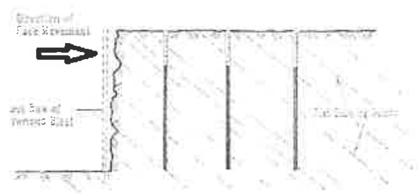
If the rock mass contains 3 mutually perpendicular sets of closely spaced joints (<0.5m), fragmentation is easily achieved. If the joints are few and widely spaced (>3.0m), they detract fragmentation, as they prematurely terminate the blast induced cracks. *Increase in spacing of discontinuities implies effective resistance to blasting. They call for reduction in blast-hole diameter, spacing & burden of holes and stemming length or increase in specific charge of explosives.*



The orientation of joints and bedding planes also affects the blast performance, as they allow venting of explosion gases if intersecting the blast-hole, thus lowering the explosive's rock breakage and heave capacity.



Joints running normal to the face tend to result in over break beyond the intended blast boundary. The orientation of joints and bedding planes has a significant effect on slope stability of high-walls. If joints are steeply dipping towards the face, there will be some unavoidable over break/back break. As the joints flatten, angled holes may become necessity to avoid/reduce toe burdens. The problem does not occur if joints are dipping away from the face.



Rock Massif classification – Lilly defined "blastability index" taking into account five rock parameters, whose representative values are given below -

Blastability Index BI = 0.5 (RMD + JPS + JPO + SGI + H)

Where RMD – Rock mass description, valued 10 for friable rock, 20 for blocky rock mass & 30 for massive massif

JPS – joint plane spacing, valued as 10 for spacing < 0.1m, 20 for spacing varying between 0.1-1.0m, and 50 for spacing > 1.0m

JPO – joint plane orientation, valued as 10 for horizontal, 20 for dipping out of face, and 40 for dipping into the face

SGI – Specific gravity influence = 25 x density of rock – 50

H – Hardness of rock from 1 -10

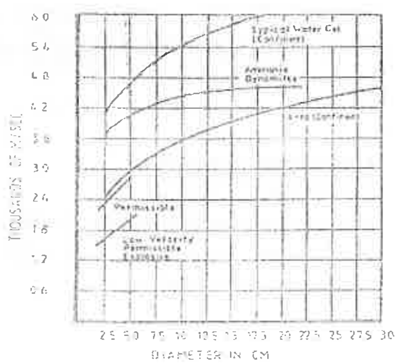
Higher blastability index of rock massif requires more explosive and higher strength of explosive (energy factor)

Explosive's parameters - Choice of explosives involves consideration of properties of the explosive as well as that of the rock massif being blasted. Explosives differ in density, rate of detonation, water resistivity and in energy they apply to the rock mass. They can broadly be classified as per strength as "***Low Explosives***" and "***High Explosives***". They are manufactured and supplied in form of cartridges – small in diameter (25mm & 52mm) as well as large in diameter (+83mm), as well as in bulk as site-mixed ANFO/Aluminised ANFO/slurries/emulsions, which are directly loaded into blast-holes.

The site-mixed slurry/emulsion explosives carry an inherent advantage that their density and strength can be varied as per site-specific requirements and the mix is non-explosive till the time it is poured into the blast-hole, and become an explosive mix once inside the hole and after completion of gassing & gelling. The large diameter and site-mixed bulk explosives can be cap sensitive or non-cap sensitive, requiring sympathetic detonation from another charge of explosive. Small diameter explosives are normally cap-sensitive. Following are important properties of explosives, some of which can be varied and are partially controllable on site too –

Density – It is weight/volume of explosive expressed in gm/cc, and this parameter determines weight of explosive that can be loaded per running meter of the blast-hole/shot-hole of a given diameter. A wide range of density of cartridged slurry explosives is available in the market off the rack, and can also be manufactured on demand. Density of site mixed bulk slurries/emulsions varies with depth of blast-hole due to hydrostatic pressure of at different depths (arising out of the explosive column & stemming above).

VOD or Velocity of detonation – It is the speed in metres/second at which detonation wave travels through the explosive column, once the explosive has been detonated. VOD is principally influenced by chemical composition of explosives. VOD of some explosives is also dependent on diameter of blast-hole and it reduces drastically if the diameter of the blast-hole is below certain critical value. *The diameter of blast-holes has to be more than the critical diameter of explosive when designing a blast.* Low explosives have VOD of 1500-2500 m/s and high ones have VOD of 2500-7000m/s.



Impedence of explosives is defined as VOD of explosive x density of charging. *It is normally correlated with impedence of rock whilst making choice of explosive.*

VOD x density of explosive = P wave velocity x density of rock

Detonation Pressure – it is the pressure in the reaction zone behind the detonating front of explosives column, and is a function of charge density and VOD of the explosive.

Dp = density x VOD^2 x particle velocity of explosives
$Dp = 2.325 \times 10^7 \times VOD^2 \times$ density

Peak blast-hole pressure is the pressure exerted by the gaseous products of detonating reaction on blast-hole wall. It is about 45% of detonating pressure. *It is a measure of heaving capacity of the explosive.*

Propagation sensitivity is the ability of an explosive to support or reinforce detonation that has already been created. *Low propagation sensitive explosive require multiple priming along the explosive column, particularly in deep blast-holes, to obtain their full strength.*

Initiation sensitivity is the ease or minimum booster with which an explosive charge can be detonated.

Gap Sensitivity is the length of air gap or discontinuity between cartridges in a single charge across which the detonation in an explosive column continues. The transmission mechanism is influenced by VOD, direction of shock wave, hot reaction products, and explosive composition. *Explosives should have good gap sensitivity to overcome any inefficacies in charging.*

Sympathetic detonation is initiation of an explosive charge without priming, called as receptor, by detonation of an adjoining charge of explosives, called as donor. *The spacing and burden of holes in the blast design have to be more that the sympathetic detonation gap, else the blast-holes will fire in non-conformity to the delay pattern.*

Absolute weight strength is the absolute amount of energy in calories available in each gram of explosive.

Absolute bulk strength is the absolute amount of energy in calories available in each cc of explosive. *This is more important as it determines the energy available in the blast-hole.*

Relative weight strength & Relative bulk strength is the measure of energy available in the explosive per weight of ANFO or per volume of bulk ANFO at 0.81 gm/cc density respectively.

Energy factor – the energy distribution in a blast is measured by energy factor which compares explosive energy to quantity of rock broken.

EF = kilo cal of explosives charged/quantity of rock broken in tonnes

Initiation devices

Safety fuse and ordinary detonator is the simplest of the initiation accessory comprising of gunpowder wrapped in textile yarn and water proof coating. It has a burning rate 1m/100s and, varied length of fuse can give delay effect. Because of its crudity and high unreliability, the SF/OD combination is used only in small scale jack-hammer drilled holes.

Electrical pyrotechnic detonators – they may be instantaneous, half-second delay, or milli-second Delay. In delay detonators, delay element is initiated to burn for a set time upon receipt of electrical signal, before the detonator is initiated. These detonators allow large number of shots to be fired, with already fired preceding holes providing the free face and the relief for displacement for holes firing subsequently. The resultant ground vibrations can also be minimized. However, they have certain limitation in large blasts and in special conditions as in cases where in-situ rock damage needs to be minimized. *Because of inherent variation in delay composition and burning speeds, absolute firing times of the detonators may be different to their normal (scatter/overlapping effect), and there may be instances that holes designed to fire later may blast earlier than desired, choking the blast, or causing misfires.* Else the blast needs to be designed by lengthening the delay interval, which may be detrimental to the blast performance itself and the resultant ground vibrations.

Detonating Fuse – It contains 10-15gm/m core of high VOD (6500m/s), cap sensitive explosive (usually PETN) and is used to detonate other high explosives with which it comes in contact. It is used in combination with cast booster but has an inherent disadvantage that when used down the hole, it partially burns or even detonates bulk loaded blast-hole column top downwards. There is thus loss of energy and explosive column gets under-loaded to result in venting of holes, poor fragmentation, excessive fly rock and increase in ground vibrations.

In large blasts, the DF is used with **detonating-relays**, which provide for short delay interval between holes and between rows of holes. The relays comprise of two short-delay detonators placed in a plastic mould with holes on either end for insertion of DF. Delay intervals of 15, 17, 25, 35, 45, 60 & 100ms are available. The detonating relays are not used down the hole.

Low-energy detonating cord – It contains core of 0.4-1.0gm/m of PETN, and is used in conjunction with **non-electric delay detonators.** This initiating system gives the same advantage as that of precise short-delay electric circuits. True bottom priming can be achieved with this combination. It also eliminates hazards associated with electrical firing (extraneous electrical current) and is resistant to accidental detonation,

Shock Tubes – It is further improvement of low-energy detonating cord and comprises of a small diameter plastic tube, inside of which is coated a reactive substance that maintains propagation of shock wave @ 2000m/s. The shock wave is adequate to initiate delay element of a non-electric detonator. There is no desensitization of explosives column and is used in conjunction with down-the-hole non-electric delay detonators. Its circuit is initiated by a special starting gun.

The non-electric detonating systems have less/zero scatter/overlap, but have a great disadvantage that the health of the circuit cannot be tested as in case of electric firing systems. There is danger of cut-offs also in such type of initiating systems. *Higher delays are used with larger diameter of holes where burden is also higher, as they would give adequate time for progressive relief of burden minimizing risks of cut-off. In fractured ground or blocky rocks, where cut-offs are likely, smaller diameter holes with smaller burdens should be used to minimize cut-offs.*

Gas-mixture initiating system – In this initiation system, a flammable gaseous mixture is used inside a plastic tube for conveying the detonation wave, and it is used in conjunction with special down-the-hole non-electric delay detonators. The tubing from the hole is connected to trunk lines through plastic connectors. Gas, comprising of fuel and oxidiser, is charged into the circuit from gas supplying unit just before firing the circuit and the firing is by a special gun. It carries a major advantage over other non-electric initiation systems that circuit can be checked for loose connections by checking pressure at the outbye end of hook-up. *As in case of other non-electric initiation systems, higher delays are used with larger diameter of holes where burden is also higher, as they would give adequate time for progressive relief of burden minimizing risks of cut-off. In fractured ground or blocky rocks, where cut-offs are likely, smaller diameter holes with smaller burdens should be used to minimize cut-offs.*

<u>**Special Initiation Systems**</u>

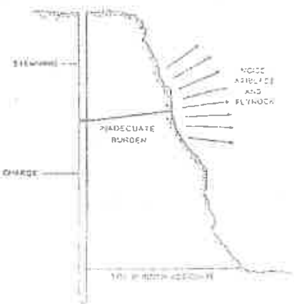
Sequential Blasting Machines – They are used in conjunction with electric down-the-hole delay detonators. The sequential timer of the machine produces part of the delay in the firing circuit. It can fire 10 blasting circuits, each capable of firing 25 detonators connected in series. The sequential exploder comprises fully solid state condenser discharge type of 10 independent blasting circuits. The energy to each of the 10 circuits is electronically switched on sequentially at a programmed delay interval (adjustable from 7ms to 99ms), and an interlocking circuit in its design prevents firing of the circuit till all the capacitors are charged to their rated voltage. If any of the circuits does not fire during blasting, firing of the subsequent circuits gets automatically stopped, short-terminating the blast and preventing its choking. *Mass blasts can be fired with this machine, with fair level of delay-interval accuracy, giving better fragmentation, low blast-induced ground vibrations and less in-situ rock damage.*

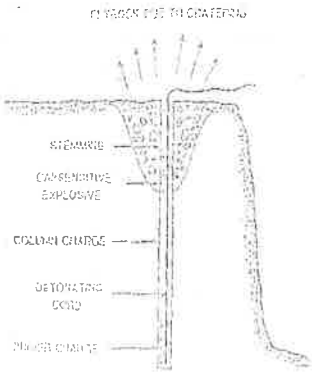
Electronic delay detonators or programmable digital detonators – These detonators come as factory programmed and are also available as an on-site programmable version. The digital detonator contains a energy storage capacitor, microchip/miniature electronic timing circuit, and explosive components. The capacitor can store enough energy to run the micro-chip independent of external power for 8 seconds to fire the fuse head of the detonator. The microchip circuitry includes an oscillator for timing, memory for retaining its programmed delay and communication functions to receive and deliver digital message to and from the control equipment. The timing circuit controls firing of the fuse head of the detonator. They carry an inherent advantage that the system not influenced by static electricity, radio waves or stray ground currents. These detonators are frequency selective and can only be activated by high frequency AC power only.

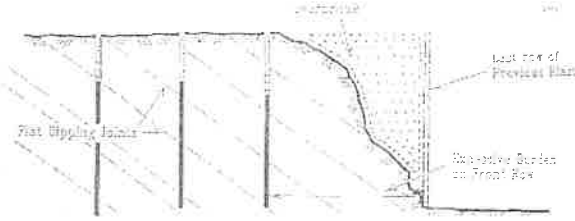
A logger is used to communicate with the detonators during hook-up, operating at inherent safe voltage. It recognizes and tests each digital detonator as it is clipped. In factory programmed detonators, the timing desired by the user, comes in programmed. In on-site programmable detonators, the required delay time for each detonator is entered and written into logger's memory and the memory is used to programme each detonator during the firing sequence. The logger can be used after hook-up to test the response from each detonator. A blaster (equipment), containing the required voltages and codes capable of firing the detonators, is used to fire the blast in conjunction with the logger. The fully programmable detonators can give delays in 1ms increments from 1ms to 8000ms with an accuracy of 0.1%. They are safe to use, fully testable at any time as there is two-way communication between the detonators and the control equipment.

Correct sequencing can be used and is assured. Thus larger blasts can be fired without compromising on key blast outcomes governed by delay timing effect. Fragmentation can be maximized and ground vibrations can be reduced as each hole blasts at a unique delay. The system also allows relief (free face) to be developed anywhere (by programming *the delay interval as to take into account the swell factor too) to ensure clearance of material.*

Blast Geometry







Burden B – It is governed by a number of factors like blast-hole diameter, bench height, type of explosive, density of rock mass, number of rows being blasted, delay interval between rows, and rock/rock massif characteristics (joints & their frequency & orientation, cementation of joints/weak planes, bedding planes & their direction). As large diameter holes can accommodate more explosive, it is but obvious that burden has to be more, unless low energy explosive is used. With higher bench heights, larger diameter of holes is required. *Too close burden causes wastage of explosive energy, venting of explosive column, more of air blast, and fly rock. Excessive burden leads to cratering effect.* Role of rock massif has already been discussed in preceding text. Various scientists have given their empirical approach in deciding the parameter, based on results obtained by them in field trials. Few are -

Anderson - $B = K\,(\text{Hole Dia in mm} \times \text{Bench Height in m})^{0.5}$, where K= constant
(for good fragmentation bench height : burden ratio should not be more than 4)

Konya - $B = Kr \times Ks \times \{2 \times (\text{sp. gr. of explosive/sp. gr. of rock}) + 1.5\} \times \text{dia of expl.}$

Where

Kr = correction for no. of rows, 1 for two rows & 0.9 for 3rd & subseq. Rows
Ks = correction for geological structures,

1.30 for heavily cracked, weak joints, weakly cemented layers
1.10 for thin well cemented layers and tight joints
0.95 for massive intact rock
1.20 for bedding steeply dipping into the cut
0.95 for bedding steeply dipping into the face

Vutukuru & Bhandari – $B = (0.024 \times \text{hole dia in mm} + 0.85)\text{m}$

Concept of stiffness (bench height/burden ratio) has been utilized in some blast design studies

Stiffness	*Fragmentation*	*Air blast*	*Fly Rock*	*Ground Vibration*	*Remarks*
1	Poor	Severe	severe	Very high	High back break
2	Fair	Fair	Fairly high	Fairly high	
3	Good	Reasonably controlled & moderate			
4	Excellent	Controlled/negligible			

Berta – uses explosive energy balance theory, presuming 15% of explosive energy is utilized in breaking rock

$$B = \frac{0.15 \times \text{Coupling factor} \times \text{Sp.gr. of explosive} \times \text{VOD} \times \text{length of explosive column}}{\text{density of rock} \times \text{seismic velocity of rock}}$$

Spacing S - is dependent on burden, blast-hole depth, type of explosive, rock-massif, and delay type being used. Too close spacing causes crushing and cratering between holes, boulders in burden region, excessive blast vibrations and air blast. Too high spacing will result in inadequate fracturing between holes, humps on face and toe problems between holes. *Spacing vis-à-vis the explosive type should be such that cracks initiating from the blast-hole consequent to detonation of explosive charge, should propagate up to mid-way the spacing.* Again, various empirical approaches have been professed in deciding the spacing, which are –

S : B ratio = 1.0 when firing single row instantaneously
= 1.3-1.5 when firing single row with delays
= 2.0 when firing multi-row with delays

Vutukuru & Bhandari – S = (0.09 x burden + 0.0.91)m

Stemming length T depends on rock type being blasted. Stemming prevents venting of gases. Premature ejection of explosives or venting of gases will result in poor fragmentation and also lead to restriction in movement of surrounding rock. It will also lead to high air over-pressure (air blast).

T = 12 to 30 times the hole diameter (less for hard & competent strata)
T = 0.7-1.3 times the burden

Stemming material – angular material/stone chips are best stemming material as the chips tend to lock in place. The chip size can increase with hole diameter and particle size of $1/25^{th}$ diameter of hole has been suggested.

Sub grade drilling –Its length depends on nature of rock type and nature of parting at bench bottom. Sub grade drilling prevents toe formation. Sub grade drilling up to 30% of burden has been suggested.

Open bedding plane at bench bottom	– 0%
Easy Toe conditions	– 10-20% bench height
Difficult Toe conditions	– 20-30% bench height

Hole diameter to Bench Height: Bench height is one of the principal governing factors for drill hole dimension. Higher benches require larger diameter of holes, as smaller diameter holes may be impracticable to drill. larger burden with larger diameter blast-holes are required as bench height increases.

Upto 3m bench height	– 32mm
3-8m bench height	– 100mm
8-12m bench height	– 150/169mm
12-20m bench height	– 250mm
More than 20m bench height	– 311mm

Hole diameter vis-à-vis the explosive type – Explosives, especially ANFO and bulk explosives have a critical diameter, and sensitivity of explosives comes down drastically below this diameter. The detonation pressure and detonating velocity of explosives also come down with diameter. The hole diameter thus should be above critical diameter of explosive being used.

Decking – The explosive column in a blast-hole can be divided by inserting one or more inert material segments. It helps in charge distribution where better energy is required, as in case of bedded rock massif. Decking is also warranted in high benches/deep blast-holes, else total charge in the hole will be exceptionally high which may be uncalled for in terms of energy factor required to break the rock. Decking helps in reducing ground vibration as every deck can be given a different delay. Recommended deck thickness is not less than 6 times the hole diameter.

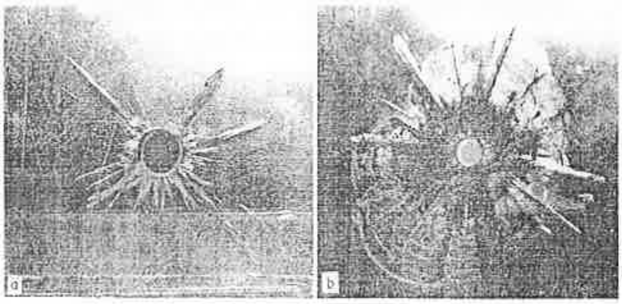
Explosive distribution – Total explosive required to be charged in a blast hole, is given by –

$$Q = q \times (B \times S \times H)$$

Where q – specific charge of explosives (kg/m^3)
B – Burden
S – Spacing
H – Bench height

$$q = 0.0163 \times \text{compressive strength of rock}^{0.62} \times B^{-0.4}$$

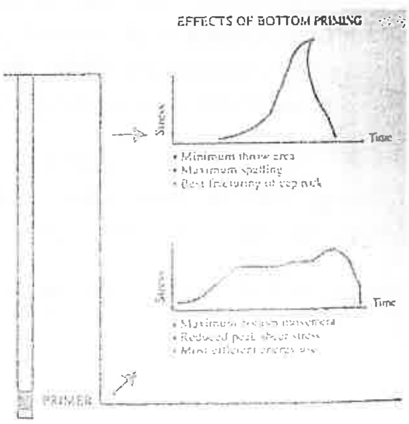
De-coupling – An explosive is said to have good coupling with the blast-hole if it completely fills the entire cross-section of the blast-hole, as in case of slurries/site mixed bulk explosives which are directly poured down. A good coupling with the blast-hole ensures complete transfer of explosive energy to the blast-hole walls and in turn to the rock, thus giving better fragmentation. De-coupling is done when crushing of rock is not desired. The excessive crushing during detonation of an explosive is as a result of high dissipation of strain energy. De-coupling has significant influence on resultant ground vibrations too. Decoupling can be done by using lesser diameter of explosive than the diameter of blast-hole and placing the charge centrally in the hole as to maintain air-gap with the blast-hole walls. Advantage of the air cushion is taken when the explosive is fired.



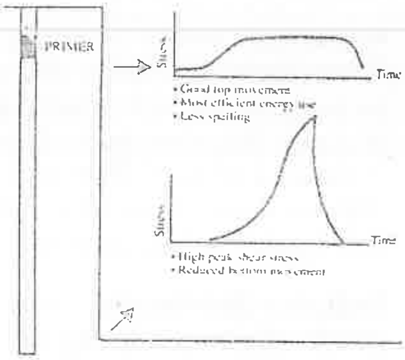
More cracks are obtained with the same charge when it entirely fil he hole. In (a) the volume of the hole was four times larger than in (b), bu he charges were the same.

Primer/priming of explosives – the primer should have high VOD & density, should be of same diameter as column charge and its weight should be adequate to ensure that the column charge can reach its steady state VOD

Primer location – It affects the magnitude and the shape of stress wave in the rock mass, movement of rock mass during blasting, shearing of rock mass at grade level and breakage of cap rock above stemming/explosive interface. Studies have shown that peak stress levels occur in the direction of explosive detonation. The intensity and shape of the stress wave is related to speed at which the wave travels in the rock and VOD of explosive. ***Bottom priming*** gives the benefit of increased gas confinement and greater movement, at the base/

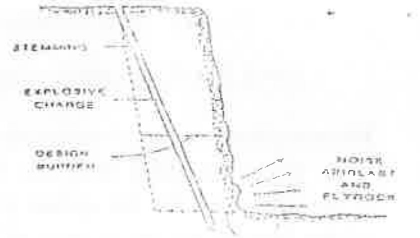


bottom of blast-hole coupled with reduced peak shear stress. It also gives minimum throw, maximum spalling and best fragmentation of the cap rock (at the top of the hole). It gives better results maximising the fragmentation in the cap zone. ***Top priming*** on the other hand will give high peak stress and reduced ground movement at the blast-hole bottom, and low spalling but high ground movement in the cap zone.



As the bottom part of the blast-hole is constricted, its breakage and movement is critical for successful blasting. The bottom charge should be of higher VOD and should also be better confined. Bottom priming is thus normally preferred. Multiple priming is suggested in deep and large diameter blast-holes, in heavily jointed strata, when using relatively insensitive explosive, and in bulk loaded large diameter blast-holes.

Angled/Inclined holes – they give vertical element to the blast, thus giving better throw. Holes need to be drilled precisely parallel to the high wall to give uniform burden along the entire length of blast-hole, which in turn will give uniform movement of burden, reducing chances of vents & fly-rock. The projection of burden is up and forward and thus tough toes are also avoided.



Before charging, each blast-hole need to be surveyed for its inclination and depth as to record all deviations, and the individual blast-hole charge pattern is accordingly modified. Deviations result in more fly-rocks, air blasts and toe formation.

<u>**Delay pattern**</u> – In multiple row firing, it is important to not only have a proper spacing and burden but also a proper delay sequence in between holes of the same row as well as between holes of different rows. *Key blast outcomes are governed by the delay timing effect.* The onset of rock movement depends on material response in conjunction with stress and gas pressure stimulus generated by the explosive.

The basic concept in multi-row firing is to ensure burden relief for each hole, which is ensured if at least 3 adjoining holes, two holes in the row ahead and one in the same row have successfully detonated/fired in advance (at an earlier time interval) and the rock mass has responded to provide relief to the hole being fired. If decking is done, the above needs to be ensured for each deck instead of hole so that none of the decks are over-burdened. In large multi-row, multi deck blasts, the blast geometry is checked through computer applications for a number of detonation stimulations.

Fragmentation can be maximized, ground vibrations can be reduced, and in-situ rock damage can be minimized if each hole blasts at a unique delay, as it would ensure proper relief for the rock being blasted, to ensure proper clearance of material.

Hole to hole delay interval or delay between adjoining holes – The concept of providing delay between adjoining holes, is to ensure that cracks which radiate out from the blast-hole fired first, reach mid-way of the spacing, after which the adjoining blast-hole should fire/

detonate. The delay interval thus ensures complete propagation of cracks across the spacing. The interval is thus dependent on the type of explosive used, nature of the rock, spacing and crack propagation velocity. Crack propagation is faster in strong & massive rocks.

Many approaches, including based on minimum response time of rock, have been put forth by scientists. An empirical approach professed by Konya, suggests hole to hole delay interval as -

$T_h = T_H \times S$ Where T_h – hole to hole delay in ms
T_H – delay constant, 1.8-2.1 for weak & bedded rocks, and 0.9-1.2 for strong & massive rocks (increase delay constant for fractured rock by 50%)
S – Spacing in m

Row to row delay interval – The purpose of providing delay interval in burden is to provide proper relief and displacement to the rock which will be generated on detonation of blast-hole in the succeeding row. *The hole in the succeeding row should blast once the fragmented rock generated from the blast-hole in the preceding row has moved by at least 1/3rd of the burden distance.* This takes care of the swell-factor. The burden-delay interval is thus also dependent on type of explosive used, minimum response time of rock/heave velocity of the fragmented rock, and the burden. Shorter delay times will give higher rock-piles closer to face and fragmentation will be more (including on account of in-flight collision of broken mass), but it will also cause more back break, more violence, air blast, ground vibrations and fly rock. Longer delay times will yield scattered rock piles, decreased ground vibration levels and decreased back breaks as more of explosive energy gets utilized in the direction of the free face.

$T_r = T_R \times B$ Where T_r -- row to row delay in ms
B – Burden
T_R – delay constant
2 – violent & excessive air blast, back break
2-4 – high pile close to face, average to moderate air blast and back break.
4-6 – scattered pile with minimum back break,
7-14 – side casting of blast

Computer Applications – In large blasts, optimization by computer aided techniques is now normally done. A large number of objective-defined computer programmes have been devised for the purpose. Inputs on rock type & its properties (density, compressive strength, tensile strength, p & s wave velocities, crack attenuation factor, rock quality factors, and geological and structural details of the rock mass), explosive's performance (density, VOD, explosion pressure, shock energy, gas energy, etc.), and blast geometry (hole dia, bench height, hole depth, burden, spacing, etc.) are put to various detonation simulations, to predict various objectives like fragmentation, throw, muck-profile, fly-rock distances, ground vibration, in-situ rock damage, etc.

Softwares are also available to play back the detonation sequence to allow blast engineer to assess areas of blast that could be affected by over-confinement/choking due to inadequate burden-relief, and thus take controlling measures (re-define detonation simulation) to ensure that the blast detonates successfully.

Figures depict how re-programming of detonating interval gave better success. The delay between spacing & burden was 42ms & 65ms in Case 1 (depicted above), and 65ms & 150ms in Case 2 & Case 3 (depicted below). Down the hole delay in all cases was 500ms. However, scatter factor was 4% in (1) & (2) and 0.1% in (3). Only 1st row blasted with 100% success in (1), 4 rows blasted with 100% success in (2) and in (3), success was 100% barring for corner regions of the blast.

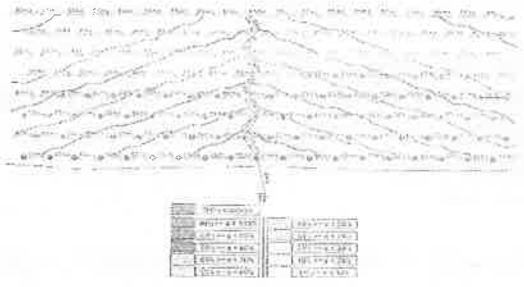
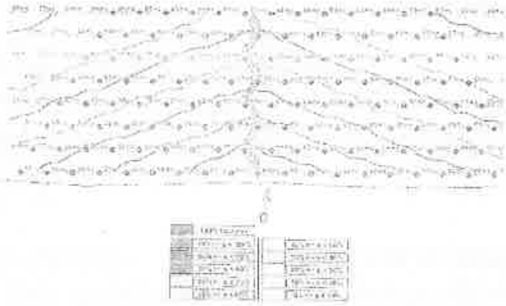
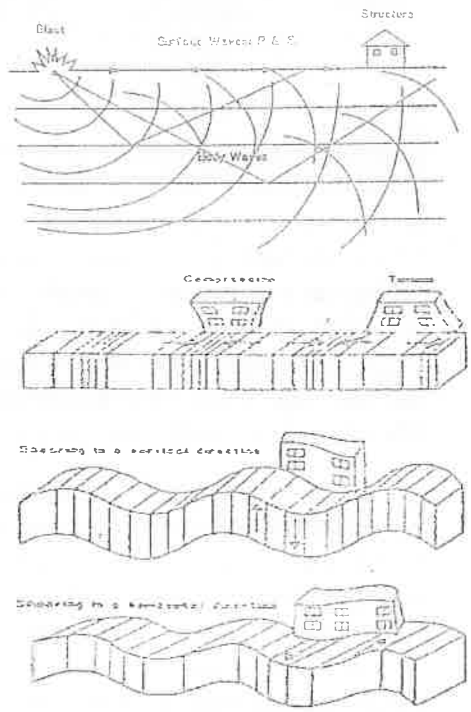


Figure 5 – Simulation results for design 2: Inter-hole = 65ms, [illegible] = 150ms, down hole = 500ms, [illegible]



Blast Performance Assessment – The blast can be assessed visually by observing boulder formation/secondary blasting, fragmentation size distribution, muck-pile geometry, throw, fly-rock, and to some extent by air-over pressure. Through high speed photographic techniques, initiation sequence of blast-holes, stemming ejection velocity, venting of holes or degree of confinement provided by the stemming material, face movement, uplift velocity of bench top etc., can be measured. Though inter-related, visual and high-speed observation may not truly reflect on potential in-situ rock damage or damage to high wall the blast might have caused.

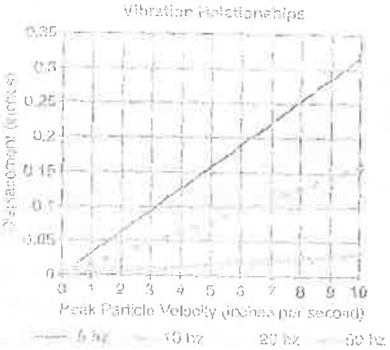
Blast Induced Ground Vibrations measurement and its control through choice of explosives vis-à-vis the structural integrity of rock mass, blast geometry and delay timing configuration, is one measure whereby stability of high-walls and damage to in-situ rock can be assessed and minimized.

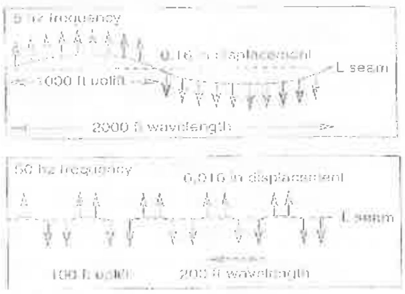


Consequent to detonation of an explosive charge, the initial shock front outside the zone of shattering applies force to the rock through which it moves. Particles in the path of such waves move back & forth in line of wave advance and are called longitudinal/compressional/primary or P wave and travels much faster than other types of waves generated by the blast. When the longitudinal wave strikes a free face or discontinuity or a change of material at an angle it is refracted or reflected to give rise to shear/secondary/S-wave.

In addition there are waves which travel on surface, collectively called as surface waves. These surface waves cause the particles in its path to vibrate in vertical direction (R wave) and in transverse direction with no vertical displacement (Q wave), causing shearing in vertical and horizontal directions respectively. These waves not only affect surface structures through compression and tension and through vertical and horizontal shearing effects but also the in-situ rock. The ground vibrations are normally measured in terms of peak particle velocity, displacement (amplitude of the wave), and acceleration. Frequency of vibrations is a common variable factor between displacement and peak particle velocity.

In soft & weak strata, dominant frequency response is typically between 3-10Hz. Low frequency resonance increases the potential for damage. The vibration relationship is shown in adjoining graph. If the blast produces ppv of 127mm/sec (5 ips), particle displacement would be 10 times greater for dominant frequency of 5Hz than for dominant frequency of 50Hz. Higher frequency vibrations produce less displacement. In addition there is also an inverse relationship between frequency and strain. A wave of frequency of 5Hz will generate 10 times more strain than that generated by vibration having frequency of 50Hz.





Another factor which needs to be evaluated for damage potential is length of the wave. It can be calculated by dividing seismic velocity of the rock by frequency of the vibration. Longer wavelengths are detrimental to the stability of high walls. So every effort needs to be made to control ground vibration's dominant frequency and amplitude.

The frequency of ground vibrations are primarily controlled by rock massif. However the blast's delay configuration also plays a role in frequency of blast-induced ground vibrations. The low frequency vibrations can be shifted to high frequency one, and requires proper delay timing between holes of the same row as well as between rows, accuracy of firing times (within $\pm$ 1ms – achieved by electronic detonators only), and equal-confinement of explosive in each blast hole. Use of air decks has also helped reducing the peak particle velocity and vibration amplitudes, as well as displacement along weak planes. To arrive at proper delay timings computer modelling is used to simulate potential vibration level and frequency content. The vibration goal is to get high dominant ground vibration frequency and less displacement.

Actual ground vibration monitoring, as close to the blast-site as possible, can give true picture of threshold PPV values. The damage threshold value of PPV for in-situ rock damage is considered to be 600mm/sec and minor damage above 300mm/sec is expected. Rock damage can be predicted from far-field observations of PPV. Different researches have professed site-specific predictor equations, and vibration measurements need to be carried out to arrive at the site specific constants. Most commonly used is the scaled distance concept, in which,

$$\mathbf{V = K\,(\,D/W^{1/2}\,)^{a}}$$

Where V = PPV in mm/sec
D = distance of point of interest from blast
W = explosive charge/delay
K & a = site specific constants

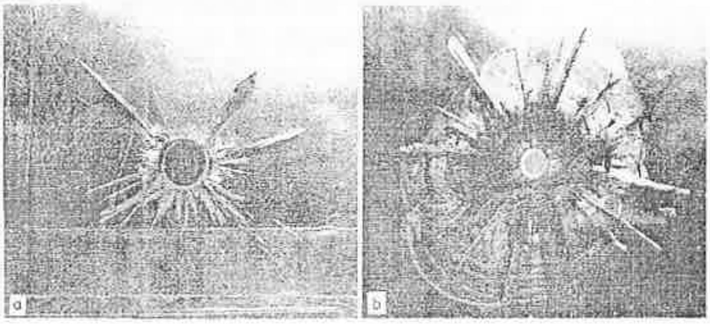
The equation, unfortunately, is not applicable for near field blast zone, and thus can be used to predict damage criteria for high-walls around the blast site.

Holmberg & Person have given the following equation to assess PPV near to the blast-hole –

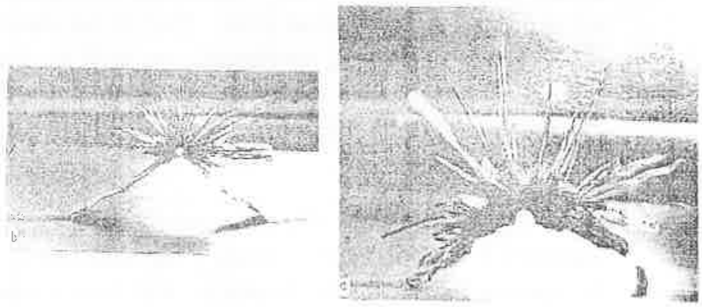
$$V = K\,[p/R_0^{a}\ \{\tan^{-1}(D/R_0) - \tan^{-1}(H/R_0)\}^{a}]$$

Where V = PPV in mm/sec
P =
Ro =
D =
H =
K & a = site specific constants

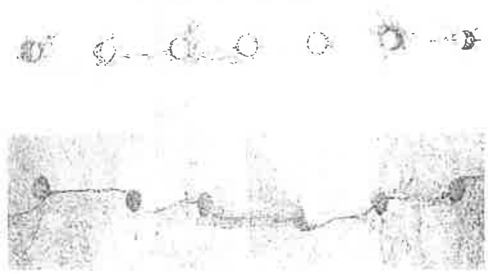
<u>Smooth Blasting – Contour blasting & Pre-splitting</u> - If the final contours of walls in open cuts and walls of underground excavations need to remain standing long, the badly blasted contours can have devastating consequences, from technical, safety, and economical points of view. One way to ensure that the quality of in-situ rock of high walls and walls of underground excavations is not destroyed is perimeter blasting, technically termed as smooth blasting, contour blasting or pre-splitting, which ensures that the final contour looks like knife-cut.



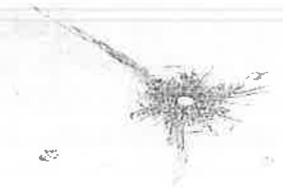
As already discussed in preceding text, detonation of an explosive charge crushes the rock close to the blast-hole walls and also initiates radial cracking around the crushed zone. Air cushion between the explosive and the blast-hole wall reduces the intensity of compressional shock wave. *More cracks are formed with the same charge of explosives when it completely fills the hole.* In first picture, blast-hole volume was 4 times the blast-hole volume in the picture on the right side, but explosive charge was same.



The dependence of crack formation on explosive charge is shown in adjoining photographs, where the same burden has been blasted with varying explosive charge. *For minimising stress on in-situ rock, it is essential to avoid overcharging.* In the picture on the right side, charge used is four times the charge in the model on the left hand side.



If an empty hole in a rock is put to tensional stress by detonating an explosive charge in an adjoining hole, it has been proven that there is a three-fold increase in stress at two points of the empty hole – one which is nearest to the hole and the other which is farthest. If the empty hole is close enough to the charged hole, cracks generate at the diametrically opposite ends of the empty hole. At low concentration of charge in the holes and spacing close enough, a line of cracks is created through the holes, almost without any cracks in other direction. As can be seen from adjoining photograph, hole closest to the charged hole receives the dominating effect. The holes are at distances of 1, 2, 3 & 4 cm from the charged hole.

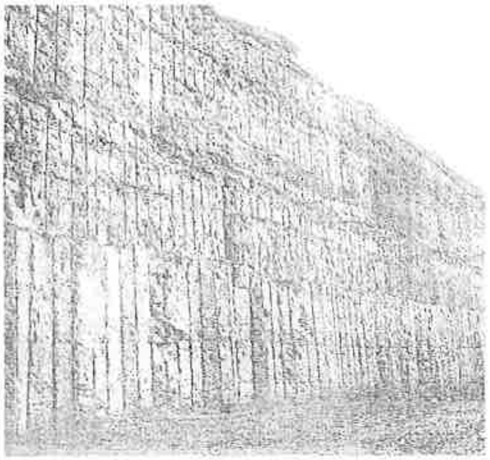


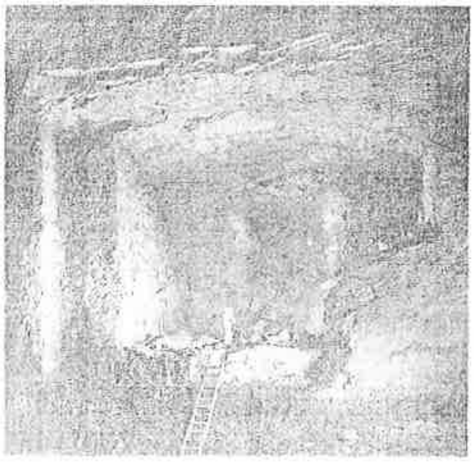
This effect is used in smooth blasting techniques to create cracks in precise direction, even independent of the direction of free face in front of the row. If the closely spaced lightly charged holes are blasted with the main round, it is termed as contour blasting. If these holes are blasted in advance and a crack is created in the rock massif, and the main round(s) (between free face and the crack at the rear) are blasted subsequently, the technique is termed as pre-splitting.

In contour blasting, the spacing/burden ratio of closely spaced holes should be < 0.5-0.8. Too less a burden will not only give an uneven final contour, but also a wall which is disturbed by cracks, which definitely will not be strong. The holes should be spaced at 3-0.7 times the blast-hole diameter distance. Only 1/3rd holes should be charged, as the uncharged holes would provide the easiest course for the cracks to propagate. Ignition of contour holes should be instantaneous, and top priming is preferred.

In pre-splitting too, the parameters remain the same, as in contour blasting. It gives an added advantage that the rock-massif is largely screened off from the ground vibrations of the main round(s) and it gives greater reliability against in-situ rock damage.

Excellent drilling precision is required in smooth blasting techniques. Special explosives are available for contour blasting/pre-splitting in form of continuous line of high explosives placed in a plastic pipe, equipped with special jointing sleeves. The explosive column is centralized in the blast hole and advantage of the air cushion is taken when the explosive is fired.





Explosive substitutes – In-organic expansion cement (normally called as rock breaking compound) is mixed with right proportion of water and is poured into closely spaced holes of appropriate diameter. High expansive stress of the order of 300-400 KSC is generated after the cement is left over a period of time, to initiate & propagate crack in the rock mass in the direction of the holes, which finally widen to separate the rock massif to a give smooth wall.

Identification of in-situ rock damaged zone – Seismic imaging, before and after the blast, is emerging as a major tool for assessing in-situ rock damage in high walls arising out of blasting. In this method, spikes are grouted at the back of high-wall at 0.75-1.0m interval and geo-phones are fixed on them. Seismic refraction survey, using hammering as a source to generate P waves, is done before blasting and using Seis-Optim software, signals are processed to generate depth versus depth P wave velocity profile. The process is repeated after the blast. Pre & post blast P wave velocity of each block is then compared.

Any significant reduction in P wave velocity in any of the blocks indicates in-situ rock damage due to blasting.

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Mechanism of Rock Breakage by Blasting, Influences of Rock and Explosive Properties on Blast Results

1. INTRODUCTION

Man has to break rock ever since the Stone Age when it formed his main source of raw material. Till this date, blasting with commercial explosives is the cheapest and easiest method of rock breakage in mining and construction projects throughout the world. However, it is an established fact that only 20 – 30% of explosive energy is utilized in breaking the rocks, while rest of the energy is wasted in producing the undesirable environmental disturbances such as ground vibration, noise/air overpressure, flyrock, dust pollution etc. Many theories have been developed to explain the behaviour of rock under the effect of explosive detonation (Obert and Duval, 1950; Duval and Atchison, 1957; Ash, 1977; Hagan, 1973 & 1977 etc.). However, the mechanism of rock breakage is still remains a problem to be solved and defined in the technology of application of explosives to breakage (Jimeno et al., 1995).

In this paper, the mechanism of rock breakage by blasting will be reviewed. The important properties of rock and rock mass will also be studied that have maximum influence on the blasting.

2. MECHANISM OF ROCK BREAKAGE

Jimeno et al. (1995) listed out eight breakage mechanisms those are involved in the fragmentation processes with explosives. The various mechanisms are:

(1) Crushing of Rock
(2) Radial Fracturing
(3) Reflection breakage or spalling
(4) Gas extension fractures
(5) Fracturing by release of load
(6) Fracturing along boundaries of modulus contrast of shear fracturing
(7) Breakage by flexion

(8) Fracture by in-flight collisions.

When an explosive charge inside a blasthole is detonated, the explosive is converted into a hot gas at intense pressure. A steep wave front travels into the rock, crushing it for roughly twice the radius of the original blasthole, depending upon the resistance of the rock. In many rock types, the cavity that is formed has about four times the volume of the original hole around the charge (Figure 1). Many radial cracks start to form as the cavity expand. However, a few of the cracks become dominant and the other stop growing. The expanding gases continue to work on the rock, extending the cracks, and moving the rock upward and outward. This activity takes place in the zone of intended work on the rock, breaking it and moving it for excavation. Beyond the perimeter of damage rock zone, the pulses are called elastic waves or seismic waves, meaning that there is no further damage to the rock or any permanent displacement of the rock properties. Seismic waves generated from blasting source travel in all directions. As they travel through the medium, they cause particle of the medium in motion which is called vibration.

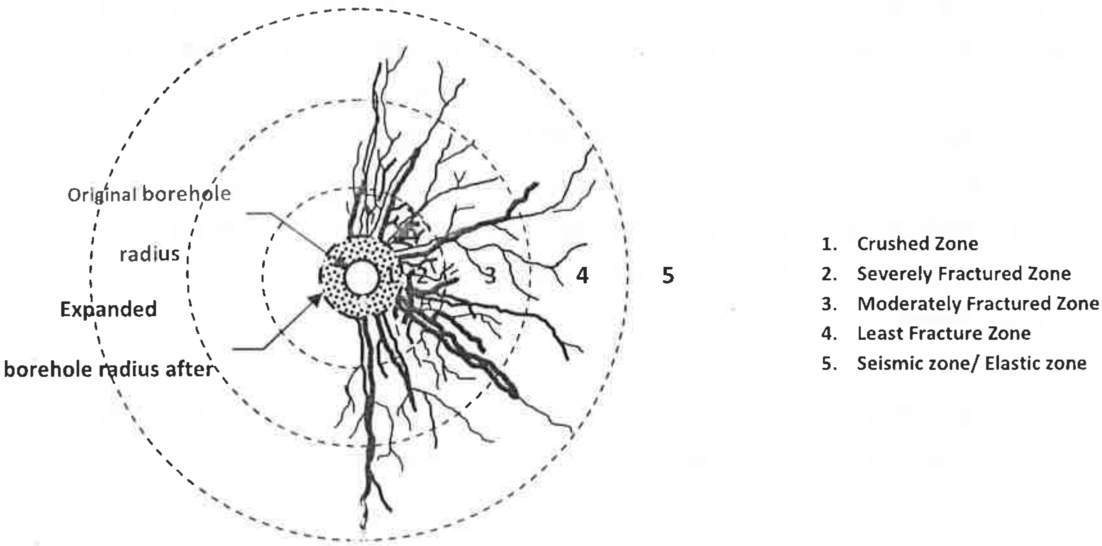

Fig. 1: Different processes of rock breakage

When the strain wave reaches the free surface, two waves are generated, a tensile wave and a shear wave. If the tensile wave is strong enough to exceed the dynamic strength of the rock, the phenomenon known as spalling will occur. This mechanism does not contribute much to the global fragmentation process, estimating that eight times more explosive charge would be require if the rock are to be fragmented solely by reflected waves.

After the stain wave passes, the pressure of the gases cause a squasi-static stress field around the blasthole. During or after the formation of cracks by the tangential component of the wave, the gases start to expand and penetrate into the fractures. The number and length of the opened and developed cracks strongly depend upon the pressure of the gases, and premature escape of these due to insufficient stemming.

During and after the mechanisms of radial fracturing and spalling, the pressure applied by the explosion gases upon the material in front of the explosive column make the rock act like a beam embedded in the bottom of the blasthole and in the stemming area, producing the deformation and fracturing of the same by the phenomena of flexion. The rock fragments created by the previous mechanisms and accelerated by the gases are projected towards the free face, colliding with each other and thereby producing additional fragmentation.

3.0 ROCK AND ROCK MASS PROPERTIES

Rock and rock mass properties largely influence blasting performance. They are the uncontrollable parameters in blast design. The properties of rock and rock mass which influence rock blasting are listed below:

(A) Rock Properties
- Density
- Dynamic strength of the rock
- Porosity
- Internal friction
- Conductivity
- Composition of rock and secondary

(B) Rock Mass Properties
- Lithology
- Joints/Discontinuity planes
- Stress field
- Water content
- Temperature of the rock mass

Reichhof & Moser (2000) summarised the influence of rock and rock mass parameters on the blastability and fragmentation as given in Table 1. From the table, it can be shown that blastability is influenced by almost all the parameters listed out. Few parameters such as tensile strength, shear strength and Poisson's ratio have been marked as not influencing the fragmentation.

Out of several parameters listed out, a few parameters have been quantified such as compressive strength, acoustic impedance, Young modulus, density, joint frequency and joint orientation to bench face. These parameters may be taken as the most important rock and rockmass properties to be considered for designing rock blasting.

Table 1: Influencing rock and rock mass parameters on blastability and fragmentation
(After Reichhof & Moser, 2000)

Influencing Parameters	Influence on		Quantified
	Blastability	Fragmentation	
ROCK MATERIAL			
Compressive strength	Yes/No	Yes/No	Yes
Tensile strength	Yes	No	No
Shear strength	Yes	No	No
Acoustic impedance	Yes	Yes	Yes
Young Modulus	Yes	Yes	Yes
Poisson's ratio	Yes	No	No
Mineral content	Yes	Yes	No
Angle of internal friction	Yes	Yes	No
Density	Yes	Yes	Yes
Porosity	Yes	Yes	No
JOINT PARAMETERS			
Joint status (open or close)	Yes	Yes	No
Joint width	Yes	Yes	No
Joint frequency	Yes	Yes	Yes
Type of filling materials	Yes	Yes	No
Shear strength of filling materials	Yes	Yes	No
Friction properties of filling materials	Yes	Yes	No
Joint distance to a borehole	Yes	Yes	No
Angle of incident – stress wave to joint face	Yes	Yes	No
JOINT ORIENTATION			
Joint orientation with respect to bench face	Yes	Yes	Yes

Kutuzov (1979) had recommended the explosive charge or powder factor required in bench blasting based on compressive strength, density and joint spacing as given in Table 2.

Table 2: Rock classification according to their facility of fragmentation by explosives in open pit mines (after Kutuzov, 1979)

Powder Factor (kg/m^3)		Joint spacing (m)	Uniaxial compressive strength (MPa)	Rock density (t/m^3)
Class Limit	Average value			
0.12 – 0.18	0.150	< 0.10	10 – 30	1.40 – 1.80
0.18 – 0.27	0.225	0.10 – 0.25	20 – 45	1.75 – 2.35
0.27 – 0.38	0.320	0.20 – 0.50	30 – 65	2.25 – 2.55
0.38 – 0.52	0.450	0.45 – 0.75	50 – 90	2.50 – 2.80
0.52 – 0.68	0.600	0.70 – 1.00	70 – 120	2.75 – 2.90
0.68 – 0.88	0.780	0.95 – 1.25	110 – 160	2.85 – 3.00
0.88 – 1.10	0.990	1.20 – 1.50	145 – 205	2.95 – 3.20
1.10 – 1.37	1.235	1.45 – 1.70	195 – 250	3.15 – 3.40
1.37 – 1.68	1.525	1.65 – 1.90	235 – 300	3.35 – 3.60
1.68 – 2.03	1.855	> 1.85	> 285	> 3.55

Rock and rock mass properties have been used by different researchers for the determination of charge factor (specific charge) or to define blastability of rock. According to Rustan (1998), blastability is the ability of rock or any material to fragment when being blasted. An overview of rock and rock mass parameters used by different researchers to determine specific charge or blastability is presented in Table 3.

Table 3: Different rock and rockmass properties used by different researchers for determination of specific charge or blastability.

Name of the researchers	Parameters used for determination of specific charge or blastability index
Hino (1958)	Defined blastability index as ratio of compressive strength to tensile strength. Higher the value, easier will be fragmentation.
Broadbent (1974) Heinen & Dimock (1976)	Used in-situ seismic wave velocities for determination of specific charge in open pit copper mines.
Ashby (Hoek & Bray, 1977)	Developed for the Bougainville Copper Mine based on fracture frequency and Joint shear strength.
Langefors and Kihlstrom (1978)	Used rock constant 'c' to determine specific charge.
Kutuzov (1979)	Used joint spacing, rock density and uniaxial compressive strength for determination of charge factor for general bench blasting.
Borquez (1981)	Used RQD, joint alteration factor and joint strength to determine blastability factor (K_v) = 1.96 – 0.27 ln (ERQD) ERQD = RQD × Alteration factor.
Rustan et al., 1983 Rustan and Nie, 1987	Determined fragmentation gradient (n), K_{50} and critical burden based on impedance (Density, P-wave velocity) and rock structures and friction properties of the discontinuity.
Lilly (1986)	Defined blastability from rock mass description (RMD), joint plane spacing (JPS), joint plane orientation (JPO), specific gravity influence (SGI) and Hardness. BI = 0.5(RMD + JPS +JPO+ SGI + H)
Ghose (1988)	Developed blastability model for selection of specific charge for coal measure rocks in open pit blasting based on density, spacing of discontinuity, point load strength index, joint plane orientation.
Berta (1990)	Used impedance factor to explain the transfer of explosive energy to rock fragmentation for the selection of specific charge.
Mutluoglu et al. (1991)	Used seismic wave velocity for optimization of specific charge in coal and lignite mines.
Adhikari (1994)	Used density (ρ), rock types and degree of jointing to determine specific charge for bench blasting: $q = a + b\rho$, a&b are rock coefficients.
Scott (1996)	Used dynamic compressive strength, density, Young's Modulus, block size, structures, target fragment size, heave, confine scale, water for blastability model to select charge factor for dragline bench, cast blasting in dragline bench and shovel operation in coal measure strata.

Apart from different properties of rock and rockmass, some researchers used drilling indices or rock quality index (RQI) to define the blastability and specific charge required for blasting

(Leighton et al., 1982; Lopez and Muniz, 1987, Muftuoglu et al., 1991; Jemino et al., 1995; Yin and Liu, 2001). Amongst the rock mass properties, joints (discontinuities) have maximum influence on rock fragmentation. Figures 2 to 4 show few types of joints found in some of the opencast mines. Following are the nature and type of joints that have influence on blasting.

(1) Dominant Joints Parallel to the Face

If the dominant joint is parallel to the face, fractures between boreholes will prematurely link. The premature linking will cause coarse or blocky burden fragmentation. Endbreak will be severe. Borehole spacing can be increased and fragmentation size will then decrease. If the intent of the blast is to produce rip-rap, a reduction of explosive load with close borehole spacing in the direction of the dominant joints will accomplish the task.

(2) Joints Perpendicular to Face

When the dominant joint direction is perpendicular to the face, little endbreak will occur. However, backbreak will be significant. If large blastholes are used and many dominant joints occur between holes along a row, blocky breakage will occur between holes. The blocky breakage can be corrected by reducing spacing, but the backbreak may get worse as spacing is reduced. The use of smaller blastholes with a better distribution of explosive in the rock mass may be the best solution.

(3) Joints at an Angle with Face

When dominant joints are at an angle with the face (preferably between 25^0 and 65^0), fragmentation is good and both endbreak and backbreak are normally within acceptable limits.

(4) Joints at Less than 30° Angle to Face

Joints that form an acute angle with the face cause both breakage and wall stability problems. Burden breakage will be blocky and the back wall will be shattered, rough and broken.

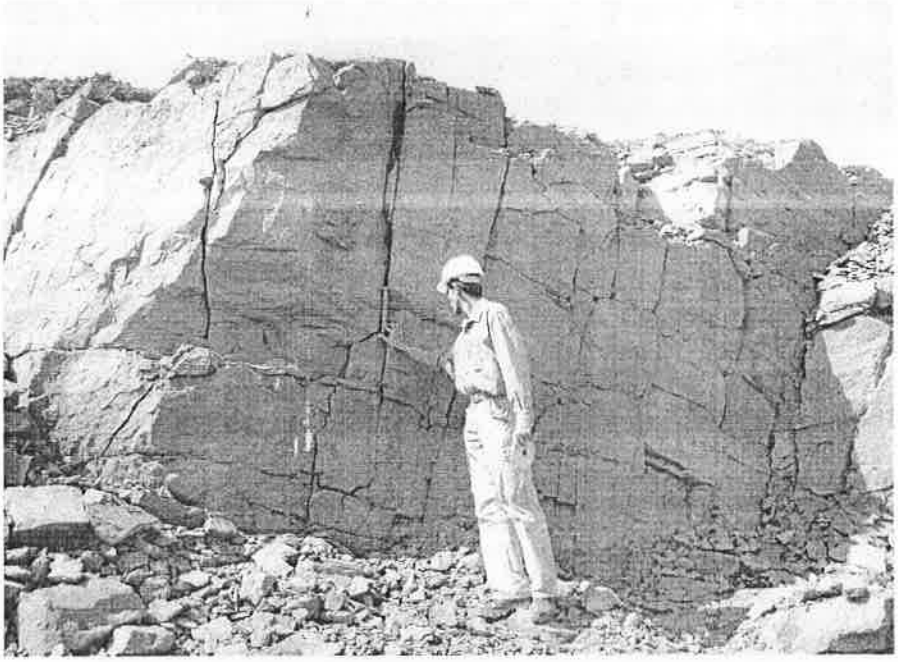

Fig. 2: Vertical joint almost perpendicular to face (Murgabeda Iron Ore Mine)

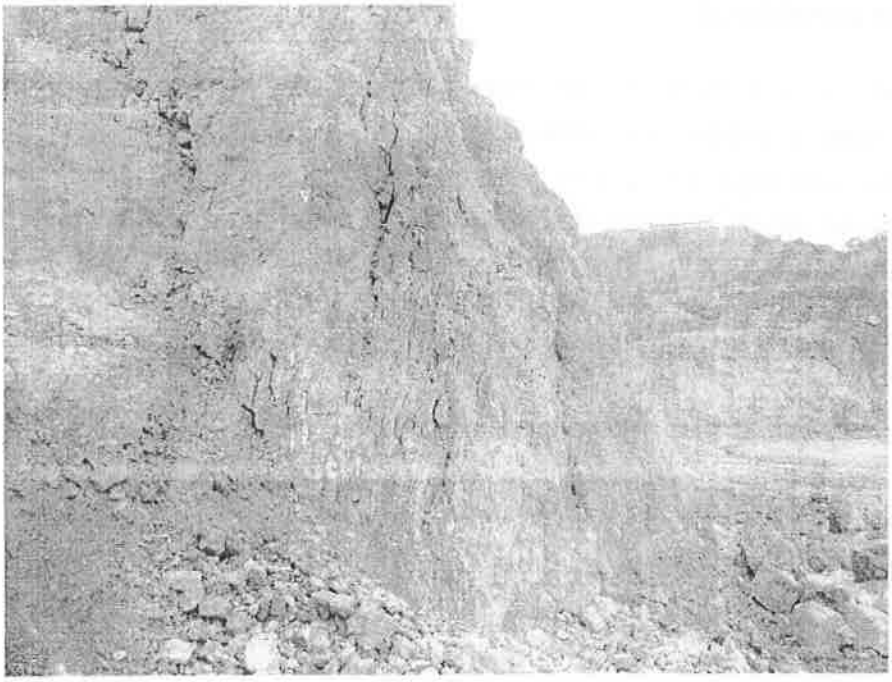

Fig. 3: Heavily jointed limestone strata at Adunik Limestone Mine (Meghalaya)

Fig. 4: Joint plane oblique towards face (Banduhurang opencast Mine, UCIL

4. PROPERTY OF EXPLOSIVES

An explosive material is a material that either is chemically or otherwise energetically unstable or produces a sudden expansion of the material usually accompanied by the production of heat and large changes in pressure (and typically also a flash and/or loud noise) upon initiation which is called the explosion. Commercial explosives are different from those high military explosives. The most important properties of commercial explosives are:

(1) Strength and energy
(2) Detonation velocity (VOD)
(3) Density
(4) Water resistance
(5) Sensitivity (controlled action and uncontrolled action)
(6) Fumes

Strength is one of the most important properties of explosive. There are different ways to measure strength of an commercial explosive. The most common terms to express strength of explosive are Bulk Strength and Weight Strength. Bulk strength is the energy per unit volume of explosive (kJ/m^3) and weight strength is the energy per unit mass of explosive (MJ/kg). This is also called as absolute weight strength (ABS). When the bulk strength and weight strength of an explosive are referred to a standard explosive such as ANFO, then they are called as Relative Bulk Strength (RBS) or Relative Weight Strength (RWS). Relative weight strength and relative bulk strength of an explosive can be measured by the following equations.

$$RWS = \frac{(AWS \times 100)}{(AWS\ of\ ANFO)}$$

$$RBS = \frac{(RWS \times Density)}{(Density\ of\ ANFO)}$$

The density of the majority of explosives varies between 0.8 and 1.6 g/cc. Explosives with higher density are having greater strength and more breakage power. The density of an explosive is an important factor in calculating the necessary amount of charge for a blasthole. The lineal charge concentration of an explosive (Q_l) in a blasthole of diameter (D) and a density ρ_e can be calculated from the simple formula as:

$$Q_l = 7.854 \times 10^{-4} \times \rho_e \times D^2$$

Where,

Q_l = Quantity of explosive charge per length (kg/m)

ρ_e = density of explosives (gm/cc)

D = Explosive charge diameter (mm).

Detonation velocity refers to the speed with which the detonation wave is propagated through the explosives. It is the parameter which defines the rhythm of energy release. The factors that affect detonation velocity are charge density, diameter, confinement and aging of the explosive. The detonation velocity of an explosive increases with an increase in density, diameter and confinement. However, with an increase in aging of the explosive detonation velocity generally decrease. The detonation pressure of an explosive is a function of its density and square of the detonation velocity.

$$P = 432 \times 10^{-6} \times \rho_e \times \frac{VOD^2}{1 + 0.8\rho_e}$$

Where,

P = detonation pressure (MPa)

ρ_e = density of explosives (gm/cc)

VOD = Velocity of detonation (m/s).

Water resistance is the capacity to resist a prolonged exposure to water without losing its characteristics. Most of the explosives are made of water resistance except ANFO. Sensitivity of explosive can have two meaning viz. Control action and uncontrolled action. Controlled action refers to the sensitivity to detonation such as by the blasting cap. The uncontrolled action refers to sensitivity to shock, friction and heat which are associated with safe handling of the explosives. The detonation of any commercial explosive produces stream, nitrogen, carbon dioxide and other toxic gases such as carbon monoxide and nitrogen oxides (nitrous fumes).

5.0 CONCLUSION

Understanding the various mechanisms of rock breakage is important in any surface blast design. The various mechanisms of rock breakage are crushing of rock, radial fracturing, deflection breakage or spalling, gas extension fractures, fracturing by release of load, fracturing along boundaries of modulus contrast of shear fracturing, breakage by flexion and fracture by in-flight collisions.

Rock and rock mass properties greatly influence blast fragmentation and the total blasting performance. Amongst the rock mass properties, joints and their characteristics have maximum influence on blast performance. Several researchers developed blastability index to correlate powder factors with rock and rock mass properties. Explosive properties have similar significant impact on blast performance. Selection of explosive types to be used for a project site depends on its properties and rock geology. The most important properties of explosive are strength and energy, detonation velocity, density, water resistance, sensitivity and fumes

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Developments in Blasthole Geometry, Initiation Sequence and Flyrock Control

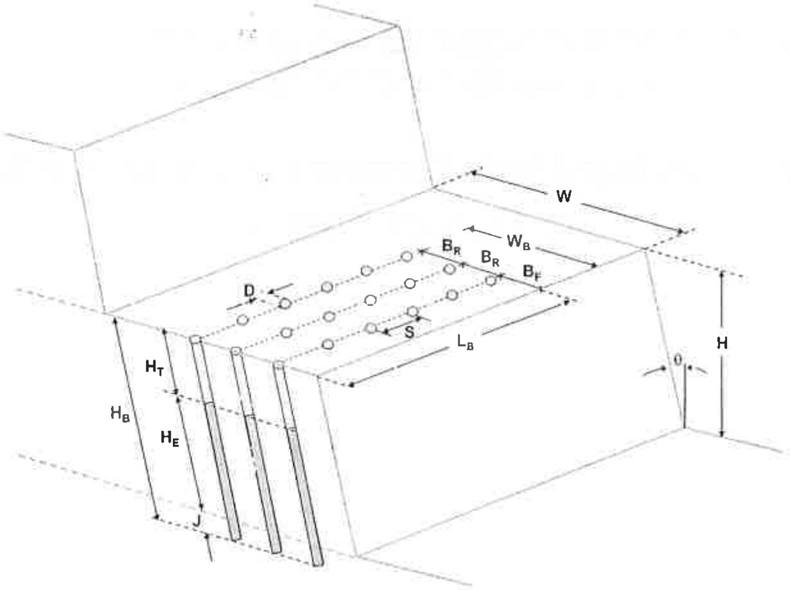
1. INTRODUCTION

The general objective of rock breakage by blasting is to obtain desired fragmentation with minimum disturbance to surrounding environment. This objective can be achieved by better understanding of the rock geology as well as judicious selection of different blast design parameters. An optimal blast design not only results in proper fragmentation but also reduces undesirable side effects such as flyrock, ground vibration and noises. Proper fragmentation of blasted rocks improves the efficiency of subsequent operations viz. Loading, transport and crushing to required size. Considerable developments have been taken place in drilling and blasting of rock during the past decades. A wide range of drilling machines is now able to drill different blasthole diameter with faster rate and less blasthole deviation. The use of conventional safety fuse for initiation is now almost removed by other advanced initiation system like electric detonator, detonating cord, non-electric detonator (Nonel) and electronic detonators.

With the improvement in initiation systems, better rock fragmentation can be achieved and the ill-effect of blasting can be reduced to controllable limits. However, the basic principle of rock blasting remains the same. In this paper, different basic blast design parameters will be reviewed. The different initiation sequence in bench blasting and delay timing will also be discussed. The general causes of flyrock and their possible reduction are also included in the paper.

2.0 BASIC BLAST DESIGN PARAMETERS

The basic blast design parameters in any bench blasting are: blasthole diameter, bench height, blasthole length, inclination of hole, burden (nominal burden and effective burden), spacing nominal spacing and effective spacing), sub-drilling, Stemming length, length and width of the blast, explosive types and initiation system, explosive charge weight and powder factors. The different blast design parameters are shown in Figure 1.



H	=	Bench height	B_F	=	Front burden
W	=	Bench width	B_R	=	Burden between rows
θ	=	Slope angle of the bench face	S	=	Spacing of holes
D	=	Diameter of hole	H_E	=	Explosive charge length
H_B	=	Blasthole depth	H_T	=	Stemming length
L_B	=	Blasting length	J	=	Sub-drill length
L_W	=	Blasting width			

Fig.1: Different blast design parameters in bench blasting

2.1 Blasthole Diameter

The choice of hole diameter depends mainly on degree of required fragmentation, amount of rock to be executed (work time scheduled and time limit), sensitiveness of the surrounding environment, bench height, capacity of loading equipments and geology of the formation. It also depends on the overall economics in relation to the initial investment and the operating cost. Better and finer fragmentation could be achieved with smaller hole diameter. In sensitive environments usage of larger blasthole diameter may not be feasible. Control of ground vibration and flyrock are also much easier in smaller diameter than larger hole diameter. However, drilling cost generally increase as blasthole diameter decreases.

2.2 Bench Height

The selection for an optimum bench height and width depends on inherent stability of the formation, thickness of the formation, drilling and loading equipment to be deployed etc. Higher bench height required more blasthole length and larger drill diameter. This could also result in ground vibration and flyrock problems when the dwelling areas and different structures are existed near by the blasting site. The ratio between bench height (H) and burden (B) is called 'stiffness ratio'. Higher values of stiffness ratio can be obtained with greater bench height. When the stiffness ratio is high, it is easy to displace and deform rock as shown in Figure 2. According to Ash (1963), the optimum stiffness ratio is greater than or equal to 3.

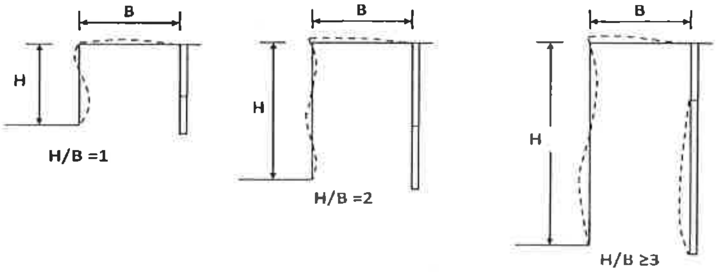


Fig. 2: Bending condition in bench blasting with different stiffness ration (*After Ash, 1977*)

2.3 Hole Depth

The required hole depth depends on the bench height, inclination of hole and sub-grade drilling. Sub-drilling, on the other hand, depends on the strata condition at the toe portion. With a horizontal bedding plane and softer formation at the toe, use of sub-grade drilling may not be necessary. However, with higher dip of bedding plane or presence of harder strata at the toe portion, more sub-drilling length is required to avoid toe problem and irregular floor. Length of sub-grade drilling generally varies between zero and 0.3 times the burden.

2.4 Burden and Spacing

Burden is the minimum distance from the axis of a blasthole and the free face whereas spacing is the distance between blastholes in the same row. The values of burden and spacing depend upon blasthole diameter, properties of rock and explosive, bench height and the desired degree of fragmentation as well as muck displacement. Depending upon the properties of rock mass, burden value generally varies between 25 and 40 times the hole diameter.

Different researchers have develcped have suggested numerous empirical equations to calculate burden value. Out of these formula, the most commonly used equation for the calculation of burden value as given by Konya & Walter (1991) is:

$$B = \left[\frac{2\rho_e}{\rho_r} + 1.5\right] \times D_e$$

Where,

B = Burden in inches

ρ_e = Specific gravity of explosive

ρ_r = Specific gravity of rock

D_e = Diameter of explosive in inches

Pal Roy (2005) developed burden equation based on hole depth, bench height, loading density and Rock Quality Designation (RQD) of rock. The equation is:

$$B = H.\left[\frac{D_e}{D_h}\right].\left[\frac{5.93}{RQD}\right] + 0.37.\left[\frac{L}{C}\right]^{0.5}$$

Where,

B = Burden (m),
H = Bench height (m);
D_e = Diameter of explosive (mm);
D_h = Diameter of blasthole (mm);
RQD = Rock Quality Designation;
L = Loading density of explosive (kg/m) and
C = Charge factor (kg/m^3)

The value of spacing is calculated in association with the burden, delay timing between blastholes and the initiation sequence. In general, spacing value varied between 1.2 and 2.0 times the burden value. Very small spacing causes excessive crushing between charges and superficial crater breakage, large block in front of the blastholes and toe problem. Whereas excessive spacing between blastholes causes inadequate fracturing between charges, toe problems and irregular face as depicted in Figure 3 (Dick et al., 1983).

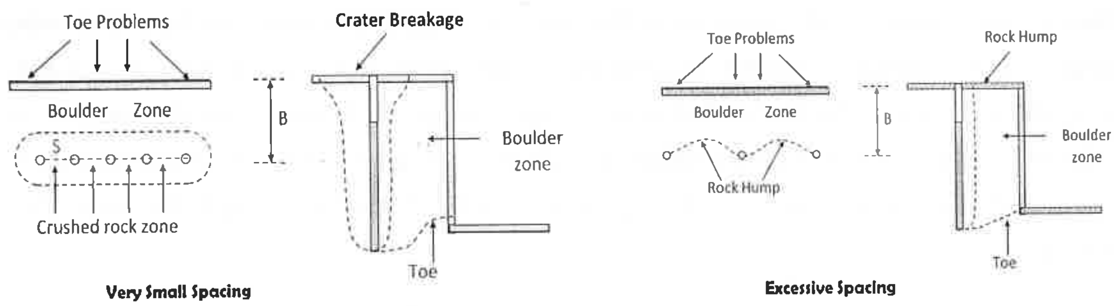


Fig. 3: Influence of spacing in bench blasting (*After Dick et al., 1973*)

2.5 Sub-drilling

Sub-drilling is the length of the blasthole below the floor level. The length of sub-drill depends on the rock formation at the floor level. In hard rock floor, if the sub-drilling is small, the rock will not be completely sheared off at the floor level. This will result into toe problem, leading to loading cost. However, if the sub-drilling is excessive, this could lead to increase in drilling cost, blasting cost and increase in ground vibration level. The value of sub-drilling is generally 0.3 times the burden value.

2.6 Stemming Length

Stemming is the portion of blast hole which has been packed with inert material above the charge so as to confine and retain the gases produced by the explosion before the actual burden movement. Stemming length depends upon the nature of rock blasted, required throw, fragmentation as well as the type and size of stemming materials. Stemming length can be varied widely, ranging between 20 and 60 times hole diameter. Whenever possible, stemming length of more than 25 times the blasthole diameter should be maintained in order to avoid flyrock, airblast, cutoffs and overbreak.

2.7 Explosive Type

The type of explosives to be used depends on properties of rock to be fragmented, ground water condition and availability in market. In hard and massive formations, explosive with higher density and higher strength is required for proper fragmentation. However, in softer formation and heavily jointed rock mass, low density explosive with lower strength may be used.

2.8 Specific Charge/Powder Factor

The quantity of explosive (kg) required to fragment one cubic metre of rock is called as specific charge or powder factor (kg/m^3). The specific charge increases with an increase in diameter of blasthole, rock strength, degree of fragmentation, displacement and swelling desired. The wide range of specific chare for different types for rock in case of surface bench blasting is given in **Table 4**.

Table 4: Ranges of specific charge for bench blasting in surface mines (after Jimeno et al, 1995)

Types of rock	Specific charge (kg/m^3)
Massive and high strength rock	0.60 – 1.50
Medium strength rock	0.30 – 0.60
Highly fissured rocks, weathered or soft	0.10 – 0.30

3.0 INITIATION SEQUENCE

The introduction of millisecond delays between holes or between rows resulted into better blast fragmentation, reduction in ground vibration and help in controlling flyrock. With systematic design of firing sequences, muck displacement, swelling of the rock and overbreak can be controlled. Konya and Walter (1990) described various initiation patterns such as V-cut (square and angular cut), *Box-cut* (alternating delays), *angled corner-cut* (fired on echelon, simultaneously along rows and progressive delays), *equilateral triangle pattern*, *progressive delay firing* along holes in a row etc. However, initiation sequences most commonly practice in bench blasting operation are:

(1) Single row, hole-to-hole blasting
(2) Multi-row, row-to-row blasting
(3) Multi-row diagonal pattern of firing
(4) Multi-row V-cut pattern (Narrow and Wide V-Cut).

The diagonal pattern and V-cut pattern of blasting, designed for ore blast using Noiseless Trunkline Delay (NTD) in one of the working benches at Banduhurang Opencast Mine of Uranium Corporation of India Limited (UCIL) are given in Figures 4 to 6.

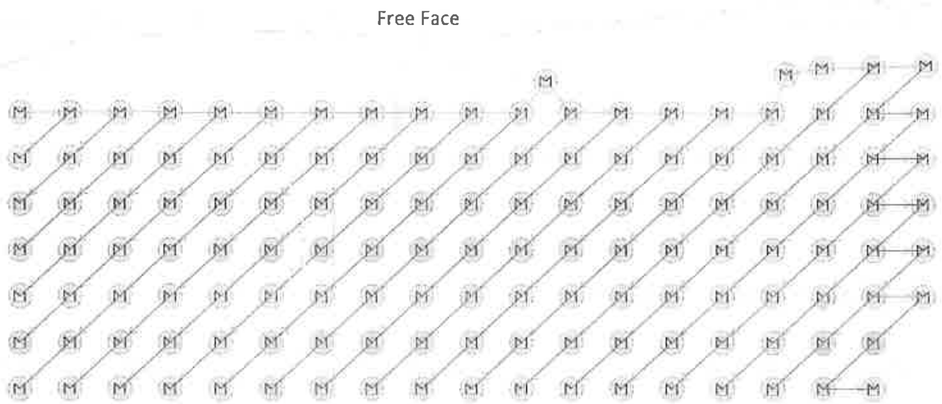


Fig. 4: Diagonal pattern of firing

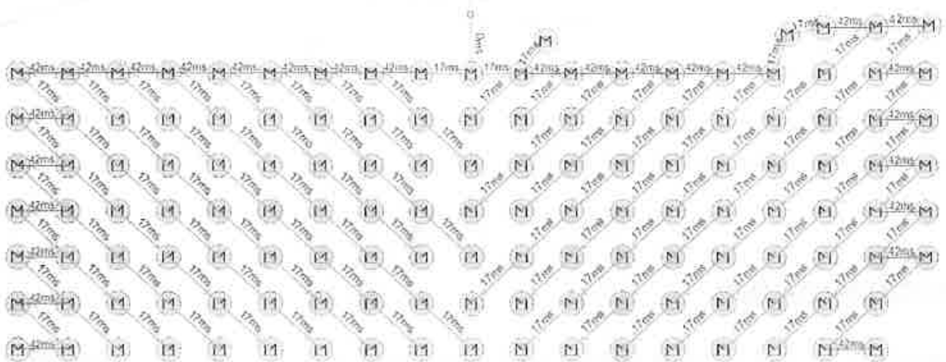


Fig. 5: Narrow V-cut Pattern of firing

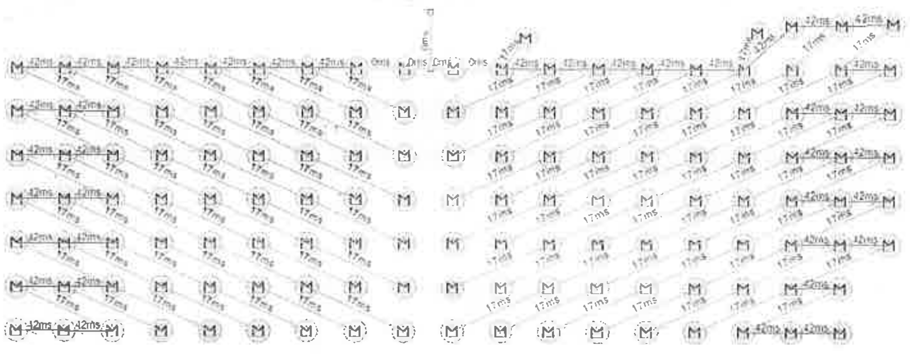


Fig. 6: Wide V-cut Pattern of firing

One of the most important significant of initiation sequences is modification of burden and spacing values into effective burden and effective spacing. In Figure 7, the designed values of burden (B) and spacing (S) can be modified into effective burden (B_e) and effective spacing (S_e). Using proper initiation sequences, the effective burden can be reduced and effective spacing value can be increased. This results in better rock fragmentation.

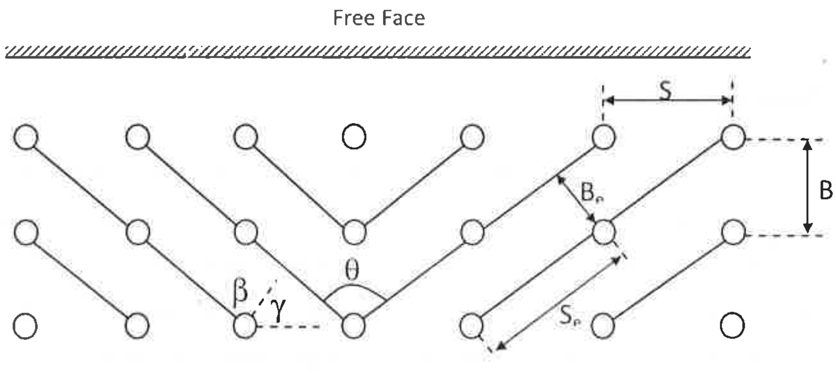


Fig. 7: Simple V-cut pattern of initiation

The important criteria to be followed in case of multi-row sequence initiation in bench blasting with single free face are:

- Each charge should have a free face at the moment of detonation.
- The relationship S_e/B_e should be between 3 and 8, and preferably 4 and 7.
- The blasthole should be on a staggered pattern with a high degree of equilibrium.

- The rows with the same delay should form an angle θ of 90° and 160° and preferably between 120 and 140°.

- The angles β and γ, which form the principal direction of rock movement and the side(s) and rear excavation boundaries, should be as large as possible to minimize disruption of new faces.

The general rules for increasing rock displacement in multi-row sequenced bench blasting are:

- Decrease the value of effective spacing
- Increase the angle of breakage (θ)
- Increase the number of holes with an adequate effective free face

In order to achieve faces in acceptable condition, the following should be carried out.

- Increase the angles β and γ, which form the principal direction of rock movement and the side(s) and rear excavation boundaries

- Increase the surfaces of the effective faces in the perimetral blastholes.

- Lower the value of B_e in the contour holes.

3.1 Delay Timings

The delay timing plays a fundamental role in the fulfilment of adequate rock fragmentation, swelling of muck and displacement, control of flyrock, ground vibration and overbreak. Many researchers recommended the effective delay timing between holes. Bergman et al., (1974), Langefor & Kihlstrom (1971) recommended that delay timing between blasthole should be 3 to 6 ms per meter of burden. Andrews (1981) established a low limit of 3 ms/m and high limit of 16.6 ms/m. Fadeev et al (1987) established equation for the effective delay timing based on rock density and powder factor as:

$$\text{TRB} = 2\left(\frac{\rho_r}{CE}\right)^{1/2}$$

Where

TRB = Delay Time between Blastholes { 4 – 8 ms/m of burden}

ρ_r = Rock density (t/m^3)

CE = Powder Factor (kg/m^2)

Delay Time between Rows (TRF) = 2 - 3 TRB

The delay timings, according to Lang and Favreau (1972) should permit the succession of the following events.

- Propagation of the compression and tensile waves from the blasthole to the free face (appx. 0.58 ms/m).

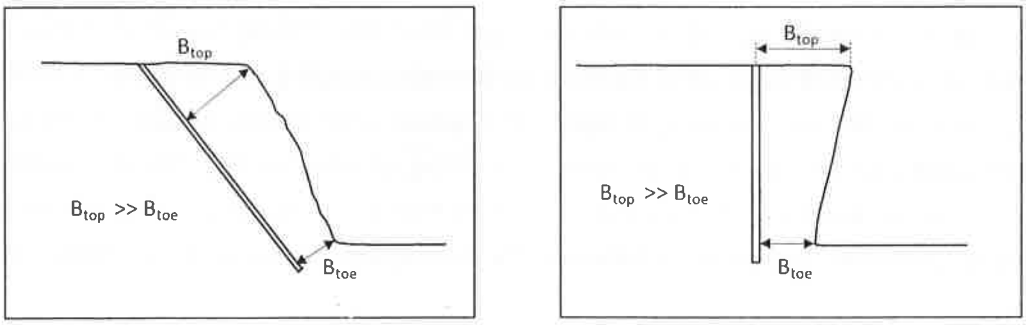
- Readjustment of the initial field of tensions, due to the presence of radial cracks and the effect of the reflection of the shock wave on the free face. The adjustment time can be estimated between 10 to 20 ms after the initiation, depending on the type of rock and explosive.

- Acceleration of the fragmented rock by action of the gases, up to a velocity that assures an adequate horizontal displacement. The larger the timing, the better the movement, estimated between 30 and 50 ms after initiation.

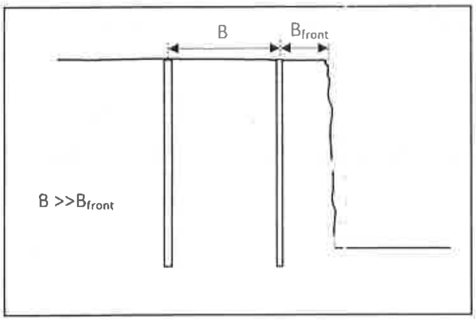
4.0 FLYROCK AND ITS CONTROL

Flyrock is the uncontrolled propelling of rock fragments produced in blasting. Amongst the undesirable side effects produced by blasting, flyrock is the most harmful effect which can cause direct fatality. It also constitutes one of the main sources of material damage. It is hardly seen or reported direct fatality due to ground vibration and noise (air overpressure) generated from blasting operation. However, flyrock contributes major fatality accidents in opencast mines.

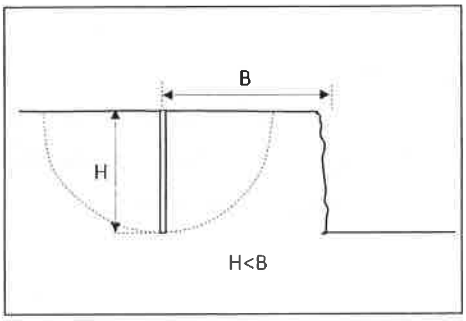
The causes of flyrock are mainly attributed to the poor blast design, inadequate stemming column, geology of the site, irregular shape of the face, inaccurate drilling, excessive explosive and too much hole inclination The common causes of flyrock in opencast mines/quarries are given in Figure 8.



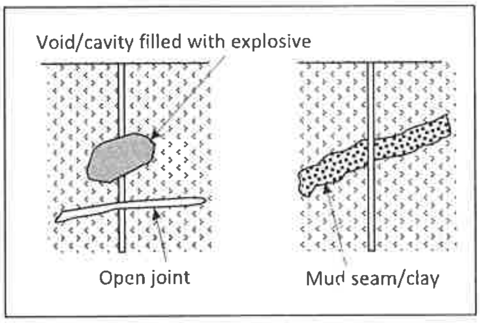
(A) Incline hole causing less toe burden **(B) Under-confined at the toe**



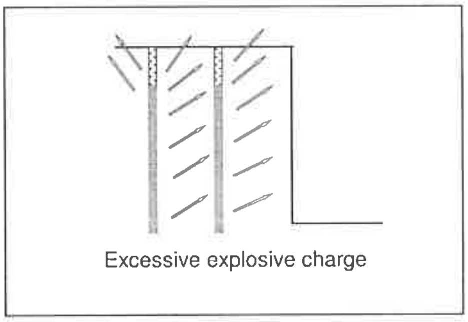
(C) Too less front burden



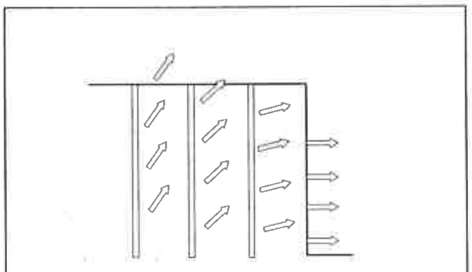
(D) Too large front burden



(E) Cavity, open joint and mud seam



(F) Excessive charge and very small top stemming



(G) Inadequate delay timing (Back rows)

Fig. 5: Common causes of fly rocks in bench blasting in surface mines

4.1 Flyrock Control

The most effective way to control flyrock is through proper blast design and careful monitoring of the total blasting operations, right from the drilling operation to final firing of the blasting round. It is difficult to perform systematic charging of blastholes, if holes are not drilled properly. Before charging of holes, the actual hole depth should be measured and the face condition should be inspected. Final firing of the blasting round should be done only when all the workers are taking proper shelter at a safe distance. The following are the important control measures to be followed for flyrock occurrence in bench blasting.

(i) The primary means of controlling flyrock is through proper blast design and delay timing. The consistency of the burden, specially the front burden (distance between the first row to free face) must be maintained.

(ii) Bench height to burden ratio less than 1.5 should be avoided. The spacing of about 1.5 to a maximum of 3 times the burden is suggested to reduce the flyrock.

(iii) While loading a shot, the blaster must be aware of his true powder factor in terms of the amount of explosive to be charged for the quantity of rock to be fragmented. Charging of excessive explosive quantity must be avoided.

(iv) When firing more than one row of holes, sufficient delays should be used between two sequences of rows or holes. Based on the observations, delays of at least 25 ms should be used between two rows. When the number of rows increases, the delay timing should also be increased to 42/50/59 ms.

(v) Length of stemming column should be greater than or at least equal to 25 times the hole diameter. For better protection, it may also be taken as greater than or equal to the burden.

(vi) The blasting site should always be inspected before marking the holes. If any open joints and bedding planes are present in the bench, an adjustment should be made in the drilling pattern.

(vii) The holes should be drilled in conformity of the face. Wherever possible, vertical holes should be preferred against the inclined holes.

(viii) Before loading, blasting officials should always check the hole depths and ensure that the holes are drilled as per the blast design.

(ix) Any change in the blast design should be carefully considered from the standpoint of its potential effect on flyrock.

(x) All loosened pieces of the rock from the blasting site should be cleared before charging.

(xi) Statutory provisions should be strictly implemented.

4.2 Flyrock Control by Muffling

Muffling or covering the blasting area is usually required when blasting is going to conduct closed to the residential areas or important structures. The muffling system should comply to the following characteristics.

- Reduce weight and high resistance.
- Ease of union or overlapping of the elements.
- Permeability to gases.
- Ease in placing and removing.
- Economical and reusable.
- Good size to cover large areas.

Figures 6 and 8 show muffling arrangement using old conveyor belts and sand bags in some of the controlled blasting operations conducted closed to important structures.

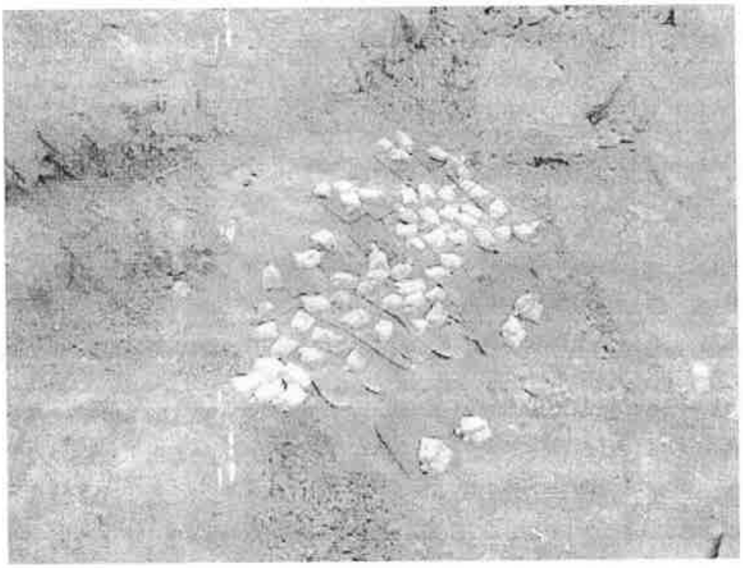


Fig.6: Muffling arrangement with conveyor belts and sand back at Millennium Centre, Aizawl Town, Mizoram

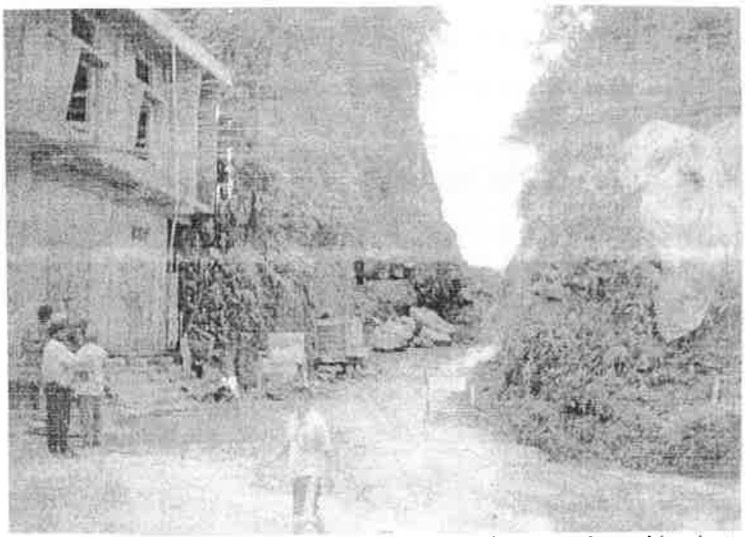


Fig.7: Muffling arrangement with conveyor belts and sand back at Durtlang-Leitan Area, Mizoram

Fig.8: Muffling arrangement with conveyor belts and sand back at Durgapur Project Limited (DPL) Area

5.0 CONCLUSION

Design of bench blasting in surface opencast mines and civil engineering constructions depends mainly on local geology, required degree of fragmentation, constraint on times schedule and the surrounding environments. The basic blast design parameters include blasthole diameter, hole depth, burden and spacing, stemming length, explosive charge (powder factor). These parameters depend largely on rock and rock mass properties which represent the local geology of the blasting area.

Blast fragmentation, much displacement and swelling can be controlled with proper selection of initiation sequences. Ground vibration and flyrock can also be controlled using systematic delay timing and initiation. The most common initiation sequences used in multi-row of holes are diagonal and V-cut pattern.

Flyrock can cause direct fatality and damage to structures. The most effective method of controlling flyrock is through proper blast design and careful monitoring of the total drilling and blasting operation. Muffling may be essential when blasting operation is conducted closed to residential areas and important structures.

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Techniques of Controlled Blasting for Mines, Tunnels and Construction Working – To Mitigate Various Blast Induced Adverse Effects

Abstract

Controlled blasting methods are used to control blast induced effects such as, over-break, fractures within remaining rock walls and ground vibrations etc. In both the mining and construction industries, blasting is the predominant method for fragmentation of consolidated mineral deposits. Adopting various techniques of controlled blasting such as Line drilling, Trim (Cushion) blasting, smooth (contour or perimeter) blasting, pre-splitting etc,; selecting and employing various parameters of blast design; using modern technology such as precise timing delays, varied density of explosives product by using bulk explosives; Muffle blasting at a very critical and congested areas are some of the points discussed in this paper for mitigation of adverse impact of blasting. By adopting these precautions not only the ground vibration is restricted to ease the public relation problem, but the mines' / construction's techno-economics, preservation of host rock strength and safety standard are improved to a considerable level.

1.0. Introduction:

In both the mining and construction industry, blasting is the predominant method for fragmentation of consolidated mineral deposits. Mines, quarries and construction sites that were formerly located relatively remote areas now find themselves near congested areas. The public relation problems of users of explosives have increased greatly within the past few years as explosives are being consumed in increasing quantities by mines and infrastructure development agencies. New expressways, underground metros are being constructed to connect the cities with downtown areas built through the middle of long-established business and residential neighbourhoods. Open pit mines and quarries are carrying out mega blasts very often, in order to enhance their production. Tunnel and construction blasting, now a days, emphasize more on reduction of blast induced back-break to preserve the property of host rock intact for safety and economic reasons. UG metal mines are very much keen to maximize value of their output by improving their quality standard of ore by preventing ore dilution at the stopes by reducing over-break of the wastes and wall rocks. They are also conscious about the back-fill done at the stopes, which should not get disturbed during mining and blasting process.

All blasting operation in open pits, quarry, tunneling, UG mines, constructions etc., how ever efficiently they are designed and executed, produces adverse effect to a considerable level. With the general trend towards larger blasting in mines; increased importance given for nation building such as construction of High rise buildings, Sub-way systems for underground highways, Railways & mass transport, hydroelectric installations, water diversion system etc.; increased population and spread of urbanization near to the mining sites; the effective mitigation of various nuisances of blasting to a significant level has become essential and mandatory. On the other hand, increased public awareness and increased involvement of various Govt. agencies, mining and environment laws and regulations are becoming more and more stringent every where in order to prevent damages of public properties, structures etc., and to tackle complaints from general public. Apart, various techno-economic aspects of efficiently running mines, prevention of ore dilution in stopes of UG mines, promoting safety in mines, promoting efficient workings of various construction activities, keeping the environment at high standard etc., call for minimizing adverse effects of blasting to a substantial level.

2. 0 Effects produced by any blast are:

- Displacement of pre-determined volume of rock
- Breakage of rock into well defined size elements
- Projection of displaced and broken rock to a certain distance from its original position
- Excessive breakage of part of the blasted rock
- Excessive throw of rock (fly-rock)
- Fractures and permanent deformation in the rock behind or side the blast (back-break or over-break)
- Ground vibrations
- Air-overpressure

Of the above effects shown some might be considered desirable and some might be considered non-desirable. The total amount of energy used in producing these desired and non-desired effects corresponds to the explosive energy to the rock, which is the energy of the explosive reduced by an impedance factor and a coupling factor.

3. 0 De-coupling of explosive charge –

The transfer of explosion energy to the rock is a function of both characteristics of explosives used and the characteristics of ground strata. An ideally charged hole, i.e., when charge and blast-hole diameter are almost similar (coupling is close to 1), shock pressure delivers to the side is maximum. For a de-coupled charge (i.e., charge diameter is less than hole diameter), the shock pressure on the side of the hole decreases exponentially with difference in the diameter. This principle of de-coupling of explosive charge is very important while designing blast for controlled blasting in order to reduce over-break. **Air-decoupled charge is the most effective means of reducing borehole pressure and consequently the peak stress level within the rock mass**. Thus, adopting air-decks (preferably by using air-bags) in pre-split holes, for mega / cast blasts in open pit mines minimizes over-break, ground vibration etc., to a great extent.

4. 0. Adverse effects of blasting

When blast holes are under or over charged and in the absence of proper free face, a great deal of liberated energy is wasted and converted into ground vibration and air blast (Noise), as explosion energy is not utilized in fragmenting / breaking of rock and throw. These undesirable vibration, air blast create nuisance in mining operations. Another negative aspect of blasting is Fly-rock; which is again the result of improper blast design and explosive selection. Thus, the adverse effects of blasting can be catagorised as:

4.1. Shock wave (Blast Vibration) generated –

Unwanted blast induced shock wave / vibration is responsible for major degradation of mining condition by creating nuisances in open pit & UG mines and construction & tunneling activities; in the way of followings:

a) Blast induced vibration directly responsible for damage of public structures and properties belonging to the inhabitants adjacent to the open pit blasting sites creating serious public relation problems.

b) High intensity shock wave generated by the blast is responsible for deterioration of condition of fragmentation in open pit mining by the way of side-break or back-break in blasted bench.

c) Blast induced vibration is responsible for deterioration of stability of high wall rock and OB dumps in open pit mines and quarries; affecting safety in mine workings.

d) High intensity shock wave generated by the blast is responsible for deterioration of wall rocks of Underground mines, construction workings etc., and affecting safety.

e) Because of side over-break in case of tunnels, difficult in maintaining desired dimension and preserving the condition of host rocks; affecting techno-economics of tunneling in way of putting artificial supports and additional concrete lining.

f) In case of foundation excavation, because of high intensity shock wave generated by the blast it is difficult in preserving the condition of host rocks; affecting techno-economics of construction.

g) In case of UG mining stopes, chances of ore dilution is more because of post blast over-break of wall-rocks and wastes. Additional cost also involved in maintaining support in UG stopes.

h) High intensity shock wave generated by the blast is responsible for deterioration of condition of stability of back-fill in Underground mining stopes, affecting safety.

4.2. Air blast or Noise

Air blast or air over-pressure is the result of a transient air pressure impulse generated by explosive blast. One of the major public relation problems of users of explosives face because of blast induced air blast in the way of noise. By proper blast design, using proper blasting accessories such as NONEL, electric & electronic detonators etc., air blast can be checked effectively. Air blast (noise) problem is more when using detonating fuse (DF) for open pit, quarry and construction blasting (i.e., when propagating detonating wave comes directly in contact with air); thus use of DF should be avoided in order to restrict air blast problem.

4.3. Fly-Rock

Fly-rock is the debris that is ejected or propelled through air by explosive blast. It remains a potential source of numerous hazards to people and surrounding objects. Excessive post blast fly-rock generated in open pit mines, quarries and construction blasting hampers safety to a great extent. The contributing factors for lack of blast area security and fly-rock are discontinuity in the geology and rock structure, improper blasthole layout and loading, insufficient burden, very high explosive concentration and inadequate stemming. It has also been observed that accidents due to lack of blast area security are caused by failure to use appropriate blasting shelter, failure to evacuate humans from the blast area, and inadequate guarding of the access roads leading to the blast area. Thus, by proper blast design, stemming, proper delay sequence and properly selecting explosives, blast induced fly-rock can be checked. It has also been seen that fly rock problem is more when using DF as it disturbs the condition of stemming in open pit quarries and for construction blasting. Muffled blasting is an effective way to restrict fly-rock from construction blasting, when these are carried out in congested areas.

4.4. Other nuisances like generation of Dust, Post detonation fumes etc., exist as bad effects of blasting (Note: Discussion on these points is not the scope of this paper).

5.0. Techniques of Controlled Blasting

Various controlled blasting techniques are used for reduction of over-break and propagation of vibration in mines and excavation work. However, all techniques have one common objective, that is, reduction and better distribution of explosives charge specially at the periphery of the blast in order to minimize stressing and fracturing of the rock beyond the exact excavation line. Controlled blasting techniques can be grouped into following categories:

5.1. Line Drilling

A term used in quarrying to describe the method of drilling and, if necessary, broaching for neat cut. This system involves a **single row of closely spaced uncharged holes along the neat excavation line**. This provides a plane of weakness to which the primary blast can break. It also causes some of the shock waves generated by the blast to be reflected, which reduces shattering and stressing in the finished wall of the host rock. Thus, preserving, to a great extent, the original strength of the host rock is possible.

Line drill holes are generally percussive hammer holes having spaced two to four times the hole diameter, drilled along the excavation line. The blast holes directly adjacent to the line drill holes (buffer holes) are generally loaded lighter (about 50% of primary holes) and are closely spaced (about 50 to 75 %) than primary holes. Line drilling is the best suited for homogeneous rock condition. This system is applied in very sensitive areas where even the light explosive associated with other controlled blasting technique may cause damage beyond excavation line. This technique gives maximum protection to the host rock to preserve its original strength. Apart, it has been observed that, Line drilling system with closed spacing can arrest the ground vibration to be propagated beyond the line drilling limit (excavation limit) to a great extent, as well. Because of these advantages, for construction excavations such as foundation excavation for high rise buildings / heavy machineries etc., this technique is widely used. The disadvantage of this system is high drilling cost due to closed spacing and results are often unsatisfactory because of poor hole alignment. This method is tedious and cumbersome than other techniques.

5.2. Trim (Cushion) blasting

Like line drilling, trim or cushion blasting involves a single row of holes along the specified final excavation line. This technique generally uses 2 to 4 inch diameter holes. Holes are **loaded with light charge**, well-distributed, completely stemmed and **fired after the main excavation is removed**. Hollow bamboo spacers may also be used to achieve better distribution of smaller cartridges in trim holes. By firing the trim holes with minimum or no delay between holes, the detonation tend to shear the rock web between holes and give a smooth wall with minimum over-break. In this system the main blasted muck is excavated leaving a minimum buffer or berm zone in front of final excavation line. It is better to put trim holes just before removing the final berm. To promote better shearing at the bottom of the hole, a bottom charge two to three times that used in upper portion of the hole is generally used. This technique is generally lesser in use for UG or tunnel blasting than quarry or construction blasting.

As compare to line drilling technique trim or cushion blasting is simpler and economical as increased hole spacing are used. Also, full geological information is obtained from main blast before going for trim blasting. The success of this technique lies on removing the blasted material completely before conducting trim blasting, which many a times not practicable specially in the case of ninety degree corner. Moreover, back-break / over-break from primary blast, many a times, completely or partially remove berm, which later difficult to perform trim blasting.

5.3. Smooth (Contour or Perimeter) blasting

A technique used (rarely in surface and mostly in underground blasting) in which a row or closely spaced drill holes are loaded with decoupled charges (charges with a smaller diameter than the drill hole) and fired simultaneously to produce an excavation contour without fracturing or damaging the rock behind or adjacent to the blasted face.

For promoting safety and economy in underground workings, performance of blasting in headings, drivages, tunnels and stopes becomes very important factor. An ideal blast results in a minimum of damage to the host rock with minimum of over-break. This is achieved by adopting suitable techniques of controlled blasting, which has many advantages –

i) Less rock damage means greater stability and less requirement of ground support,
ii) Drivage or tunneling operation is safer as less scaling is required,
iii) Less over break makes a smoother hydraulic surface for an unlined headings and tunnel,
iv) For a lined tunnel, less concrete required to fill excess void created by over break,
v) Lesser over-break in stopes means lesser dilution (Fig-1).

Smooth blasting some time refers to as contour or perimeter or sculpture blasting is the most widely accepted method for controlling over-break in underground headings, drivages, tunnels and stopes.

In this technique perimeter or contour holes are drilled along specified final excavation limits and are lightly loaded than that of buffer holes and production holes. The spacing is kept closer than buffer holes and production holes. Generally, as a thumb rule 10 to 12 times hole diameter in medium to tough rock and 5 to 6 times hole diameter in poor, fragmented rock are kept as spacing. In fact, burden and spacing of Buffer and Contour row should be about 75% and 40% that of production rows respectively. Loading and charging of contour holes are done with explosives of low VOD packed in small diameter cartridges in relation to drill diameter used. Unlike production drill hole blast where higher charge concentration is required, contour drill holes require low charge concentration and explosives should be lightly distributed all along the length of the bore hole. Some time use of high grammage Detonating Fuse (about 40 gm/m core wt., to 60 gm/m core wt.) for contour blasting can give effective result in tunneling. This results in an air cushion effect, which prevents over-break and reduces in-situ rock damage for preservation of strength of host rock.

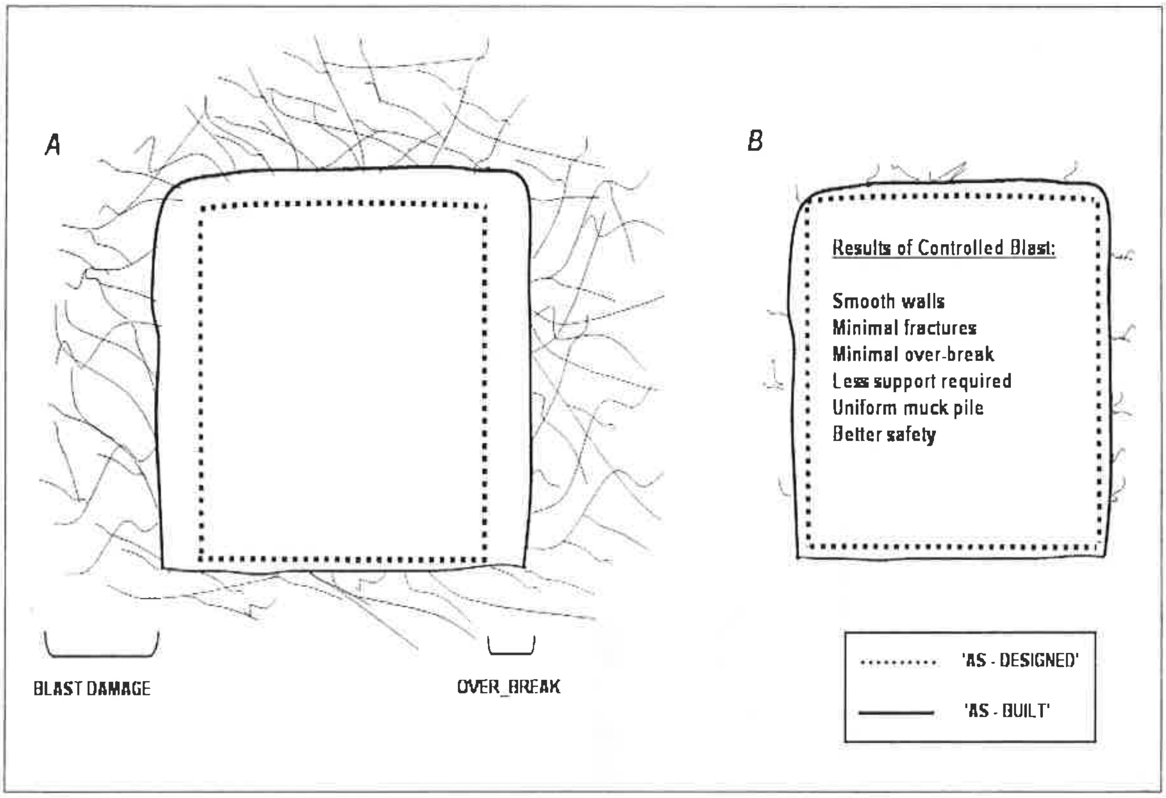
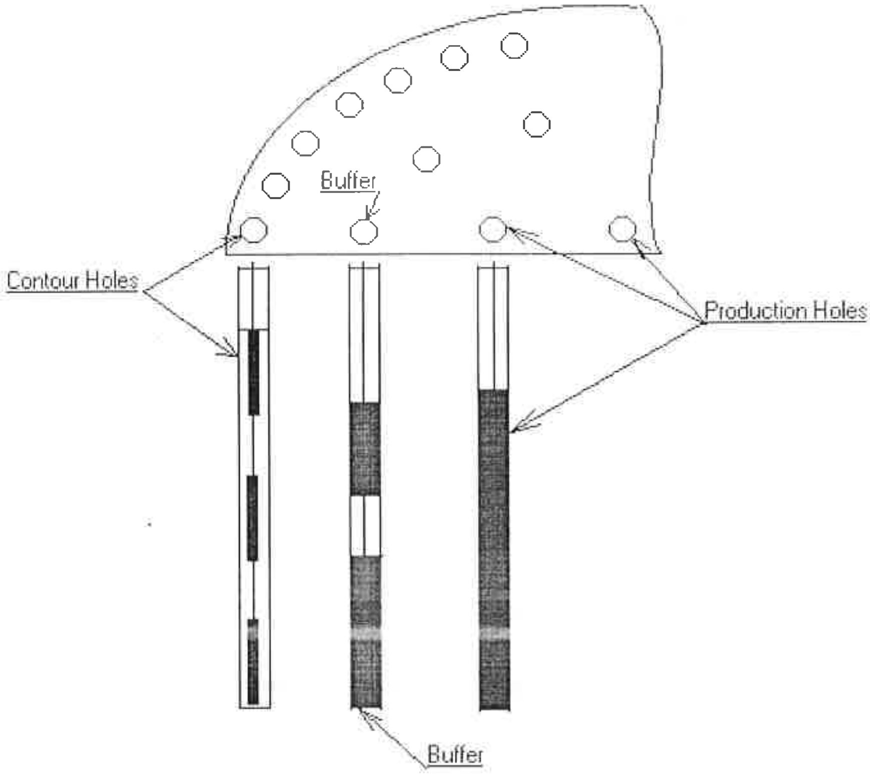


Figure - 1

Generally, these holes are fired after lifter holes to ensure maximum relief for the smooth blast holes. Other parameter to be looked into is charging of holes of Buffer row. In the buffer row the charge factor should not be more than 0.6 Kg/m³, to terminate the back break along the line of buffer row. At the same time, charge factor in the perimeter row should not be more than 0.4 Kg/m³ and it should be well distributed throughout the blast holes. Hollow bamboo spacers of 150 mm long may also be used to achieve better distribution of smaller cartridges in perimeter holes. A gradual reduction of charge factor from Cut holes at the center towards the perimeter of the tunnel will have efficient control on over-break. This gradual reduction of charge factor can be done in much better way if bulk explosive is used in UG workings, as site mixed bulk system is capable of delivering tailored density and energy product as per the need of strata condition and blast parameter required for contour or perimeter blasting (Fig – 2).

Observation of the blasted surfaces after the blast can give better clues to the accuracy of drilling and blasting and effectiveness of this technique. A measure of success is the **half-cast factor (**ratio of half-casts of the blast holes visible on the blasted surface to the total length of perimeter holes). Depending on the rock quality a half-cast factor of 50 to 80 percent can be said to be quite satisfactory. There are other means to verify the extent of rock damage behind the wall. This may be done using Seismic refraction techniques and borescope or permeability measurements in cored bore-holes. The extent of disturbed zone may vary from as little as 0.1 – 0.2 m with excellent perimeter or contour blasting to more than 2 m with un-controlled blasting.



CHARGING OF CONTOUR, BUFFER AND PRODUCTION HOLES

Fig - 2

5.4. Pre-splitting (pre-shearing)

Pre-splitting or pre-shearing is the smooth blasting method in which cracks for the final contour are created by blasting prior to the drilling of the rest of the holes for the blast pattern. Once the crack is made, it screens off the surroundings to some extent from ground vibrations in the main round. This is an effective way of restricting back-break and ground vibration in large open pit, quarry blasting. Pre-splitting helps in isolating blasting area from remaining rock mass by creating an artificial discontinuity along the final designed excavation line / plane against which subsequent main blast breaks. A row of holes are drilled at the periphery (three sides) of the main blasting block at a closer spacing, charged preferably with lesser quantity of explosives than the production blast and blasted prior to the main blast in an effort to create a fractured line and a reflective plane at the excavation limit or plane. Some of the shock waves from subsequent main blast are reflected at the pre-split plane which results in arresting a considerable portion of blast induced ground vibration generated in the main blast to propagate. This also helps in preventing back-break in case of bench blasting, which in turn improve fragmentation and generation of big boulders and reduction of secondary blasting.

5.4.1. Principle of Pre-splitting - The theory of pre-splitting is that when two charges are shot simultaneously in adjoining holes, collision of shock waves between the holes places the web in tension and causes cracking that give a sheared zone between the holes to open a narrow crack / separation along the three sides of the production or main block before the main blast goes-off. This results in a smooth wall with little or no over break. The pre-sheared plane reflect some of the shockwaves generated from the primary blasts that follow, which prevent them from being transmitted into the finished wall and minimizes shattering and over break (Fig – 3).

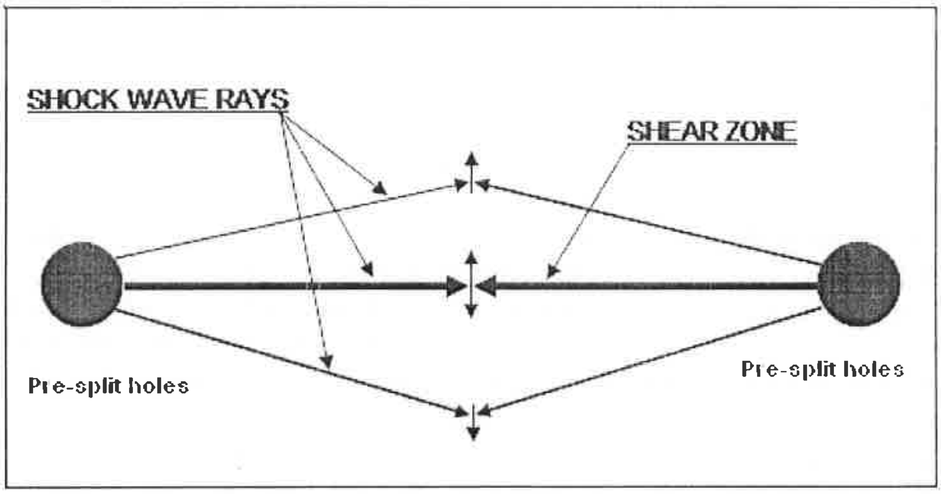
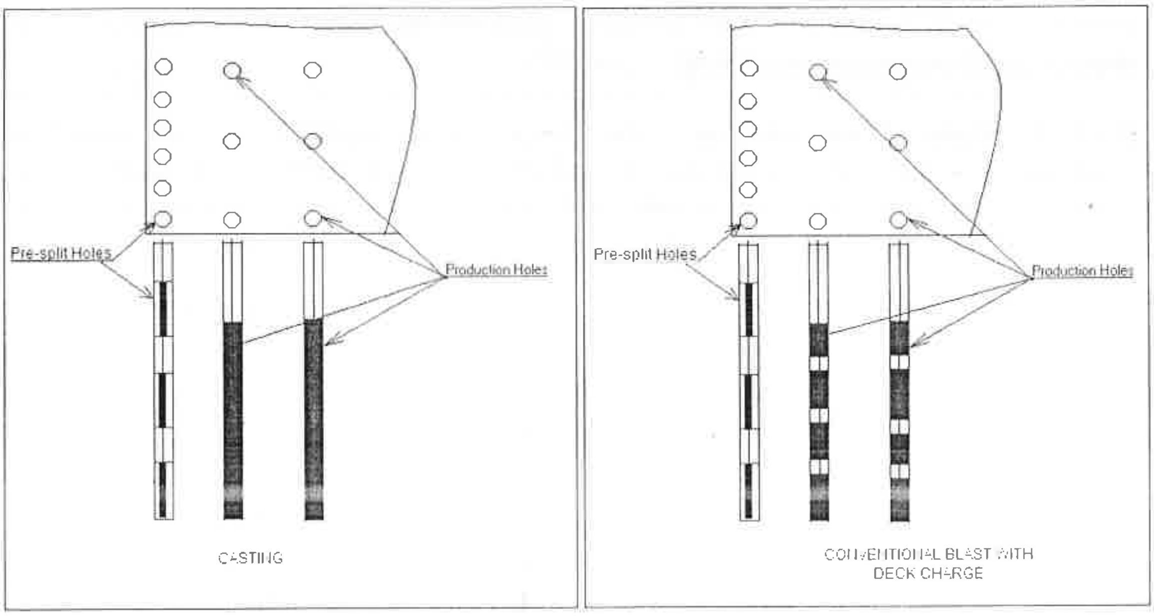


Fig - 3

The separation of timing between blasting of pre-splitting holes and production blast are kept with the help of delays. The delay gap of 200ms to 250ms between pre-split and main blast is considered to be enough.

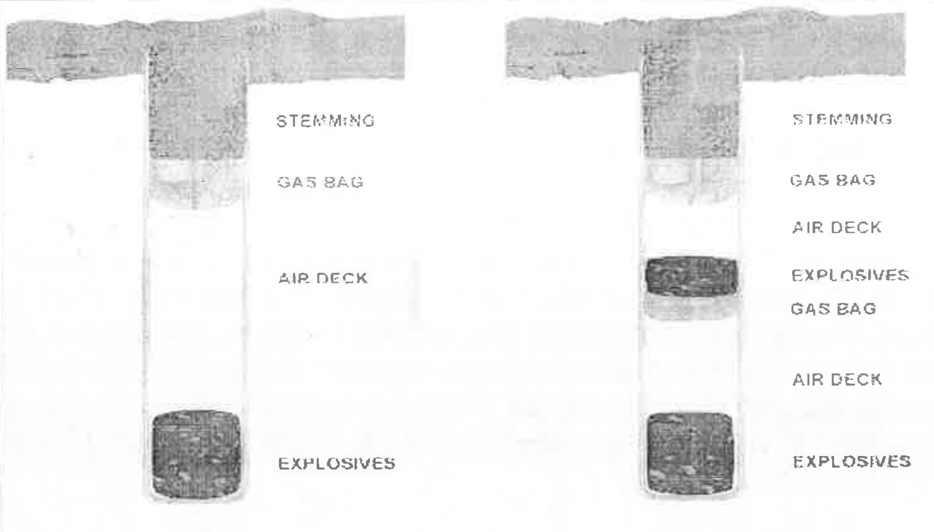
5.4.2. Charging of pre-split holes - The quantity of explosives to be used in pre-split holes, burden and spacing are estimated keeping in view the insitu tensile and compressive strength of rock mass. The borehole spacing of pre-split holes is normally kept at 8 to 12 times the blast hole diameter and the burden may be kept as of the burden of the Main Blast. Depth may be kept as of last row of main blast. Mostly, light distributed decoupled charges are used in pre-splitting holes. Air-deck in between deck charges improve the quality of pre-split fracture and avoid extension of radial cracks around the holes. Generally, the quantity of explosives kept in pre-splitting holes is 8 to 12 % of the explosives charged in one hole in the Main Blast (Fig-4).



CHARGING OF PRE-SPLIT AND PRODUCTION HOLES

Figure - 4

5.4.3. Air-decking of pre-split holes - It has also been tried to use **Air-deck** (by using **Air bags**) in pre-split holes in some of the mega blast / cast blast in open pit mines, in order to estimate the performance of blast, intensity of ground vibration percolated through the pre-split plane and the type of high wall left after the blast. The result of such blasts by using air-deck were very much encouraging with much reduction in noise (as lesser explosives used in pre-split holes as compare to non-air-decked pre-split blast), reduction in intensity of ground vibration propagated through pre-split plane, and leaving much better & competent high wall. Therefore, using **air-bags** for pre-splitting for mega blast in open pit mines is a good proposition to improve over all blast performance.



Air-decking by Air Bags of Pre-split Holes

5.4.4. Advantages of Pre-splitting of open pit blast -

(1) Field observation reveals that with the introduction of pre-splitting the back-breaks are eliminated, improving the stability of high-wall slopes and to provide uniform burden to the front row of holes for next blasting round.

(2) As back breaks are eliminated, formations of pre-formed boulders are reduced resulting better fragmentation in the subsequent blasts.

(3) Field observation reveals that, there is substantial reduction of ground vibration level to the tune of nearly 1/3rd of normal production blast due to pre-splitting.

(4) Pre-splitting is most suitable method of controlling ground vibration level in the case of Overburden Side Casting by Blasting.

(5) Mega blasts conducted in opencast mines, the interference of ground waves result. A very complex phenomenon of resultant waveform occurs, which is very difficult to control only by NONEL or Electronic Detonators. Pre-splitting of production blast is the best method of controlling or restricting the intensity of the waveform from propagating outside the Mega blasting zone and thereby, protecting of surface structures.

5.5. Muffle blasting

As discussed earlier, fly-rock is another important adverse impact of blasting operations, specially, when conducted in the vicinity of dense human habitation / congested areas. The factors which influence the fly-rock are: (1) density, strength and quantity of explosives used in relation to the spacing and burden of the pattern, (2) delay sequence used, (3) stemming column height, which should not be less than 1.2 to 1.4 of burden in critical areas, (4) condition of free-face etc.

These are blasting mats. They are used when the shots come near buildings to control flying rocks. They are overlapped like shingles over the blast site.

In case of blasting in congested areas, Muffling or covering of blast holes properly before blasting, is the common solution to prevent fly-rock from damaging human habitants and structures. Generally, mat or mesh (40 mm x 40 mm size) made of preferably of locally available steel ropes (5 to 6 mm) are used for muffling purpose. Sand bags weighing 40 to 50 kg are kept over the mesh at an interval of 3 m. Rolls of mesh should be kept over rows of holes in such a way that strip of mesh overlaps each other. It is a good practice to keep a portion of mat or mesh hanging over the free face to content fly-rock from escaping from the free face as well. It is to be seen, no blast area is kept uncovered with the mat or mesh and sand bags re placed properly. Efficiency of arresting of fly-rock depends mainly on the quality of muffling system implemented.

5.6. Using proper millisecond delay sequence and use of in-hole delays in decks

Delay blasting (with millisecond delays) permits the explosive engineers to divide the shot into smaller charges, which are detonated in a predetermined millisecond sequence at specific time intervals. Millisecond delay initiation of the explosive charge is a technique used in most open pit, quarry, tunnel and underground rock blasting operations. It serves to enhance fragmentation and direct rock movement for increasing productivity. Delay blasting is also used to manage adverse geologic conditions found on the site and to optimize the blast design and control of vibration and air blast. It can reduce ground vibration by dividing the explosive energy into smaller charges using a timing sequence and a delay interval, which provides for lateral relief for charges in the second and later rows in surface blasting operations. In case of mega / cast blasts the controlling of blast induced ground vibration, specially in sensitive areas, using multi-deck in-hole delays is a better proposition. The major advantages of delay blasting are:

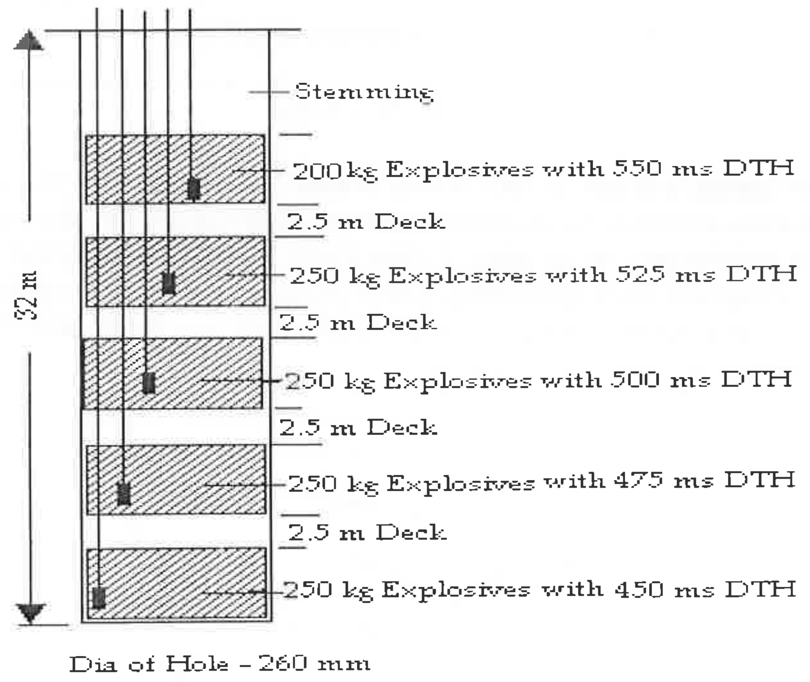
- Improved fragmentation
- Reduction of ground vibrations and air blast
- Reduction of over-break and fly-rock
- Improved productivity and lower cost

Charge weight per delay is the most important parameter for controlling blast induced ground vibration and air-blast. More the charge weight per delay in a blast, more is the intensity of blast induced ground vibration and air-blast for a given distance. A peculiar phenomena of vibration waveform arises in mega blast carried out in open pit mines, when ground vibration waves interfere with each other or superimposes each other and the resultant waveform is very much complex in nature.

Also, a strange aspect of ground vibration, now a days, coming to fore is increased problem due to another form of complex low frequency vibration wave arising out of presence of underground workings or cavity in the vicinity of open pit coal mines.

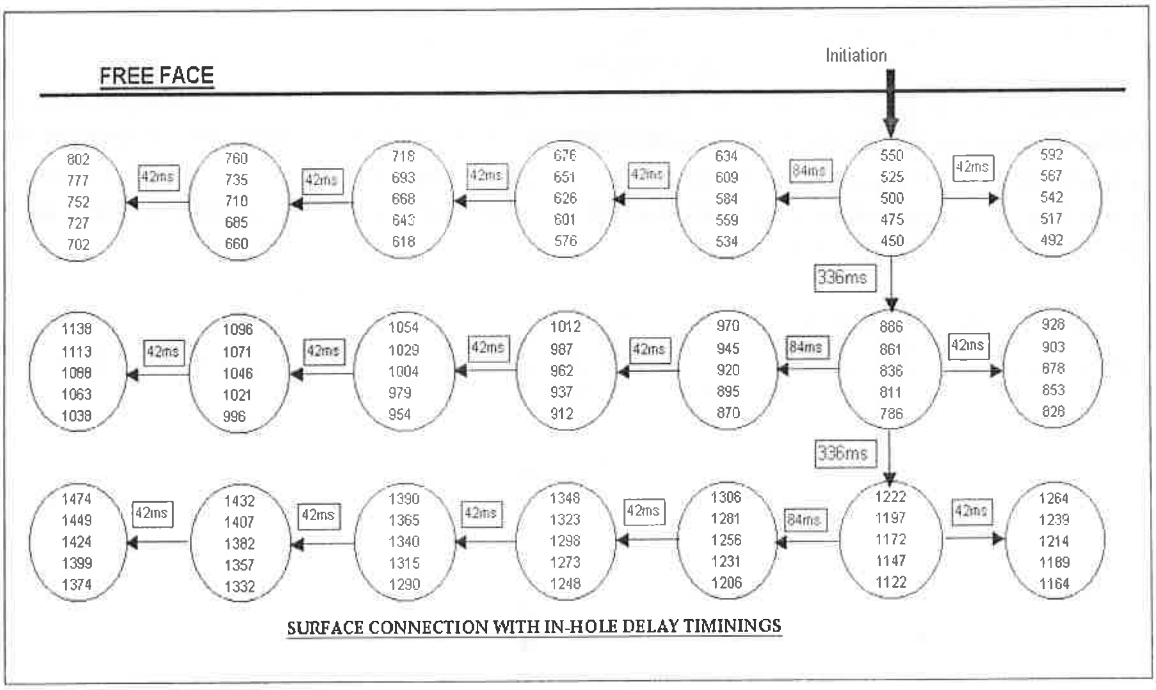
For both the above cases, by using proper MS delay sequence and use of in-hole delays with multi-deck, the intensity of such complex impact of blast can be controlled to a great extent (Fig-5 A and Fig- 5 B). Sometime, **combination of in-hole delays in multi-deck with pre-splitting of production blast can mitigate the effect of low frequency vibration to a great extent. Thus, it is advisable to practice this method of combination of pre-split with in-hole delays in multi-deck in** sensitive or problematic areas.

MS delays are incorporated into the blast design using electric or non-electric detonators or cord relays. In recent time, more accurate Electronic delay system has been introduced, resulting better blasting efficiency in terms of better fragmentation, improved over all costs, control of fly-rock & air-blast and effective reduction in blast induced ground vibration.



SECTION OF BLAST HOLE WITH IN-HOLE DELAYS

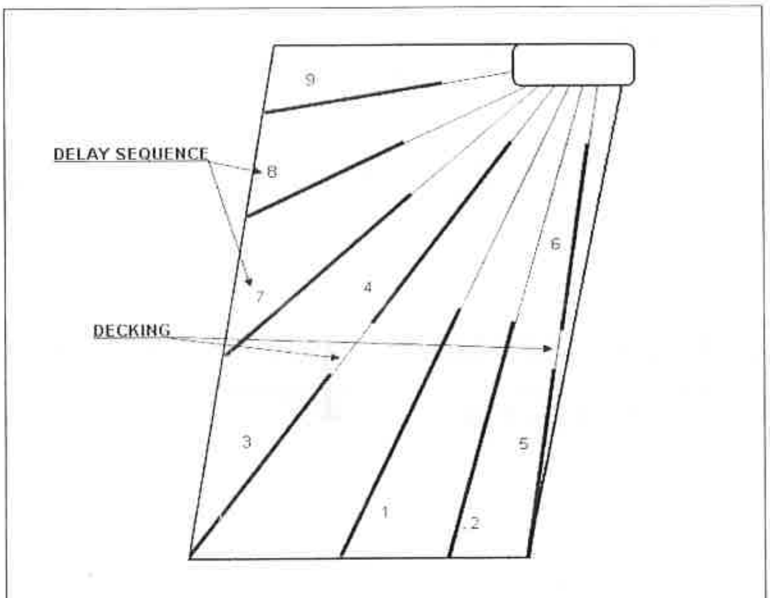
Figure – 5 (A)



SURFACE CONNECTION WITH IN-HOLE DELAY TIMININGS

Figure – 5 (B)

5.6.1. Over-break control in UG metal mines by decking - In UG metal mines stopes over-break leads to dilution of ore, requirement of more supports system – hampering safety and economy. Better blast design, orientation of face as per geologic condition, incorporating deck charging in stopes with proper delay sequence and implementing controlled blasting (as discussed above) are the way to prevent an over-break (Fig-6).



FIRING SEQUENCE OF A TYPICALLY RING WITH DECK CHARGING

Figure - 6

5.7. 'Signature-Hole' Blast Analysis for Vibration Control by using Accurate Delay Timing Electronic Detonator System

Structural response to blast-induced ground vibration is a phenomenon that has been analyzed for many years. It is becoming increasingly important, from an environmental viewpoint, to minimize vibrations induced in urban dwellings by blasting. Research developed by the USBM, universities, and others over the last more than two decades in the blasting industry, has concluded that a residential structure's level of response to blast induced ground vibration is dependent on both the peak particle velocity and the frequency of the waveform. The frequency is the number of oscillations that the ground particles vibrate per second as a blast vibration wave passes by the structure's location. Researchers have shown that, above ground structures resonate whenever they are excited by a vibration containing adequate energy matching the fundamental frequency of the structure. The value of this frequency is mainly dependent upon the mass, height and stiffness of the structure. The maximum response of a house to blast induced ground vibration occurs whenever the frequency of the ground vibration matches the natural resonant frequency of the house. Likewise, if there is little or no energy at the resonant frequency of the structure, the structural response to the vibration will be negligible.

A method of controlling blast vibrations other than by modifying the scaled distance came into use some time ago. The crucial point of the methodology is the use of a pilot-blast signal which takes account of the seismic properties of all complex geology between the blast and the target locations. Therefore, it does not require any geological model or assumption. The present study illustrates how the delay interval between blast-holes can be chosen to control and minimize the vibration energy within the structural response band of most houses. Research studies had indicated that blast vibration could be simulated by detonating a **"Signature Hole"** with the vibration monitored at critical locations, and then using a computer to **superpose the waveforms** with varying delays (Fig – 7). By choosing delay times (Δt) that create '**destructive interference'** at frequencies that are favored by the local geology, the "ringing" vibration that excites structural elements in structures, houses and annoys neighbors could be reduced. In this method, **accurate delay times are crucial to effective vibration control**, scatter in the firing times limited the method severely. **Electronic detonators have scatter less than a millisecond**. In light of all these, researchers have started finding both limitations and new potential of this new technique of controlling blast vibration.

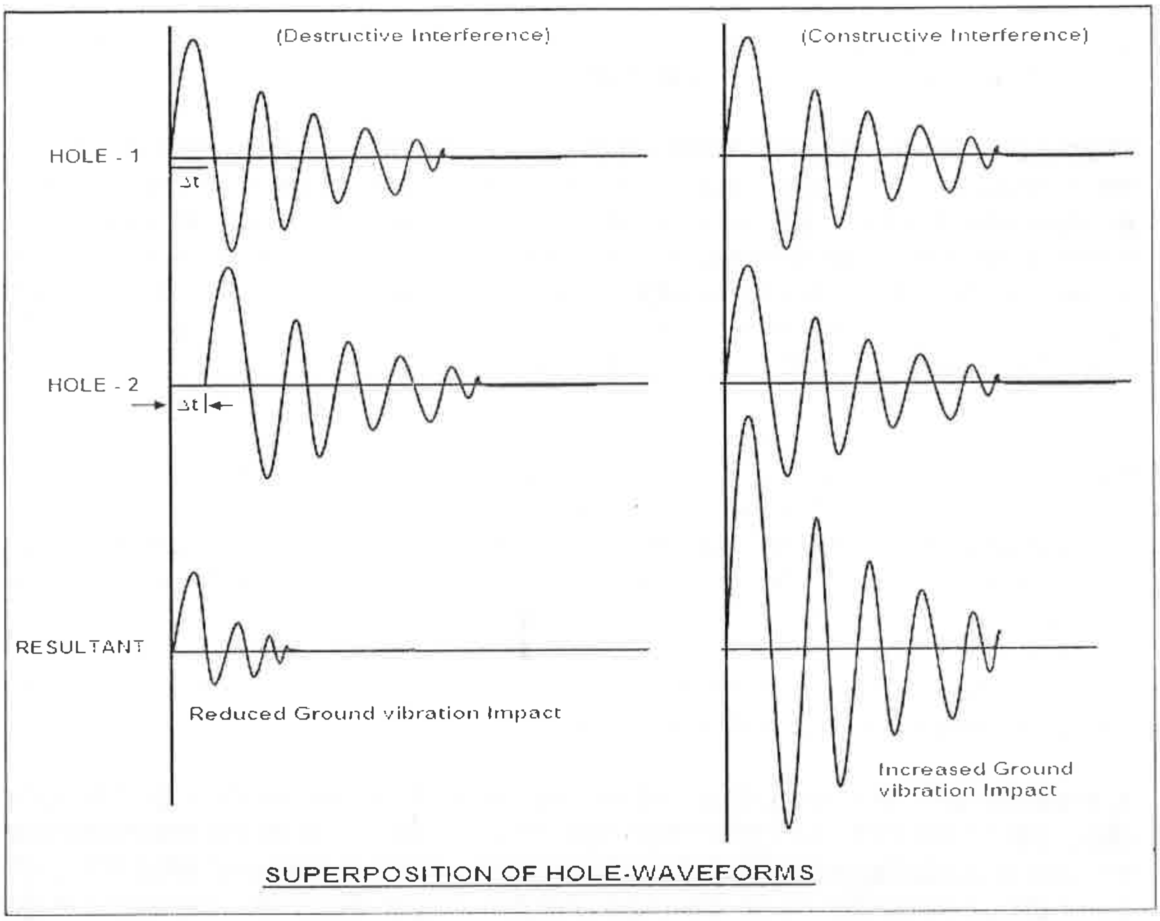


Figure - 7

5.7.1. Advantages of the technique, 'Signature Hole Blast Analysis', for Vibration Control

This technique provides optimum electronic timing while maintaining high level of production with efficiency by raising quantity of explosives per delay (kg/delay) and provide overall structural safety of blast surroundings. Moreover, blast with shorted duration results in mitigating effects of blast induced vibration. Therefore, as post-blast vibrations are reduced by raising frequencies, much larger blasts can be undertaken with better operational performance, without compromising stringent safety standards of environment. Thus, Signature Hole Analysis software tool available can be used to help optimize and improve overall operational efficiency. It has also been observed that this vibration control method is feasible for underground mining ring blasts as well.

Thus, the occurrence of programmable electronic detonators and development of computer sciences brought a new expectation in controlling the vibration level and, hence, carry out bigger blast operations both in surface and underground operations. Today electronic detonation has transformed production efficiency in mines by allowing larger blasts to handle, improving blasting cycle efficiency with enhanced safety and lesser ore dilution.

6.0. Discussion:

6.1. Blast induced ground vibration

When a structure is given an initial disturbance, it will vibrate at one or more of its natural frequencies, which are controlled by its mass and stiffness distribution. The highest frequencies of the system are always the multitude of the fundamental frequency. These characteristics in a structure are the controlling factor in response to a dynamic load such as ground vibration induced by a blast. There are two methods that can be used to calculate the dynamic properties of a structure. One of these methods is by theory (Computer Modal Analysis) and the other is by experiment (Frequency Response Function). 'Computer Modal Analysis' is done by entering into a computer the physical dimensions and the geometric and physical material properties of a structure. By adding vibration induced from traffic, blasting, construction or natural phenomena the model's response can be calculated. Whereas, 'Frequency Response Function (FRF)' is an experimental technique used to calculate the dynamic properties of a structure. This technique is widely used in different industries to solve many types of dynamic problems, such as structural failure, noise and vibration. Generally, in order to calculate the FRF of a system, the system needs to be excited with some kind of a signal. This signal is called an input signal. The input signal would be a ground vibration at the foundation of the building and the structural response to the ground vibration is the output signal. To calculate the FRF of a structure, ground vibration is generated and measured simultaneously with structural response. To do this, vibration sensors are placed on the structure and the ground. Ground vibration can be generated by detonating a small amount of explosive buried in the ground near the structure.

Blast induced ground vibration may cause damage to structures and annoyance to inhabitants in the vicinity of mines. The types of damage caused to the structures in the order of increasing intensity are (a) dust falling from old plaster cracks, (b) extension of old plaster cracks, (c) new plaster cracks formation, (d) plaster flaking, (e) plaster drops from large areas, (f) masonry crack formation, (g) partition separating from exterior walls, (h) further severe damages and building collapses.

As per legislation of some of the countries, the blasting activities must be carried out in such a manner that in a sensitive place (such as near a dwellings and public structures) ground-bourn vibration must not exceed a peak particle velocity of 5 mm/sec. in 9 out of 10 consecutive blasts and same should not exceed 10 mm/sec. in any of the blast.

6.2. Blast-induced Air Overpressures

Blast-induced air overpressures are the air pressure waves generated by explosions. The higher-frequency portion of the pressure wave is audible and is the sound that accompanies a blast; the lower-frequency portion is not audible, but excites structures and in turn causes a secondary and audible rattle within a structure. Overpressure waves are of interest for three reasons. First, the audible portion produces direct noise. Second, the inaudible portion by itself or in combination with ground motion can produce structural motions that in turn produce noise. Third, they may crack windows; however, air-blast pressure alone would have to be unusually high for such cracking. As per legislation of some of the countries, the blasting activities must be carried out in such a manner that in a noise sensitive place (such as near a dwellings) the air-blast overpressure must not exceed a peak 115 dB (linear) in 9 out of 10 consecutive blasts and same should not exceed 120 dB (linear) in any of the blast.

6.3. Over-break (back-break or end-break)

As discussed above over-break is the excessive rock-breakage beyond excavation limits. This is the breakage behind the last row of shot-holes and break that occurs at the end of shot line. Geology plays an important role on the outcome of over-break due to blasting. Poor blasting in respect of over-break occur in rock where strike of structures (faults and joints) is parallel or perpendicular to the free face. Blasting results in respect of over-break are generally good if strike of the structures is oblique to the free face. Strike parallel to the free face may result in severe back-break and end-break. Joints perpendicular to the free face results again in higher back-break. Smaller burden and spacing are recommended.

6.4. Efficient Blast design and its evaluation

Fixing of burden, spacing, delay sequence, selection of explosives, use of decks, stemming heights, sub-grade drilling etc., as per geology of strata, performance of past blasts, requirements of the mines or excavation / tunnel etc., are to be carried out. General review of previous blast designs vis a vis the results obtained are to be evaluated properly by keeping in mind the stiff burden experienced, over-break experienced, assessing past delay timings adopted, stemming length kept and type of controlled blast adopted and accordingly subsequent designs have to be fine tuned. Before establishing a final design for a particular job, a series of test blasts are to be conducted evaluating performances, analyzing major data available and find ways to plug the shortcomings.

6.5. Monitoring the blast effects

As discussed above, blasts are to be critically evaluated for their adverse impacts arising out because of any design lapses. Monitoring the blast results, level of ground vibration, air-over-pressure (noise), and over-break of high wall / host rock obtained; with the design parameters kept is the most important factors in order to improve the efficiency. The degree of controlled blast adopted and accordingly the performance of mitigation of adverse effects obtained are to be analyzed. Effective tools and modern equipments, as far as possible, are to be employed to assess these results with minimum errors. Vibrometer or seismograph for measuring ground vibration and air-over-pressure, high speed digital photography for open pit blasting for analysing throw, fragmentation image analysis technique by digital image monitoring system, seismic refraction for evaluating health (cracks generated due to blasting) of the host rock for tunneling etc., are some of the most important instruments to monitor the post blast effects.

6.6. Training

Training to blasting crew, supervisors and officials regarding various aspects of controlled blasting and ways to mitigate adverse impact, methods to analyze blasts, recent mining rules / legislation applicable, as per the requirement of legislation the implication and fixing of design parameters and, above all, application procedures of modern tools & equipments necessary for monitoring post effects of blasts. Apart from safety aspects of blasting, proper procedure of logging of blast (i.e., keeping and retaining records of design, records of performances and records of blast monitoring results) should be taught. Blasting log is an important component in order to review the performances of past blasts with the blast parameters kept and accordingly the future blast parameters can be modified suitably. Therefore, blast log evaluation programme should also be introduced in the training topic. In many of the countries, programme of blast log evaluation has become mandatory, as per their safety regulation.

7.0. Conclusions

Training to blasting crew, supervisors and officials regarding various aspects of controlled blasting and ways to mitigate adverse impact are quite important. Drilling and blasting generally is recognized as the most cost-efficient way to crush rock and minerals. However, efforts to contain costs, increase production and mitigate adverse effects of blast to meet tighter aggregate specifications, to reduce production of both fines and oversize and to appease quarry neighbors adds complexity to drilling and blasting operations. Fortunately, technology continues to move the process from an art to a science. Proper blasting techniques with computers and micro-electronics have profoundly improved the design and use of drills, drill tools, blast-initiation products, explosives and seismographs. If the progress continues at this pace, days are not far, when we run our mines or excavation activities with no or very little nuisances of blasting.

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B-99

Design of Initiation System to Improve Blasting Efficiency

Blasting efficiency may be measure in terms of)a) Charge factor, (b) ground induced vibrations, (c) Fly rock, and (d)fragmentation

The factor on which the blast engineer has a direct control includes:

(1) burden,(2) spacing, (3) sub-grade drilling, (4) stemming column length, (5) charge factor and (6) **initiation sequence/tie-ins**

The other parameters on which a blasting engineer can have a partial control include:

(i) delay timings, (ii) delay between holes and rows (iv) size and shape of blast

This lecture mainly deals with the drilling/firing patterns in which delays are obtained through the use of (i) ms delay detonators,(ii) detonating relay connectors (iii) nonel system of firing and (iv) sequential blasting machine

Presently the main problem faced by the industry is increasing dia of blast holes while DGMS is permitting only reduced charges per delay- the two are opposite to each other. It is with this in view that the use of delayed deck charge within each blast is preferred.

(1) EFFECT OF BURDEN : See Figs 1 to 3

The higher magnitude of vibrations associated with pre-split holes are indicative of the influence of greater burdens ,

Less burden lead to excessive fly rock and boulders, more noise and air blast

Thus, noise, air blast, fly rock, and vibrations are signals of an inefficient blast design

(2) SUB-GRADE DRILLING:

Minimum sub-grade drilling for minimizing vibration is ideal. However, longer subgrade drilling should only be provided in the front row where the burden is normally excessive. In worst case it could be up to 0.5B. Open bedding planes should be avoided near the floor level otherwise they will cause more noise, fly and drop in borehole pressure.

(3) COLLAR OR EFFECT OF STEMMING LENGTH:

2/3B to B (16- 25d) is the most commonly used stemming length.

Short stemming column will cause noise, air blast, flyrock, and back break. Rock containing large cracks and weakness planes, the stemming length can be as much as 2B (Fig.4). However, this will produce more vibrations,

As a guide line stemming length =12d for very hard rocks › 200kg/cm2

= 30d for soft rocks ‹ 300kg/cm2

= 2/3 B (min)

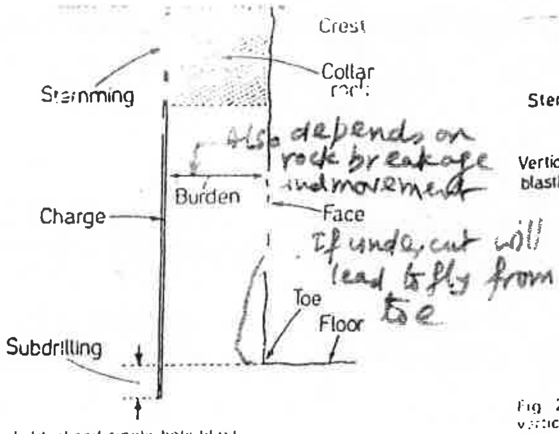


Fig 1 Idealised single-hole blast

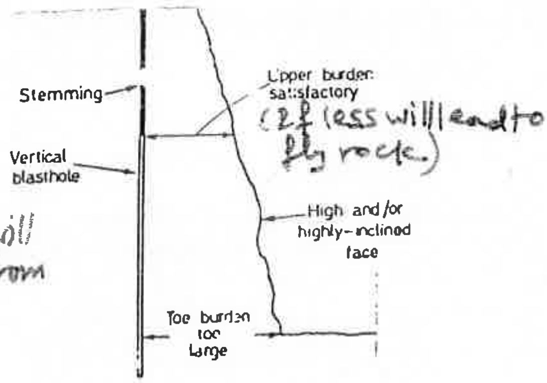


Fig 2 Excessive toe burden caused by drilling vertical blastholes in high and/or highly-inclined face

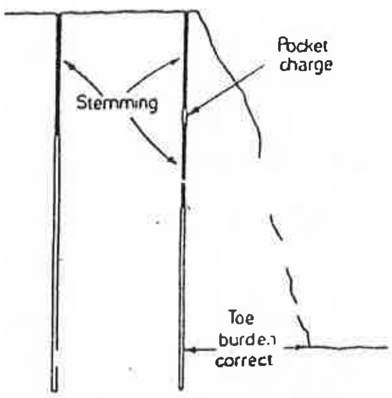


Fig 3 Use of longer stemming columns and pocket charges in front-row blastholes to reduce noise, airblast and flyrock

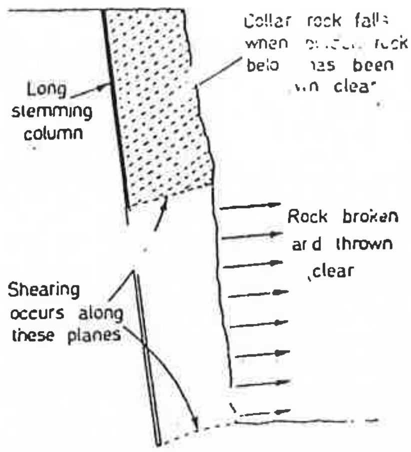


Fig 4 High shearing forces and ground vibration levels associated with use of long stemming columns in weak rocks

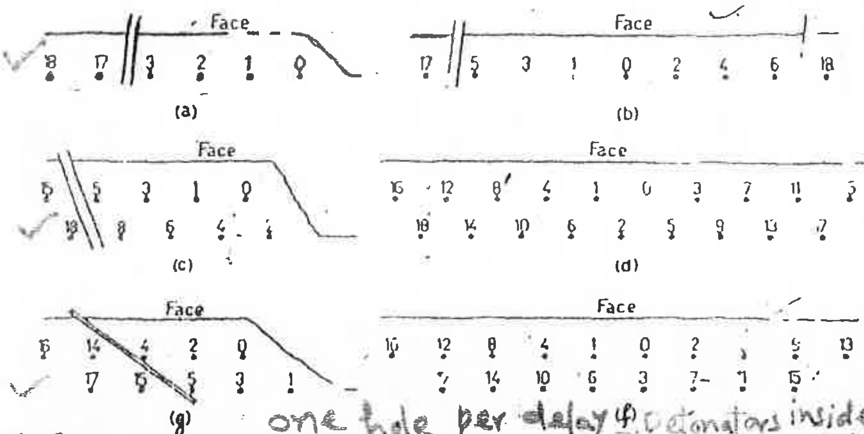


Fig 6. Firings with one blasthole per delay, initiation by millisecond delay detonators

one hole per delay (1) Detonators inside holes. Delay periods - 50-75 ms. If on surface - cut offs

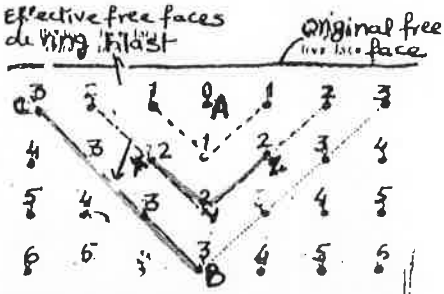


Fig 5. Formation of effective free faces during square "V" blast: XYZ and CB are effective free faces for blastholes B and D respectively

XYZ and CB are effective free faces for holes D and B

Suitable for 35-40 holes Two holes per delay)

Fig 7 Firing with two blastholes per delay - Initiation by ms delay detonators

Fig 7 Firings with two blastholes per delay. Initiation by milliseco delay detonators

(4) EFFECT OF EXPLOSIVE TYPE:

The peak blast hole pressure generated by ANFO is less as compared to NG based explosive and hence they produce less vibrations. On weight to weight basis an NG based explosive may produce twice the level of vibrations as compared to ANFO.

The CHARGE requirement depends on the way the blast is designed and should be optimized by trials. The value is dependent on the magnitude of displacement and fragmentation desired. If possible catering tests should be performed. The height of crater $Hc = E\ W^{1/3}$,where W=weight of explosive charge and E= strain energy.

The charge factor Q (KG/$\mathbf{m}^3$)is strongly related to burden and strength of rock. The relationship developed by Gupta,R.N. (1990) is

$$Q=0.278\ B^{-0.407}\ f^{0.62}$$

Where, B=burden (m) , and f=Protodykonov Strength Index $\approx \sigma_c/100$, σ_c=compressive strength of rock(kg/cm2)

(5) MULTI-HOLE BLASTING:

It is the most common method employed for blast design. However, most of the complaints are only when multi-hole blasting is done. These complaints are mainly related to excessive fly rock, vibrations, and noise. However the complaints can be greatly reduced by observing the following recommendations including:

(i) Shoot large blast holes instead of small and greater number. Once in a week is a good practice. Large blasts receive greater attention of planning, design and execution,

(ii) Fire the blast on a busy day and during busy hours when high back ground noise is present or create back ground noise just before the firing by operating pit heavy earth equipments,

(iii) Avoid trunk line cords or cover them with fines. Also the open ends of down lines should be covered,

(iv) Adequate stemming length in all holes particularly the first row,

(v) Fire in good weather conditions to avoid inversions and when winds are away from the residences,

(vi) Keep detailed records of time, data, location of blast, no. of blast holes, max. charge per delay, allocations, weather conditions, boulders/secondary blasting. These are useful for refining the design and proper analysis.

(6) SHORT DELAY BLASTING:

To control ground vibrations observe the following practices (see Fig.5):

(i) use ms delay between rows or DRC (detonating relay connectors),

(ii) Move the rock towards free face by providing delay periods of 4-7ms/m of burden,

(iii) Fire each hole using a delay detonator instead of trunk line cord because for the same delay number the delay detonators have scattered. This scatter can be used to reduce vibrations,

(iv) Trunk line detonation is instantaneous. Minimize the number of blast holes on zero delay detonator especially in V-type firing,

(v)Use a firing order which gives a long effective face V1, V2, OR V3etc.

(vi) Provide max. forward relief for blast holes in the second and later rows,

(vii) Give effective burden that are not too large,

(viii) Avoid tight blast hole pattern V1 in Fig.5 along AB which prevents normal rate of displacements.

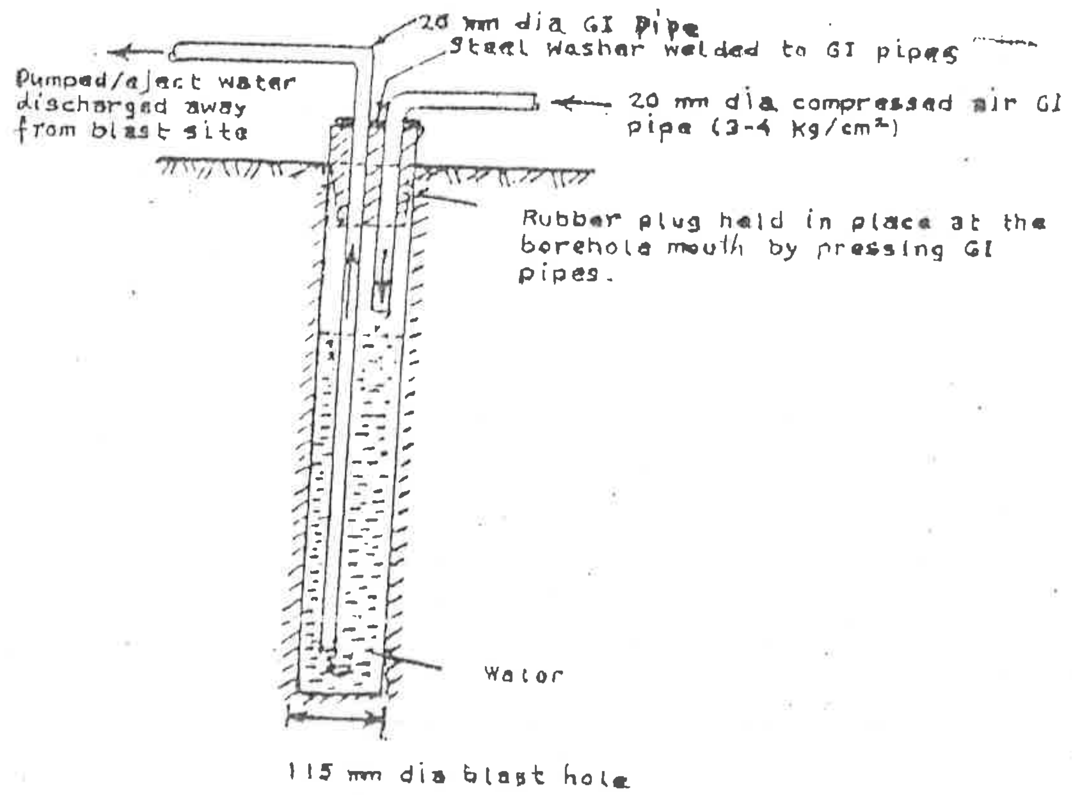


Fig.9. Typical Arrangement for Dewatering Blast Holes

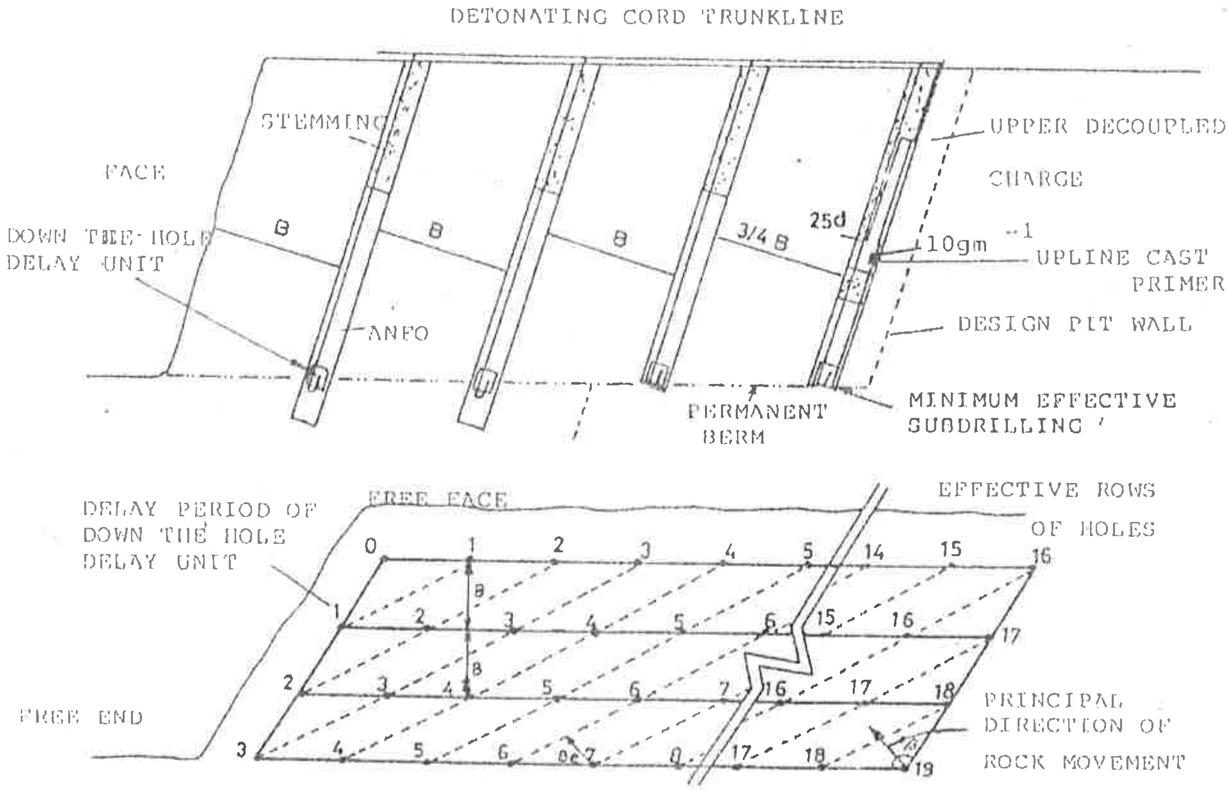


Fig.7 Hypothetical blast showing desirable design features for minimising overbreak, ground vibrations and pit wall instability

(7)SHORT DELAY DETONATORS:

They have the advantage that they can be placed either inside or outside the holes. As they exhibit scatter for the same delay number they are advantageous in reducing vibrations particularly near the blast holes. They also eliminate the problem of cut-offs thus, permitting longer period for a blast thereby reducing the ground vibration. They also give complete choice of point of initiation in a hole. They are being manufactured all over the world in 0-19, 0-24, and 0-30 series. However, in India they are available in 0-10 series (electrical). Zero number should be used only in one or two holes as they do not exhibit scatter.

Fig.6 shows the tie-ins for small number of holes so that each hole can be separated from the other by providing delay between each holes- single row/two row

Fig.7 Pattern suitable for 35-40 holes with 2 rows. Two holes for each delay number,

- If the delay periods between holes is 50-75ms the detonators should be placed inside the holes and not on surface to eliminate the possibilities of cut-offs,
- For reduction of vibrations in case of large dia holes- the use of 2 charged decks in each hole is preferable with different delay periods in each deck

Fig. 8 For large dia holes- to reduce vibration each hole has two decks and each deck has a delay detonator of different number. The height of stemming column in each deck should be sufficient to prevent sympathetic detonation. The height of stemming column depends on hole dia., explosive type, stemming material type and wetness of strata,

- The number of holes which can be fired in a short depends on the delay series and also capacity of the exploder. However, multi-channel exploders can fire large number of holes greater than 200. Sequential Blasting Machine is an example of MCE. Delay is obtained electronically between the two channels and this delay period can be selected.

Fig.9: Simple firing pattern with sequential Blasting Machine or Multi Channel Exploders. Delay timings between the two channels can be selected from 1-999ms.

Fig.10: To increase the size of blast a series of delay detonators are connected with each channel to give a multi-row firing with MCE.

Fig.11: Two deck multi row firing with MCE

(8)DETONATING RELAY CONNECTORS: With detonating cords

- DRC's have increased the possibilities of firing very big blasts at least on theory by even providing delays between each hole. However, the possibilities of cut-offs discourages the operators to go for a big blast. In addition unless covered by some fines the trunk lines will give impulsive noise and air blast.

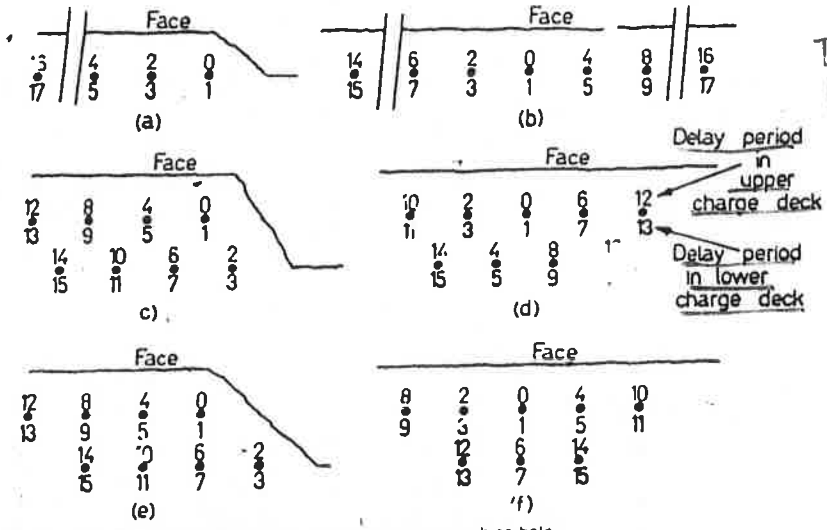
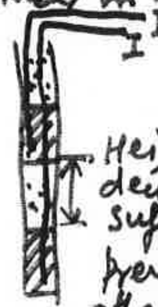


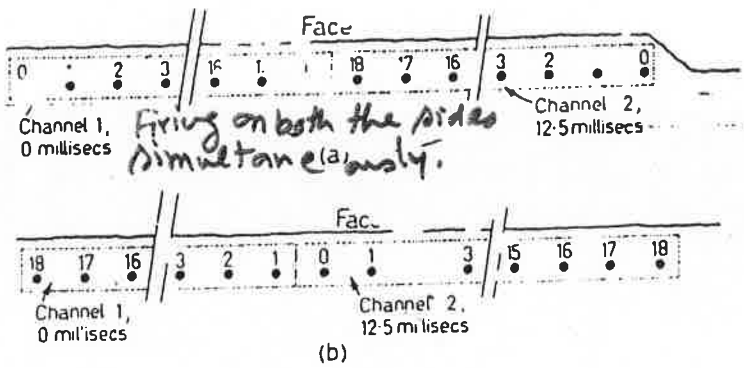
Fig 8 Firings with two individually-delayed deck charges per blasthole

Two decks in each hole with delay detonator of different number in each deck



Stemm

Height of each deck should be sufficient to prevent sympathetic detonati



Firing on both the sides simultaneously.

Firing with sequential blasting machine MCE

Fig 9 Single-row shots fired with a multi-channel exploder: one blasthole per delay

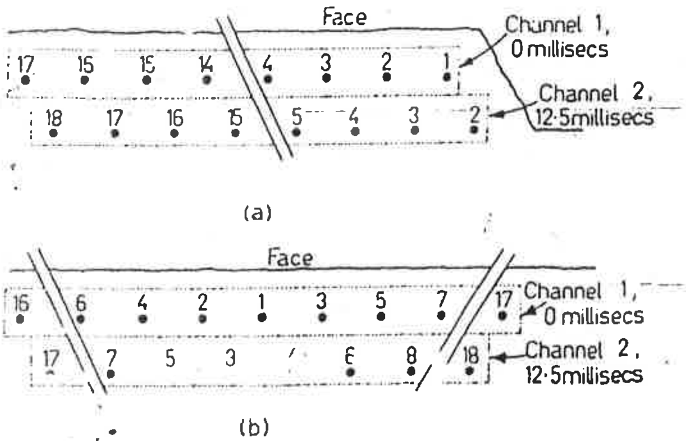
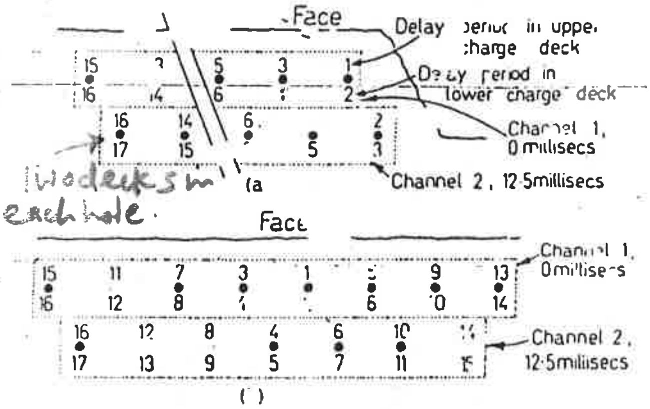


Fig 10 Double-row shots fired with a multi-channel exploder; one blasthole per delay



Two decks in each hole.

Fig 11 Blasts with individually-delayed deck charges fired by a multi-channel exploder

Fig.12 a : Firing pattern with detonating relays for small to medium blast with single row- this pattern gives low vibration'

- Firing sequence should be away from the structure,

Fig.12d: Firing in the centre with two different delay DRC's

Fig.13 e: Staggered pattern with DRC's. If the taper is not given and holes L, M,N,O,P,Q,R are fired they will produce more vibrations,

Fig.13b: V-Pattern Square: The central hole will have more restrictions at the throat along AB because of large effective burden. This pattern also gives high vibrations,

Fig.13 f: V-pattern with staggered pattern – worst still tighter than square V-pattern

Fig.13. c,d,g,h: V1 **AND V2** pattern are good as they provide larger free faces and, therefore, produce less vibrations

- **Where ground vibrations are a major problem, initiation should start towards one end of, rather than near the centre of the front row to increase total blast duration and reduction in vibration**

Fig.14: Un symmetrical pattern of firing with one end initiation, increase the total blast duration and reduces vibration,

- **IT SHOULD BE PREFFERED TO INCREASE THE NUMBER OF ROWS INSTEAD INCREASE NUMBER OF HOLESIN A ROW**

Fig.15: DRC's –blast fired towards free end.

From the above patterns following conclusions are drawn:

- **If ground vibrations are to be minimized, blasts should be fired to a free end whenever possible,**
- **Where it is necessary to shoot to an open face, the initiation should start towards one end rather than at or near the centre of the blast hole pattern.**

(8) DELAY TIMING:

FIG. 13g: ABC should detach and must move forward before DEF is fired/detonate. If the delay time is short then ABC burden will be added to DEF which will lead to a confined hole situation giving rise to high noise, vibration and fly rock. It should be 4-8m/s burden.

(9) SECONDARY BLASTING:

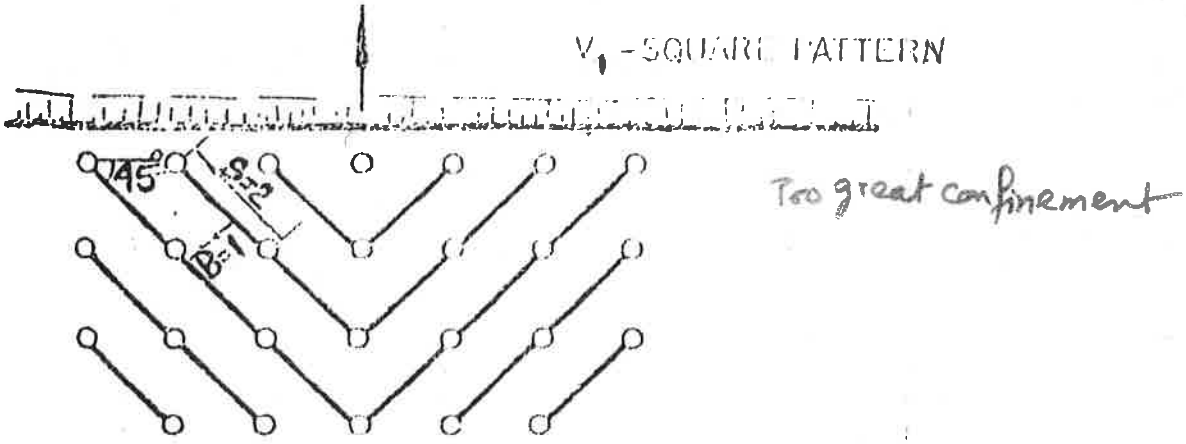
For reduced noise and vibration 'Popping' rather than plaster shooting or mud capping should be used. A large number of popping holes can be fired by using ms or preferably ½ second delay detonators.

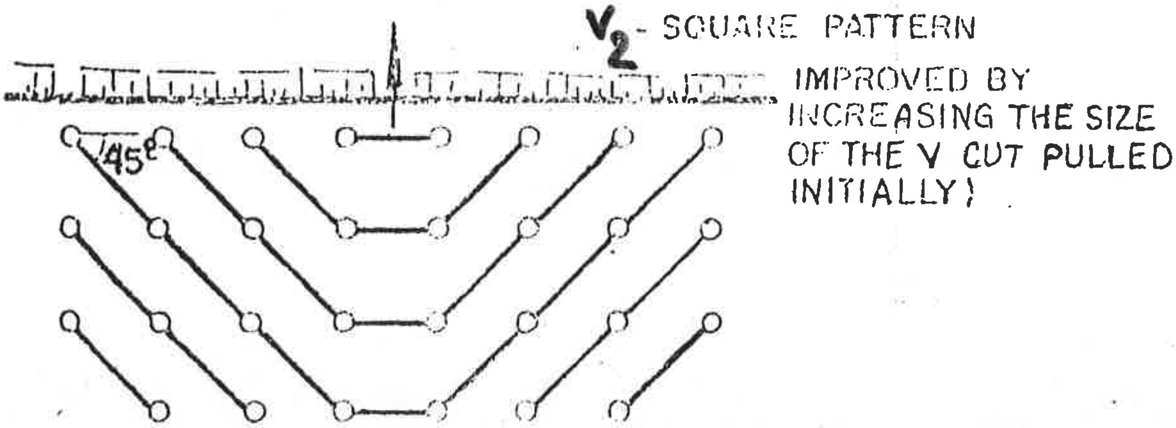
SUMMARY

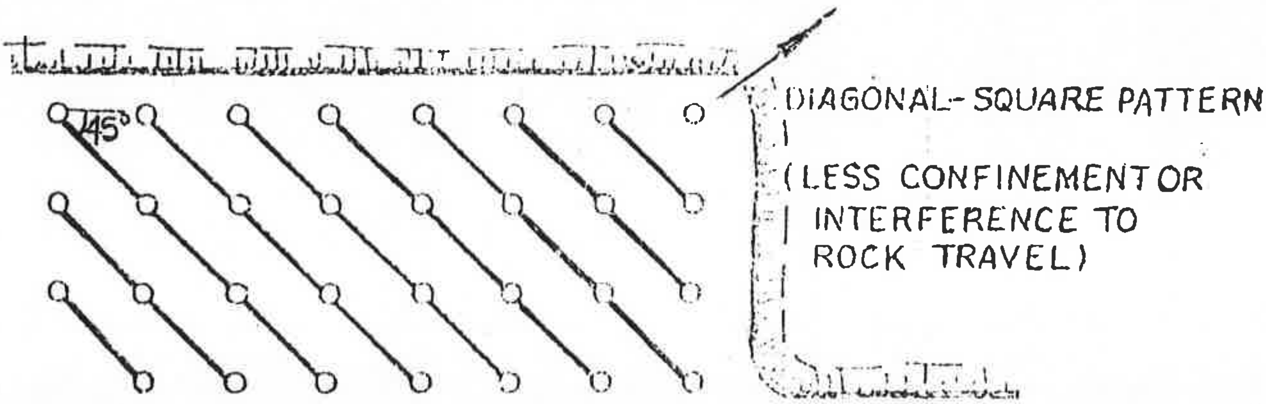
Observe the following for a good blast:

(1) Shoot large well designed blasts

(2)Locate all detonating cord, detonators etc. inside the hole, cover trunk lines of DF by at least 250mm of fines,







V_1 - SQUARE PATTERN WITH IMPROVEMENTS

FIG. Diagrams showing ways of tying-in square patterns

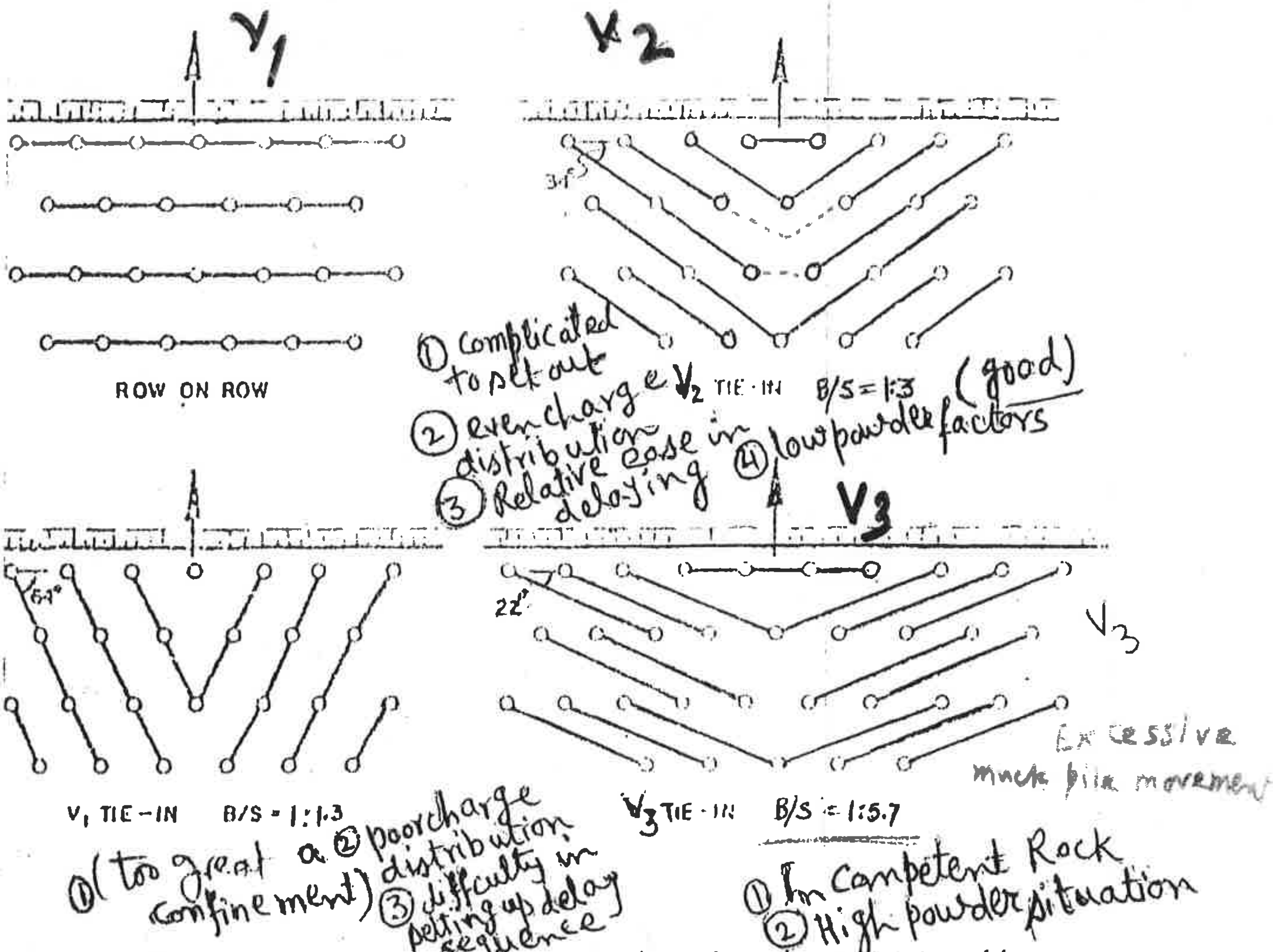
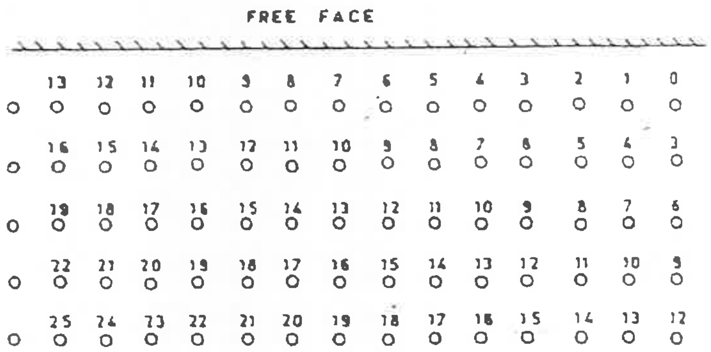
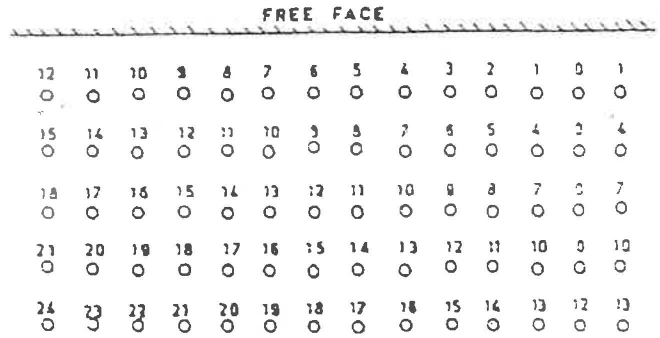


FIG. Diagrams showing ways of tying-in staggered patterns

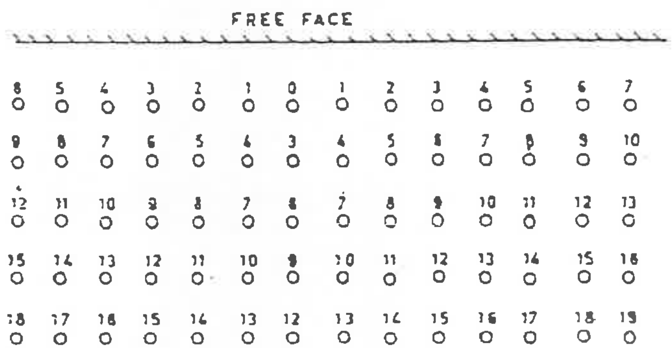
a. SQUARE V_2 FIRED TO CORNER WITH USE OF ORCs



b. UNSYMMETRICAL SQUARE V_2 AT ONE END WITH USE OF ORCs



c. SQUARE V_2 PATTERN TO OPEN FACE WITH USE OF ORCs



d. STAGGERED DIGONAL INITIATION AT CORNER WITH USE OF M S DELAY DETONATORS

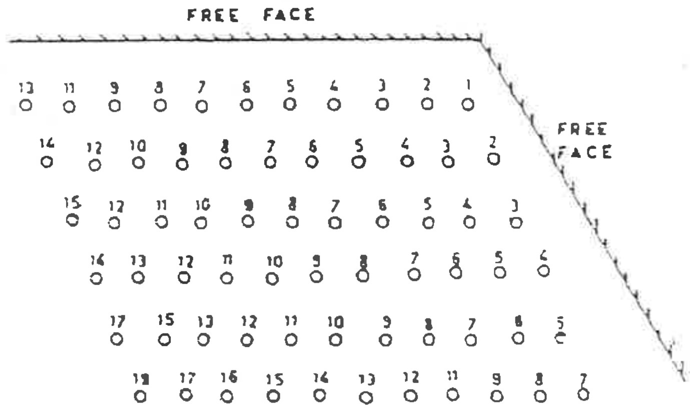


Fig. SOME OF BLASTING PATTERNS FOR LARGE BLASTS TO MINIMIZE VIBRATION (ISM COURSE 1989 AND ATLAS POWDER CO 1987)

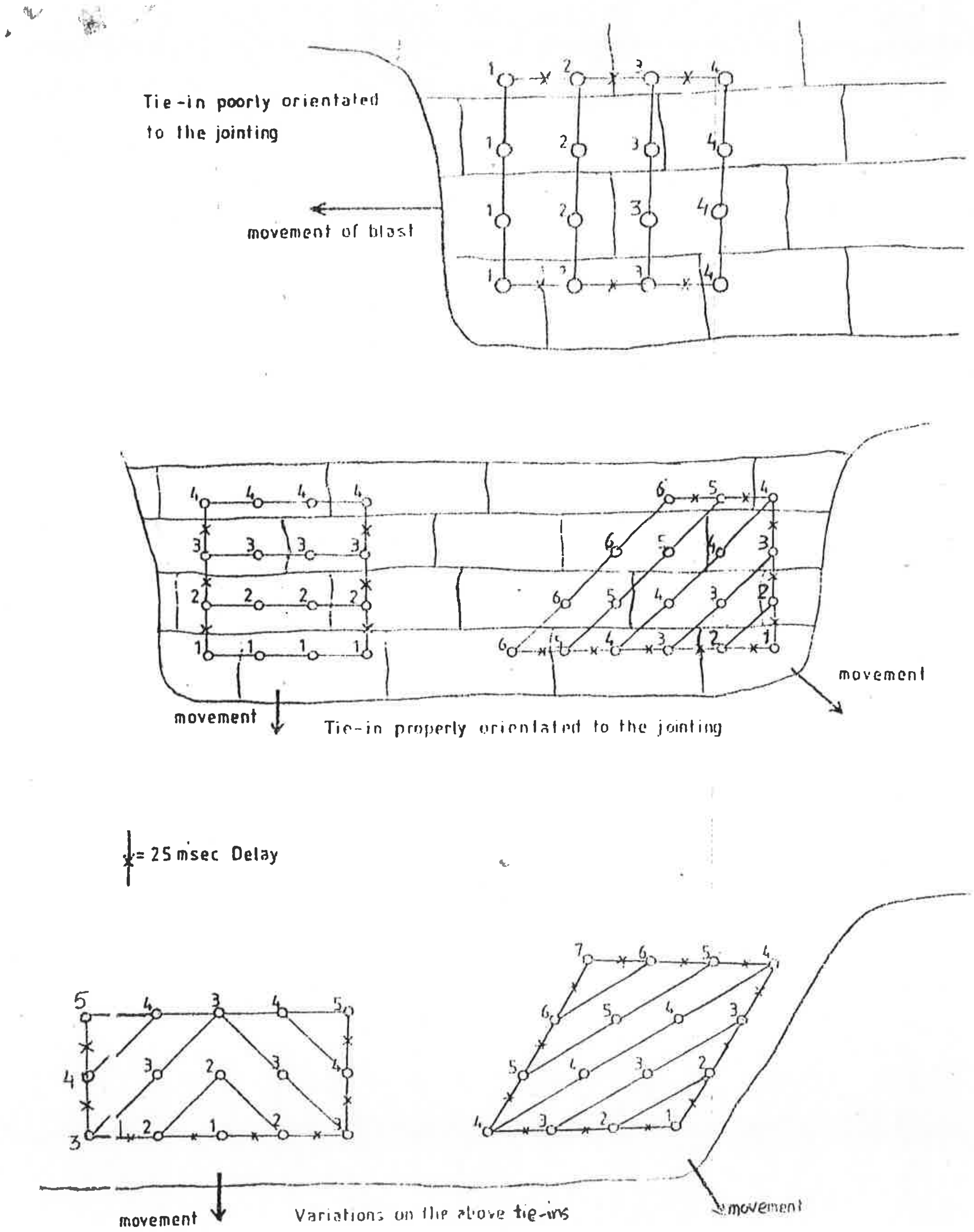


FIG. Diagrams showing how tie-in and delays can influence the movement of blasted material.

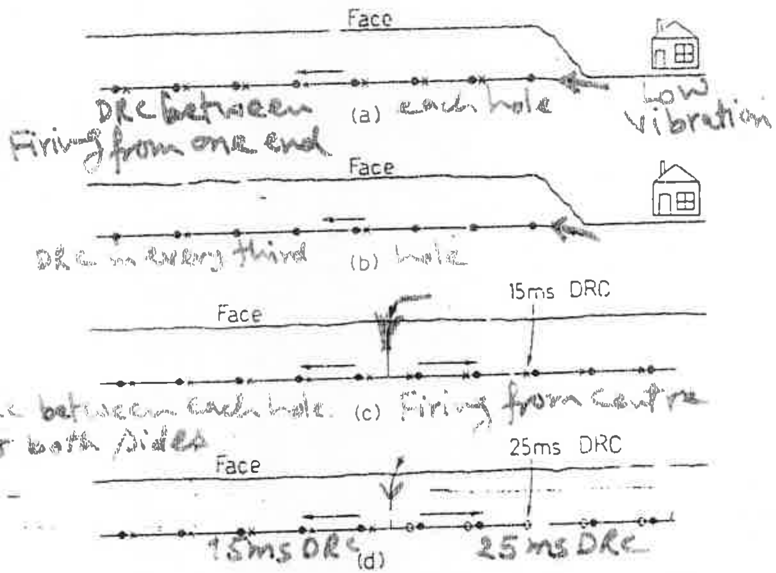


Fig 12 Single-row blasts in which delays are provided by detonating relay connectors (DRCs)

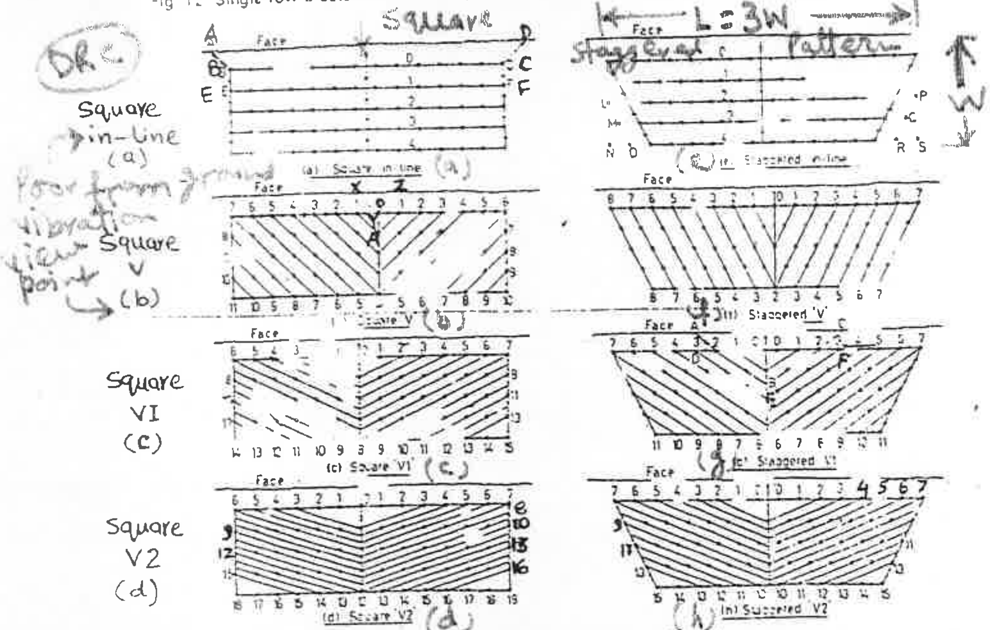


Fig 13 Blasts fired to an open face with symmetrical (or nearly so) initiation patterns based on use of DRCs
(70-100 holes) - Initiation from Centre

Initiation from one end - preferred
Greater total blast duration, less vibration

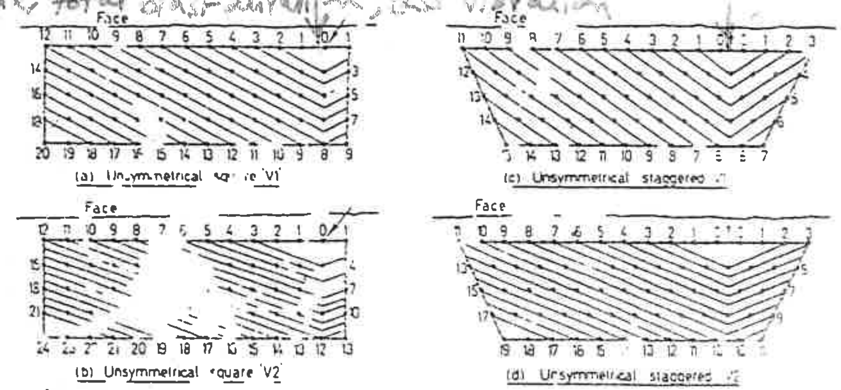


Fig 14 Blasts fired to an open face with unsymmetrical initiation patterns based on use of DRCs

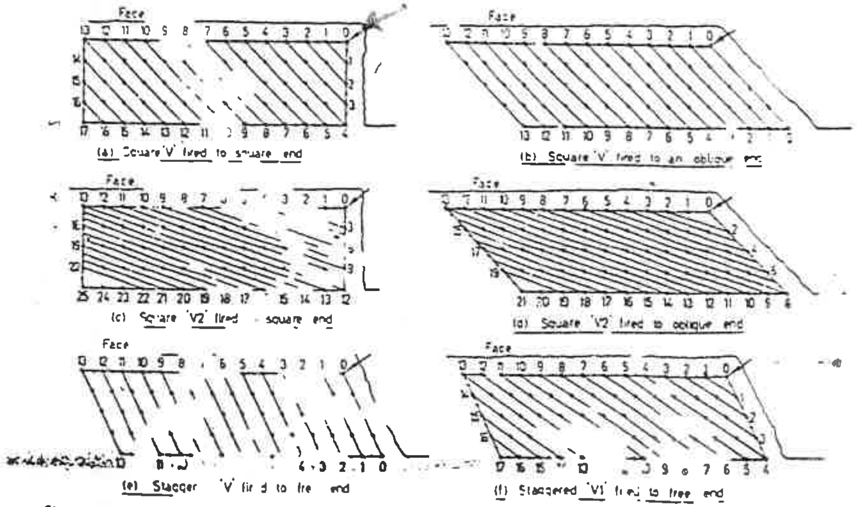


Fig 15 Blasts fired to free end. Initiation pattern achieved through use of DRCs
Blast fired toward a free end

(3) Shoot in good weather condition at mid day,
(4) Create high background noise before shooting,
(5) Minimize the weight of explosive per delay and use maximum number of delays. Zero delay should be either avoided or put minimum weight of explosive on zero delay,
(6) Maximize the delay periods between holes /rows and have large blast duration,
(7) Keep detailed records of blast data
(8) To reduce noise and air blast follows the following:

- Maximize the burden and stemming column length particularly in front row,
- Secondry blasting- by popping with ½ seconds delay detonator

(9) To reduce vibration follow:

- Maximize use of low strength explosive (ANFO)
- Avoid tight blast hole (prefer V1 OR V2 pattern),
- Initiate holes using delay detonators in place of DF AND DRC's to take advantage of delay scatter,
- Use multi channel exploder/ Sequential Blasting Machine,
- For large dia holes use individually delayed decks
- Minimize sub-grade drilling,
- Fire the maximum possible number of holes to a free face end,
- Start initiation from one end instead from centre of the front row

(10) FLY ROCK: To reduce fly rock follow:

- Maintain the stemming length 20-25 times hole dia.,
- Maintain constant burden throughout the hole length at least in the front row of holes,
- Use pre-splitting in the last row of holes
- Select an inter row interval that allows each row to push its burden forward rather than in an upward direction
- Muffling of holes/area to be blasted including during popping
- Buffer blasting to prevent flying in horizontal direction

NONEL SYSTEM OF INITIATION: See Figs 16 and 17:
The PVC shock tube contains 0.1 gm of PETN, 3mm outside dia. And 2mm inside dia tubes, VOD=2000M/S. The end of the tube is connected to a non-electrical delay detonator which is detonated by shock wave.
Advantage:
(i) Safe from electrical stray currents and radio frequencies,
(ii) Insensitive to concussions and heat,
(iii) Do not detonate any commercial explosive including NG based

HD NONEL PRIMADET SYSTEM:
It is a noiseless trunk line, permits utilization of delay interval on a trunk line by providing a MS delay connector on trunk line branches in addition to the delay in primer itself.

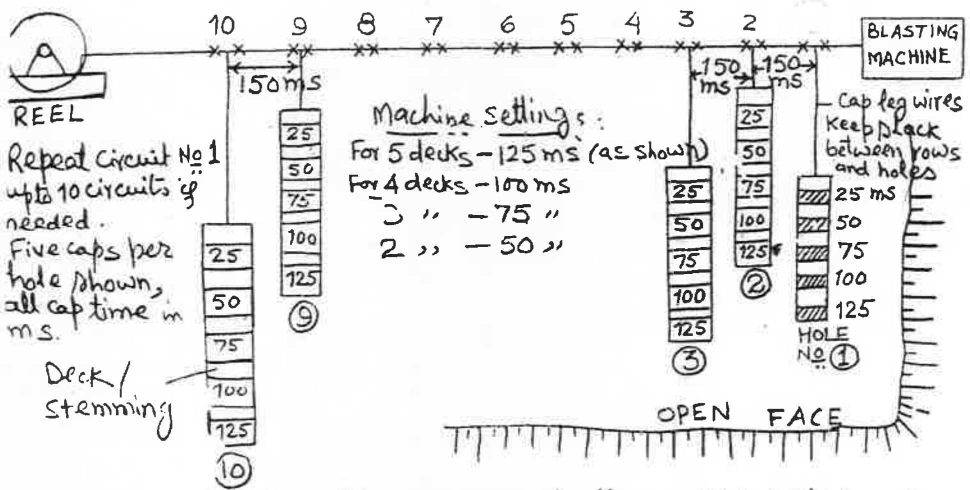


Fig 18: Multi-deck firing sequentially. Multideck firing of holes is simplified with the aid of sequential timing. With 5-deck plan, the holes are triggered 125 ms apart. Each hole also has the same series of delay caps 25-50-75-100-125, and the caps go off from top to bottom. As the last cap in hole No 1 goes off (at 125 ms) the caps in hole No 2 are triggered so that the top cap in hole No 2 goes off at 150 ms.

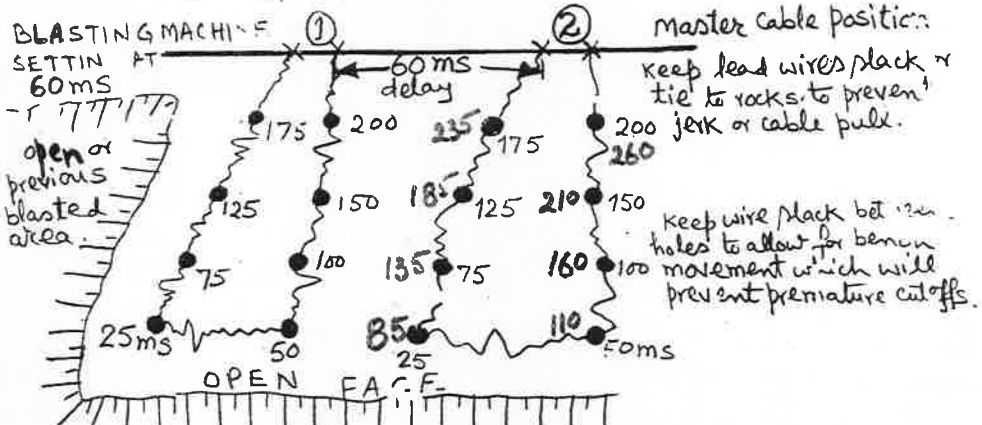
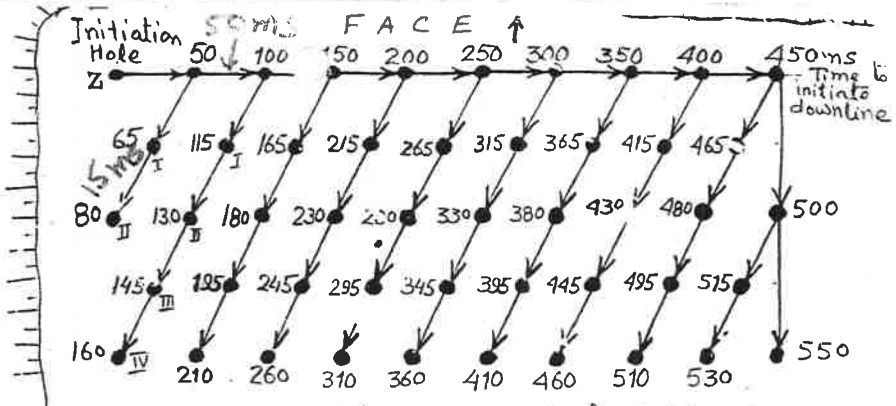
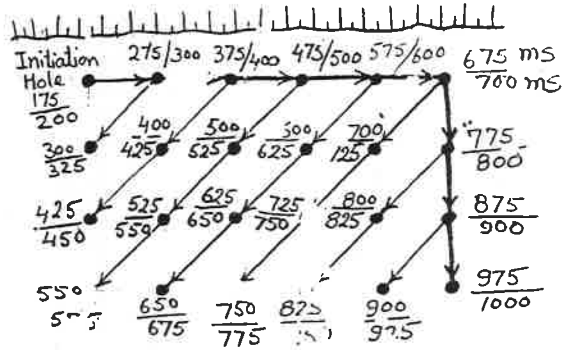


Fig 19 Use of Sequential Blasting Machine for Multi row blasting in a large blast.



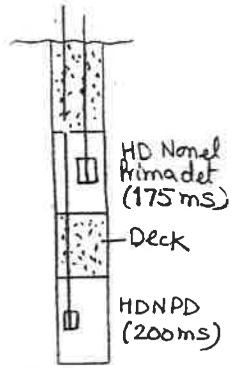
50ms trunk line delays across the front (triggering of IV, each 50 ms)
15 ms trunk line delays down each hole (I, II, III, IV)
Triggering simultaneously along the diagonal line.

Fig 16 Sequential Initiation using Nonel Detonators — hole by hole – five rows, staggered V fired to free face.



100ms Nonel Trunk line delays across the front, and 25 ms between diagonal rows.

HD Nonel Primadet (175 or 200 ms) in the hole.



Deck delays to reduce Vibrations.

Fig 17 Sequential Initiation, hole by hole with deck delays

Note: Same delay in each hole with decks. If decked explosive charges are needed for vibration control, then the sequence of intervals shown here may be used. An unlimited sequence of intervals using upto 8 delays in each hole and multiple rows may be designed whereby no two decked charges detonate simultaneously.

SEQUENTIAL BLASTING MACHINE: SEE Figs. 18,19 and 20

Manufacture by M/S RESEARCH ENERGY of OHIO, USA.

It is a condenser discharge machine with sequential timer. It has 10 circuits and 1-999ms delay.

Care should be taken to blast design and tie-ins. An error may lead to cut-offs. The current does not pass until the previous detonator is not blasted, thus, chances of cut offs with larger delay interval. Slack the electric wire to prevent cut-offs.

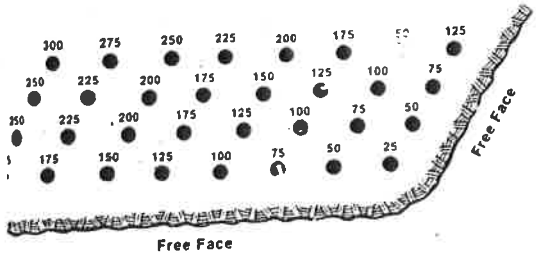
Blasting Advances- i-kon system of initiation (a computerised digital detonator)

Developments in blasting technology are essential to raising productivity and lowering costs. Good blasting can make a huge difference to the mining process. For instance, improved fragmentation can increase haulage rates and **crusher productivity, by as much as 10%. Improved fragmentation can also cut loading time by as much as 25%.** Efficient blasting can lead to **15-20% reduction in vibration propagation** in open pits. Blasting can be vital in controlling **ore dilution-controlling** the angle and direction in which rock is thrown and improving casting efficiency. The possibility to expand the blasting pattern can lead to **5-10% savings in drilling and blasting costs**. Suppliers continue to work on developments that will bring ever greater advances in productivity.

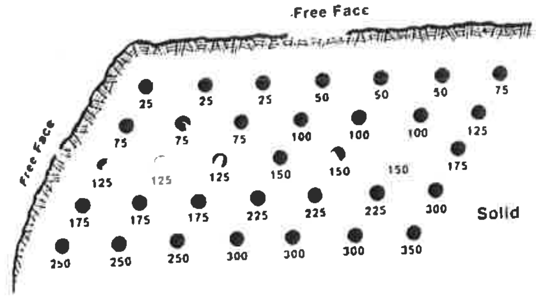
Considering underground mining specifically, electronic detonators represent a significant breakthrough in the rock breaking function and have delivered numerous benefits associated with precise timing, including **better fragmentation, reduction in the number of phases and increased advance per blast.**Although with the introduction of these electronic detonator systems comes deployment of high technology networks, software and infrastructure systems all designed to give mines centralised blasting control.

Three key impacts from the development of networked blasting systems are safety, centralised control and information benefits. From a safety perspective, **digital control networks** deliver a more reliable centralised blasting capability than any other system available today. Blast control networks also enable centralised control, because they deliver to surface a wealth of pre and post blast data from the underground workings. **These systems inform managers of potential safety and production problems, the scale, location and extent of the planned blast before the panels are centrally initiated delivering increased management and operating control. Lastly, these systems create information transparency in that they deliver to the surface inslope information and communications such as when detonators are connected to the system, ventilation conditions, temperature levels and moisture levels. Digital blast management networks are therefore a major step towards continuous mining operations, which require real time information and control systems to be viable.**

The implementation risks and realities of any new system are complex. DetNet solutions have identified both the benefits and the obstacles that have to be managed for the effective adoption of these control systems. This experience base, both positive and negative can help the path forward to a more integrated mining system that will continue to deliver more benefits, critical for the effective survival of the underground mine of the future.



A MS Pattern as an Echelon Shooting to a Corner



C MS Pattern Shooting to a Corner

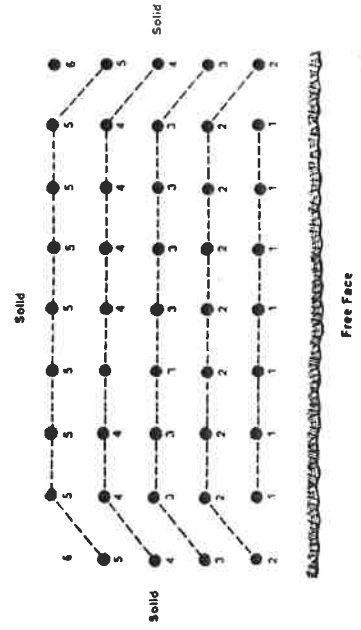


Figure 9.9 Overburden Casting Delay Pattern. Numbers Next to Holes Denote Firing Order.

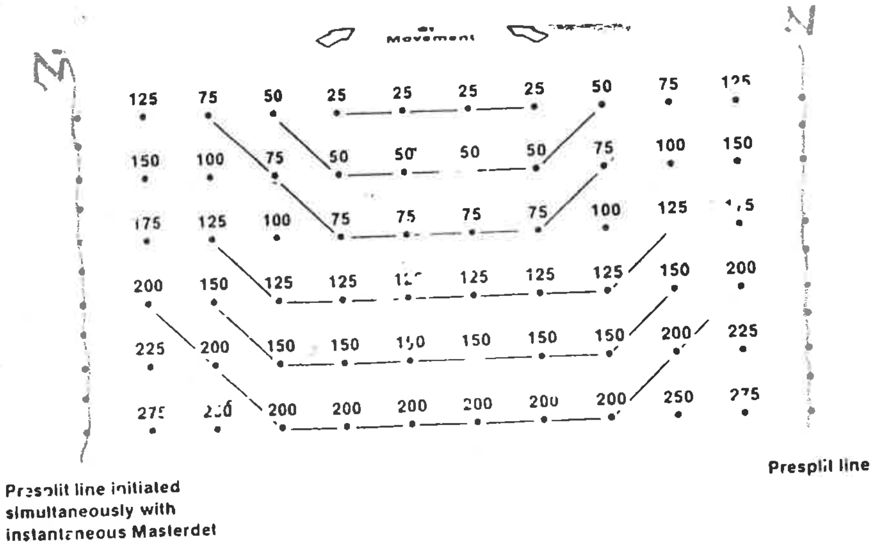
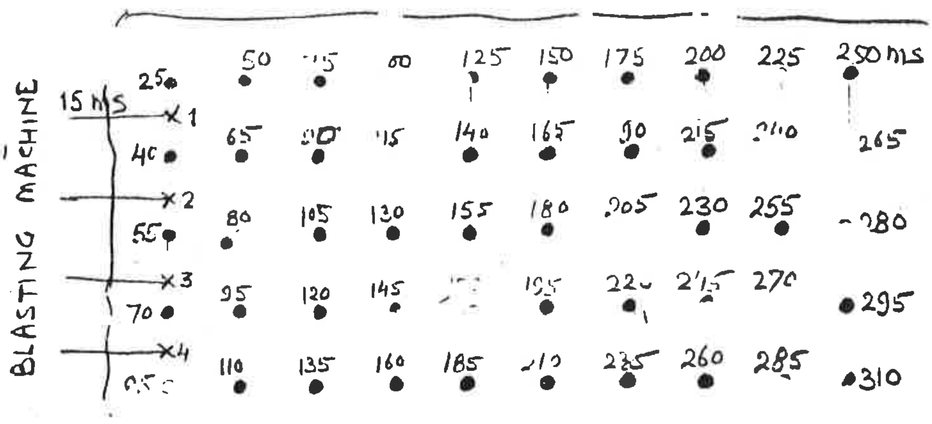


Figure 9.13 Flat V ms delay pattern configuration using Masterdet in an example of a through cut which extends to excavation limits on both sides.

-14-



25 etc - delay timing
Blasting machine set at 15 ms

Looking to the future,,Blast Control Networks(BCN) are the first step in the creation of a network revolution as they deliver computing power into mine workings. " **Once the computing power is deployed in the workings, the systems will evolve into a real time information platform, delivering key operating data to miners at the face and managers in different parts of the operation.** This integrated **Mine Area Network**,MAN, will deliver one unified blast and rock breaking critical data and control platform to the modern mine.

The MAN will deliver unprecedented safety features to the modern mine. Key areas of research are : Slope Condition monitoring – temperature, air quality and humidity measurement systems are being prototyped. These systems will integrate into a secure MAN that cannot be compromised, ensuring this occupational health and life critical data is always available.

People Tracking – computer power in the face allows mines to know where people are thus protecting the workforce from potential hazards.

MAN's will communicate data and information to both miners and mine managers that previously were unavailable in real time, if at all. A number of research projects are already underway to deliver services via a MAN. These are aimed at accurate production information, inventory ordering systems and slope condition reporting (all in real time). MAN's will also integrate mine planning and ore flow analysis.

The more computational power in the hands of the people in the workings, and the greater their system adoption, the more information can be provided to these workers and throughout the mine. The safety and operating benefits of communications are immense, and systems are being prototyped to deliver smarter communications in slope and throughout the mine. Simply put, Mine Area Networks will deliver to the right people the right information in real time. **"i-Kon has meant a 68% reduction in vibration and airblast readings.** That makes for happy neighbours. Our customers are just seeing that i-Kon is everthing that is cracked up to be. **Total accuracy, huge flexibility and the elimination of human error thanks to the testing we can do.** "Following the success of its Deltadet electronic initiation systems, noted for its centralised programming features, Delta Caps has now released Deltadet II. This fully centralised system allows the programming of 1000 detonators within 20 minutes via an intuitive serial connection. Field implementation only requires fast, straightforward operations, which significantly reduce the risk of human error. Deltadet II's plastic shell and high intensity cap also contribute to its high safety standard.

Allowing for remote programming and remote initiation, the sequence can be sent by an encrypted radio frequency protocol and the blast can be initiated without an expensive and cumbersome lead-in cable. The distance from the firing point to the blast can be several kilometres if required. The Deltadet is programmable from 0 to 10 s in increments of 1ms, providing users with 10,000 possibilities. Dyno Nobel's latest enhancement to the Nonel system is the Nonel Snapline, with new built-in safety and performance features to minimise the risk of misfires and improve handling. This translates to faster charge-ups and decreased cycle times so productivity is increased. Nonel Snapline is a low strength surface detonator series used in conjunction with the Nonel MS series.

The Snapline detonator prevents accidental initiation and reduces scatter from nominal firing time, allowing the effective use of delay intervals, and provides optimum reliability and versatility. Because it has a small explosive base charge, it significantly reduces shrapnel. The detonator is housed in a unique colour-coded plastic connector, making identification and connection easier.

It is recommended for use as a surface delay for underground and surface production blasting, connected in series or parallel to adjoining holes, providing a time delay sequence. The detonator cannot be used in conjunction with detonating cord down-lines. It provides excellent water resistance and can safely be used in temperature ranging from -40^{0}C and +70^{0}C.

Power Plugs

One way to improve fragmentation is Power Deck's bottom hole power deck technique that uses a Power Plug loaded with cuttings suspended in the drill hole to provide mass, and depending on the rock type to be blasted, an air column below the plug of pre determined length provides the free face for the blast energy. When fired, the result is a piston effect that greatly increases the pressure in the bottom of the hole. This increased pressure wave reflects off the hole bottom and back into the overburden where it breaks the rock apart with tension rather than compression. Where the hole bottom meets the hole wall, the pressure wave creates a pre-split effect that propagates from hole bottom to hole bottom. This results in a flat even bench and virtually eliminates the need for sub drill, according to Power deck.

In a conventional blast, gas pressure is immediately relieved through the top of the shot. This greatly increases the potential for air blast and fly rock. As described above, a power decked shot directs the force downwards and breaks the rock with tension rather than compression. As tension cracks form in the rock, a reservoir is created for the gas. This newly created void allows most of the gas pressure to be relieved in the bottom of the shot as the rock mass is lifted, and then all the excess pressure is relieved throughout the rock mass as it is set back down.

When the effective bottom hole deck is combined with top and/or middle decks, dramatic increases in control and reductions in vibration are possible. Typically, stemming length can be reduced without increasing the danger of blowouts, thus allowing much better fragmentation in the collar area. By adding multiple decks, vibrations can be cut down to as much as one-fifth of their original magnitude.

Multiple hole and multiple power decks can offer significant benefits to any mining operation by lowering the power factor, eliminating the need for subdrill, and greatly reducing flyrock and vibration, while at the same time, maintaining or increasing fragmentation and improving bench floors.

Loading Explosives

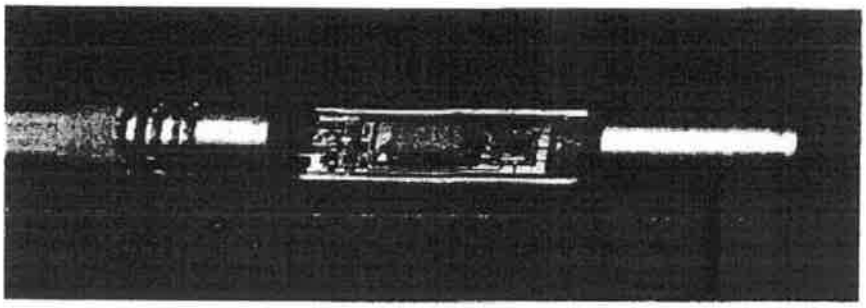
Every underground mining operation has special needs based upon the required coverage and production capacity. Breaker technology Inc provides custom fit solutions to Anfo loading operations. Producers can choose their desired carriers, conveying and lift systems, tank capacities, and number of tanks-either one or two.

Recently, a custom designed Anfo loader was supplied to a 2Mt/y limestone mine in the US Midwest. This was the first BTI double pod arrangement with two large Anfo storage tanks, each with a capacity of almost 1,000 Kg. BTIO believes at the time it was the only manufacturer in the industry to offer that much capacity. It allows mine to maximise production time by minimising the amount of time required to stop and refill tanks. This is a two-man operation with dual position controls operated remotely from the basket and from the ground. Also, this is a room and pillar mine with a room having upto three faces.The BTI unit was specifically designed to reach three faces without moving the vehicle. The boom has 270^{0} swing, so that the entire face can be covered without repositioning. The goal was to charge upto 15 faces per day.

Blast monitoring

Instantel's brand new blastware 7.0, Windows based event management, reporting and aanalysis software is the companion to Instantel's Blastmate and Minimate vibration and overpressure monitors. The most obvious new feature is version 7.0 is the addition of a powerful frequency analysis(FFT) utility. This allows all users the capability to analyse the frequency content of recorded waveforms, without the purchase of additional analysis software. New features also include the ability to scale the waveforms on event reports in order to provide relevant comparisons between records, and an improved interface for adding post-event notes. In addition scaling and notes can be applied to a selection of multiple events, all with a single mouse click.
Also, reflecting windows functionality and updated icon toolbars-n the Blastware 7.0 menu selections have been organised into a logical format, geared towards compliance or advanced users, allowing easy access to the most popular commands, functions and processes.

The conventional delay detonators are pyrotechnic based, which suffers the disadvantage of cap scattering. The Cap scattering causes inefficient blasting, poor fragmentation, more of ground vibrations and other associated problems. The electronic detonator provides the accuracy of delay of 1ms. Usually the electronic detonator system consists of three key components: digital detonator, logger and blaster. THE PROGRAMMABLE DIGITAL DETONATOR CONTAINS A MICROCHIP ENERGY STORAGE CAPACITOR, SAFETY STRUCTURES AND CONVENTIONAL EXPLOSIVE COMPONENTS. The microchip circuitry includes an oscillator for timing, memory for retaining its programmed delay, and communication functions to receive and deliver digital messages to and from the control equipment. The capacitor can store sufficient energy to run the microchip independent of external power for 8secs with enough energy remaining after this time period has elapsed to fire the fuse head. Each detonator is 1 to 8000ms. The accuracy is less than 1ms. Logger is used to communicate with the detonators during hookup. Operating at an inherently safe voltage, the logger recognizes and tests each detonator as it is clipped on to the harness wire. Each detonator responds up on connect, giving the operator reassurance of knowing the connection is good, and that the detonator has responded . The required delay time for each detonator is entered and writing into the logger memory. The information is stored in non-volatile memory, (hard memory) of the logger and used to program each detonator only during the firing sequence. At any stage the logger can be used to test the hoop-up and response from every detonator. Blaster is used to fire the blast and only deployed from the firing position once the operation is clear of personal. It is only piece of equipment that contains the required voltages and codes capable of firing the detonators. The blaster communicates to each detonator in turn via the logger. Each blaster can handle 8 loggers and each logger can take 200 detonators giving a system capability of 1600 detonator per blast. At any time, connected detonator can be tested and there is a full two-way communication between detonators and control equipment.



1- Base charge PETN, 2- Primary Explosive (Lead azide)
3 - Fuse head (bridge wire) 4 - Integrated circuit chip,
5 - Capacitor, 6 over voltage protection, 7 - Lead in wires
8 - Sealing plug

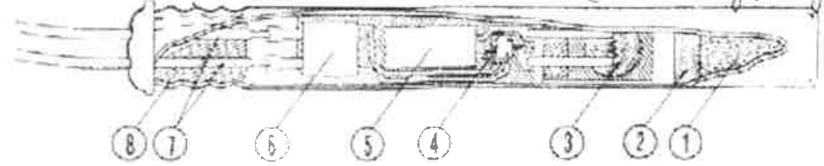
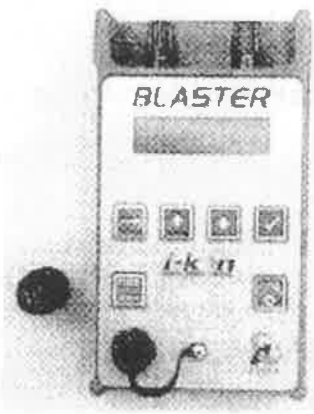


Figure The Nitro Nobel electronic detonator. Persson, 1993, Anonymous, 1993a.

i-kon™ Logger

- Used on bench or at face
- Inherently Safe device
- Four operating modes:
  - Manual, Auto, Number and SHOTPlus-i
- Four language capability:
  - English, Spanish, German & French
- "Respond-on-Connect"
  - Audible "beep" confirmation
  - Verifies detonator status
  - Reads & writes detonator ID into memory
  - Assign delay times
- Testing:
  - Verifies system integrity
  - Error reporting
- 200 detonator capacity

i-kon™ Blaster 400



- Used ONLY from a point of safety
- Four language capability:
  - English, Spanish, German & French
- Unique security key
- Blaster functions:
  - Calibration
  - Programming
  - Error reporting
  - Arming & firing
- 10 Minute blast firing window:
  - Auto abort
- Controls 2 Loggers:
  - 400 detonator capacity

i-kon DIGITAL ENERGY CONTROL

SHOTPlus®-i

- i-kon™ Blast Design Tool
  - Produces plans for Logging
  - Blast simulations
  - Timing analysis
- Download capability
  - To simplify and speed up operations
  - To change the design "on-the-fly"
- Upload capability
  - Archiving
  - Regulatory reporting
  - Simulation
  - Printing

ORICA

EXEL OR RAYDET USAGE IN THE FIELD

Alternative I: EXEL/Raydet DTH in-the-drillhole

D Cord II and CORD RELAY on the surface

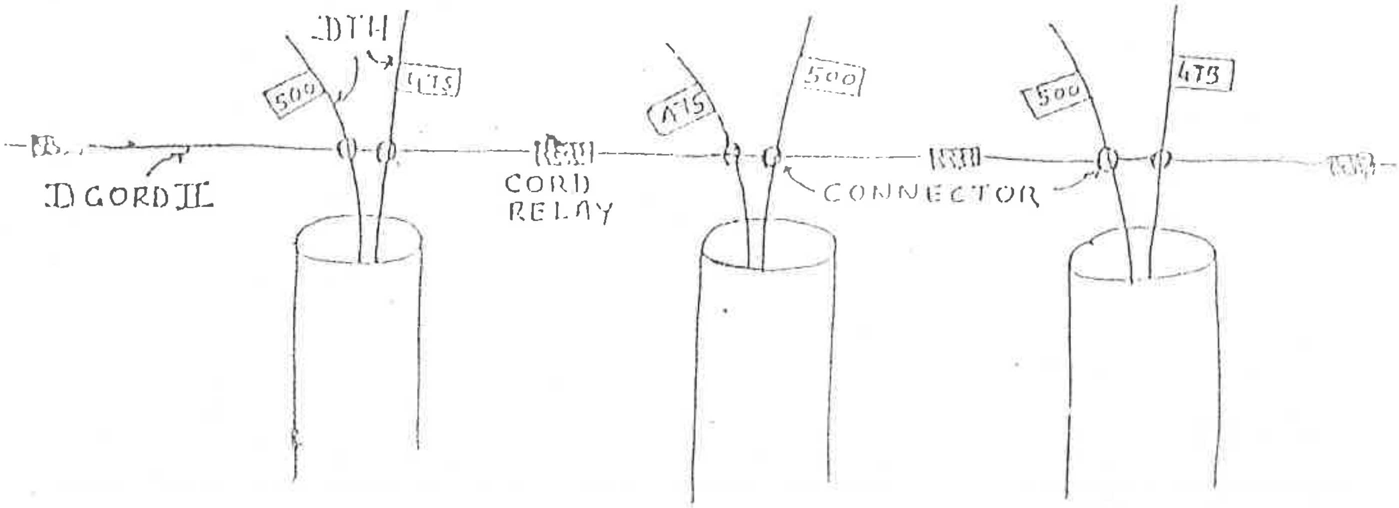


FIG.5 USE OF DOWN-THE-HOLE DELAY DETONATOR WITH A DECK IN COMBINATION WITH DETONTING CORD AND CORD RELAYS ON SURFACE

Alternative II : FOR AIRBLAST/NOISE CONTROL

EXEL OR Raydet DTH in-the-drillhole

EXEL OR Raydet TLD on the surface

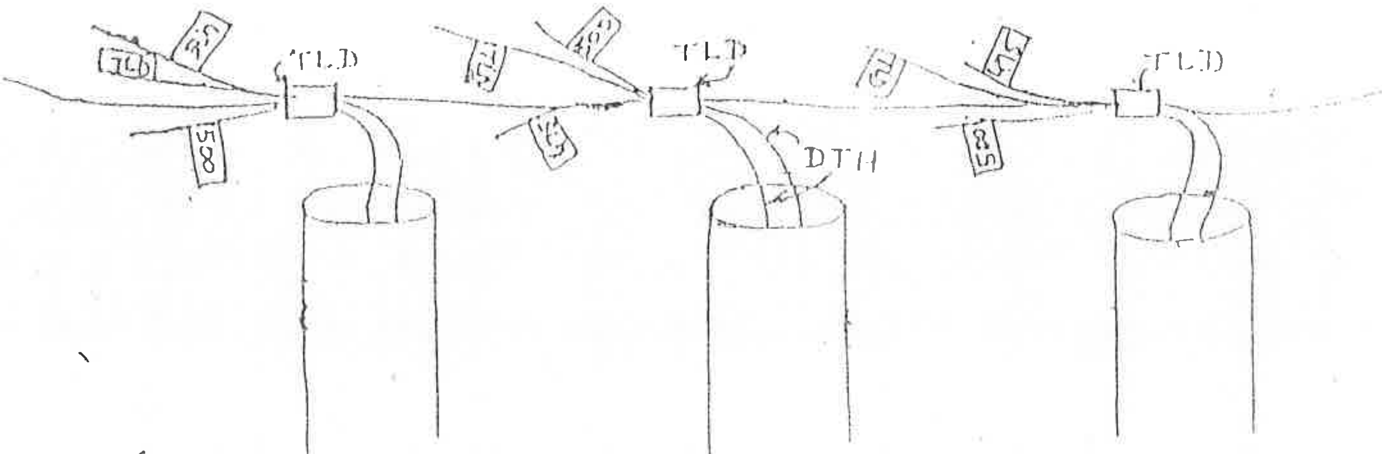
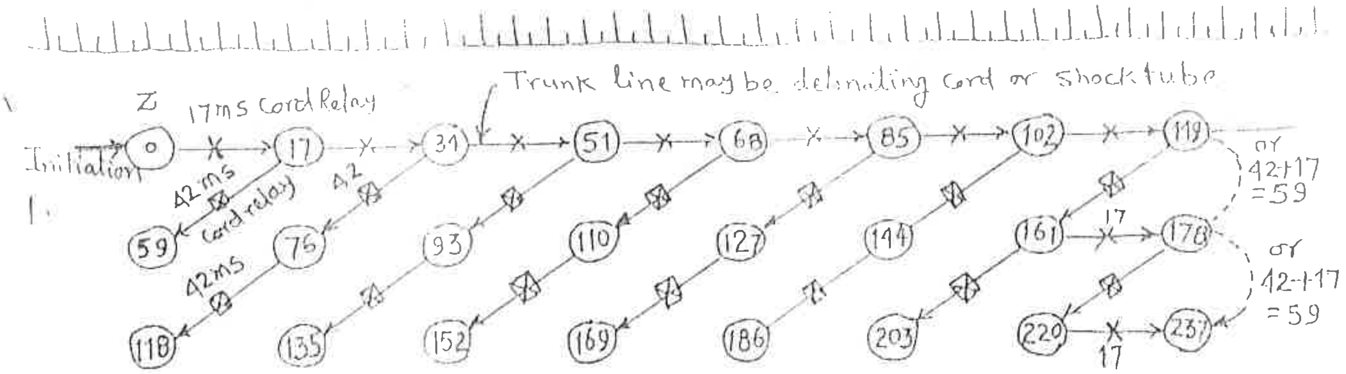


FIG.6. USE OF SHOCK TUBE DTH DOWN THE HOLE AND SHOCK TUBE TLD ON THE SURFACE



(DO NOT FIRE MORE THAN 3 ROWS PARALLEL TO FREE FACE IN A ROUND)

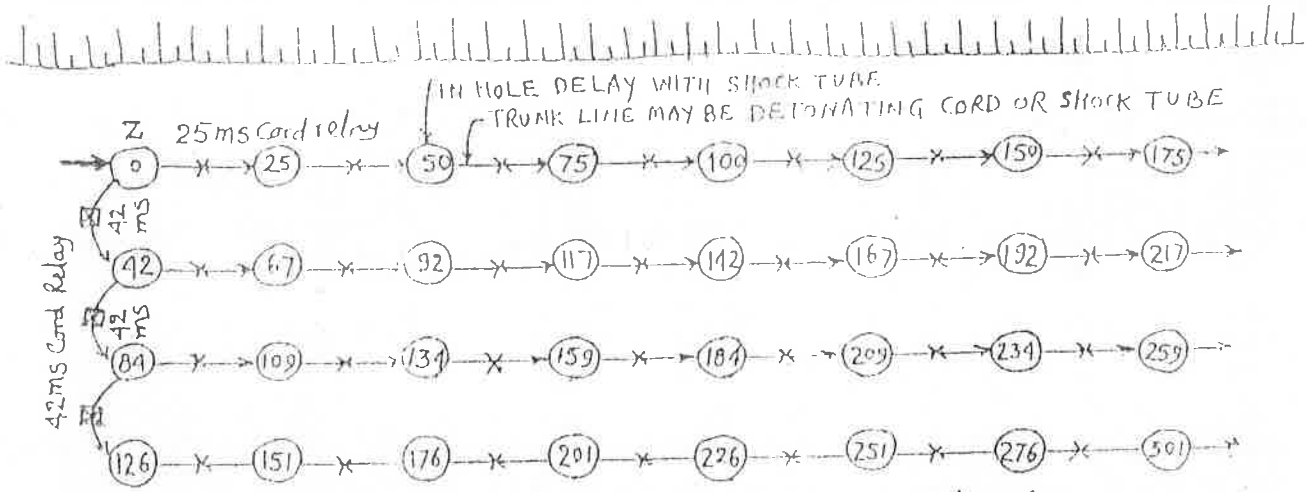
TLD CORD RELAY OF 17 ms ACROSS SPACING

TLD CORD RELAY OF 42 ms ACROSS BURDEN

EFFECTIVE DELAY INTERVAL ACROSS BURDEN = 59 ms

INHOLE DELAY WITH SHOCK TUBE

FIG.7a DRILLHOLE INITIATION PATTERN USING ONE DRILLHOLE INITIATING AT A TIME.



WITH 4 ROWS FIRING TIME WILL DUPLICATE IN THE LAST ROW

(DO NOT FIRE MORE THAN 3 ROWS IN A ROUND)

TLD CORD RELAY OF 25 ms ACROSS SPACING

TLD CORD RELAY OF 42 ms ACROSS BURDEN

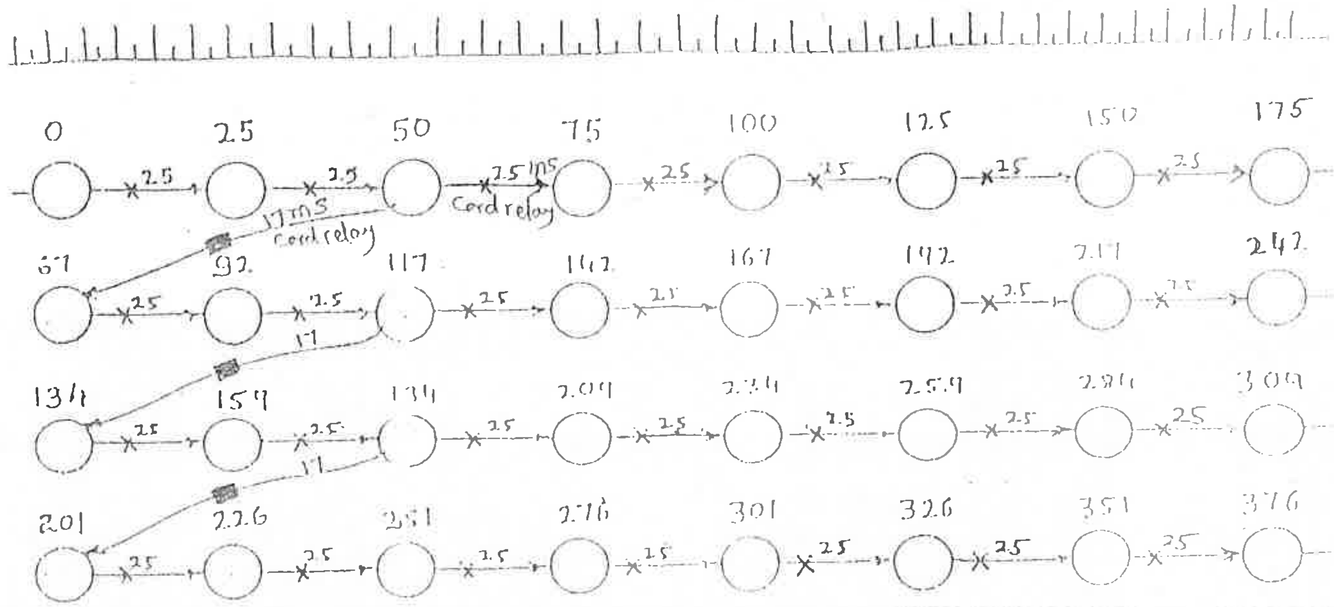
FIG.7b ROW PATTERN OF DRILLHOLE INITIATION USING ONE DRILLHOLE INITIATING AT A TIME.

FIG 8a

EXEL OR RAYDET TLD 25 MS ACROSS SPACING

EXEL OR RAYDET TLD 17 MS ACROSS BURDEN – Tube length as per requirement

EFFECTIVE DELAY INTERVAL ACROSS BURDEN : 67 MS

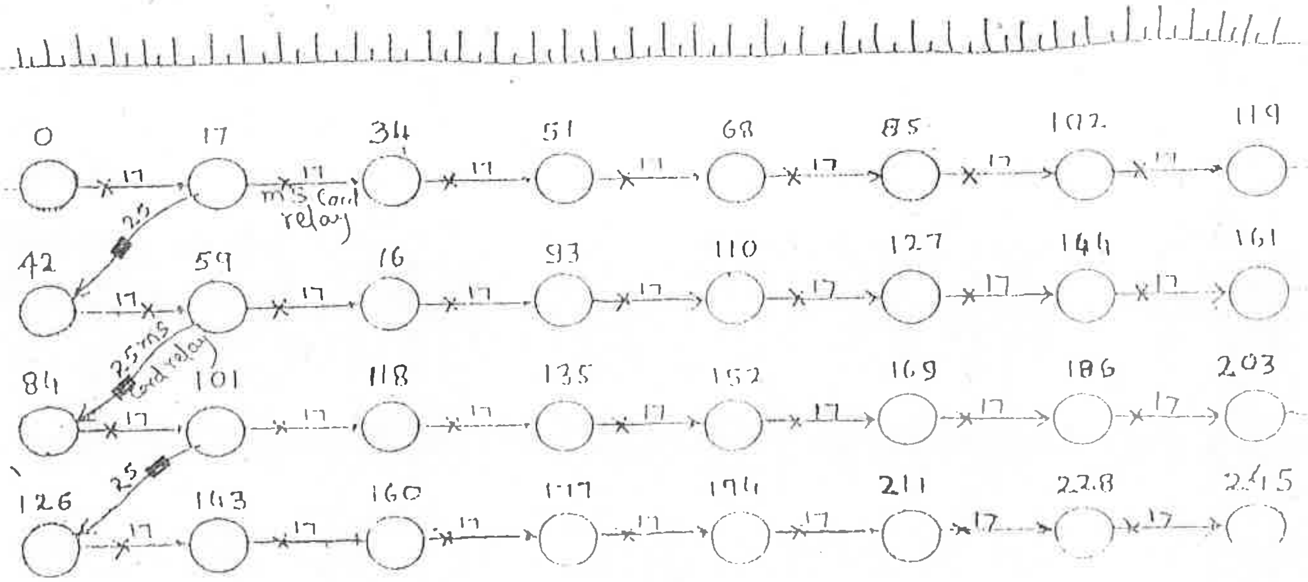


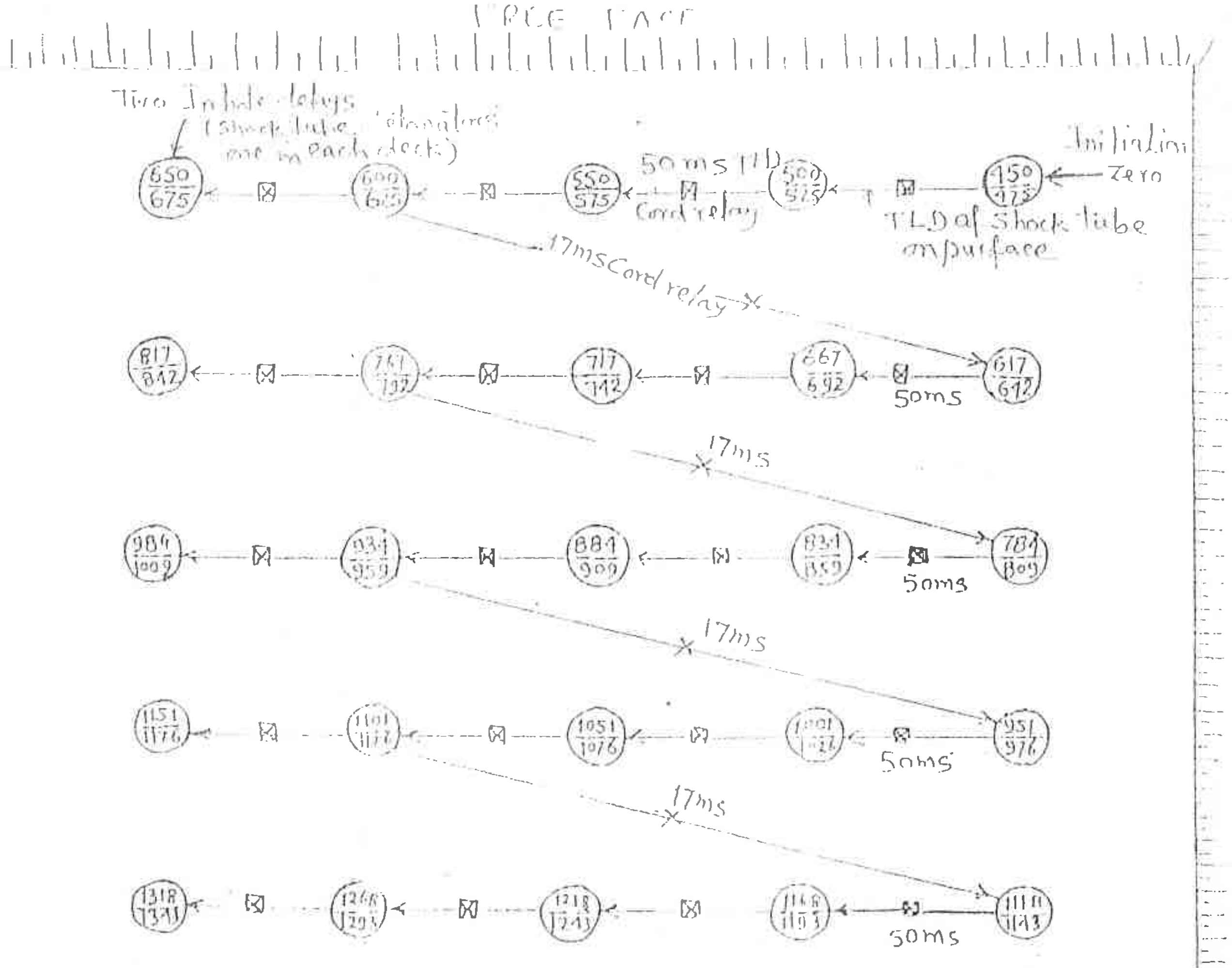
EXEL OR RAYDET TLD 17 MS ACROSS SPACING

FIG. 8b

EXEL OR RAYDET TLD 25 MS ACROSS BURDEN

EFFECTIVE DELAY INTERVAL ACROSS BURDEN : 42 MS





(DO NOT FIRE MORE THAN 20 HOLES OR FOUR ROWS IN A ROUND OF BLAST)

NUMBER INDICATES DELAY TIMINGS / TIMING OF DETONATION IN SIDE THE HOLES

FIG.9: HYPOTHETICAL BLASTING PATTERN USING IN-HOLE DELAY DETONATORS AND 50 ms TLD CORD RELAY.

Precise Digital Drilling

1. INTRODUCTION

Drilling and blasting has been used for centuries to excavate all types of tunnels, whether mining or civil. The drill and blast method of tunneling is preferred over mechanical excavation due to various reasons, like, if the shape of the tunnel is not circular, utilization coefficient of the machines is reduced in cases of poor tunnel stability, high investment costs of the machines and the long setting up and dismantling times hinder their economic use in short tunnels, in large-diameter tunnels,

Mechanization had already been introduced in all activities of the excavation cycle like mucking, supporting and surveying but drilling and blasting continued to be done with conventional methods. Thereafter drilling was mechanized with the aid of jumbos with up to three booms that helped drilling longer rounds accurately and quickly with significant manpower reduction.With the advent of globalization things changed very fast and today it is an industry that is driving the economies of several nations. Global competition has propelled countries to reach higher production levels through better techniques of drilling and blasting & excavation. We now have bigger and faster drill machines and excavators. In Explosives technology too significant progress has been made towards having safer explosives and accurate initiating systems that have increased overall control over blasting in terms of vibration, fragmentation, throw, fly rock and overall blast economics. As an example, Aitik Project of Boliden mines of Sweden has nearly achieved its target production of 36million tons/year of ore by mechanizing and computerizing all its mining, crushing and refining processes.

. In Open pit blasting the scenario was much better with mechanization in charging already introduced with bulk explosives. Bulk explosives not only raised charging rates but also reduced explosives handling to bare minimum levels. But why was underground blasting left behind? The basic reasons were the harsh conditions existing in underground mines and relatively smaller diameters to be loaded. As compared to the product used for opencast blasting a more robust explosive and a reliable pumping system were needed.

Various explosives manufacturers all round the world came up with underground bulk emulsion in the late eighties. The idea was to hasten underground production through reduction in drilling and blasting cycle time. The challenge was to build a reliable charging system tailor-made for explosive delivery in underground excavations. In the late eighties a few explosives manufacturers came up with their charging systems for underground. These systems had inherent advantages of pumped emulsion explosives like speed and safety, besides introducing a concept of flexibility of product. Changing the ratio of emulsion premix and chemical sensitiser could control the final density of the product. This provided the flexibility of controlling energy of product delivered to each hole. Product flexibility leads to greater advance per round and lower damage at the backs for better roof stability.

2. RECENT DEVELOPMENTS IN DRILLING TECHNOLOGY

Drilling is the first unit operation in mining or excavation process. Basically we have three methods of production drilling namely rotary, percussive and down the hole (DTH) drilling. There are few other innovative methods as well but their applications have been restricted to few areas or are still in experimental stage, like laser drilling, water jet drilling etc.

The developments have been mainly done in automation of the drill rigs, improved design of bits, rods, bit inserts, and the bailing systems. The following paragraphs will briefly discuss the automation and developments in surface and underground drill rigs.

2.1 RAPID DEVELOPMENT

The rapid development of excavations has been achieved traditionally using latest equipments and techniques in drilling, blasting and ground support. Innovative underground bulk explosives with automated precise drill equipments and faster bolting systems have compelled the construction companies to compete with each other over productivity. They are using the latest in mucking and hauling as well. Presently the rapid development is buzz word in construction community as it provides the advantage of following:

- Less time to complete the project
- Early beginning/mobilisation of excavation
- Time makes or breaks a project
- Reduce fixed cost expenses of project

Drilling equipment is at its most productive when drilling, hence where geological conditions allow, it is recommended to use longer feeds than are traditionally used in excavations. The longer feeds when coupled with computer control systems, onboard drill plans and accurate positioning systems ensure that a face round achieves maximum pull length. Consistent accurate drilling also contributes to better blast fragmentation, less damage to the surrounding rock mass, potential to fine-tune drill patterns and ultimately safer-faster advance rates. To further improve drilling accuracy there have been several innovations in drill steel design. The traditional HEX 35 rods are losing popularity to round 39mm steels which provide a much stiffer drill string, of high importance to reduce in-hole deviation when drilling longer rounds. Thread design has also been improved with options allowing longer life and better transfer of percussion energy from the rock drill to the drill bit.

Underground bulk emulsion explosives (Site sensitised emulsion explosives) with its water resistance and reduced noxious gases along with electronics detonators provide flexibility and automation in blasting for faster development. The mucking has been made easy with deployment of large capacity loaders with matching haulage equipment which are having High speed on grade, easy manoeuvring and minimised cycle time. The ground supporting has also been made faster with the use of purpose built mechanised and automated equipment having navigation and logging system.

With the automation of drill rigs it is now possible to make drill pattern and drilling sequence created on the PC in the office and transferred to the rig using a USB flash drive/stick. If the operator puts the boom in automatic mode it moves to the next hole in sequence, collars and drills the hole automatically. The drill hole data is logged on the USB stick which can be uploaded back in office PC (Mishra & Sen, 2010). Even manual drilling of difficult holes can be done effectively (Sometimes perimeter holes are required to be drilled manually depending on the rock conditions and profile).

The software has been developed by Atlas Copco which has made the job of mining/excavation engineer easy. This software (Tunnel Manager®, It is registered trade mark of Atlas Copco.) can be used to design the drill pattern required to match the rock properties, profile of the development heading and explosive properties (Figure 1). The drill plan can be effectively designed along with bolt plan design, tunnel line definition, tunnel laser definition, drill round reports, Measurement While Drilling (MWD).

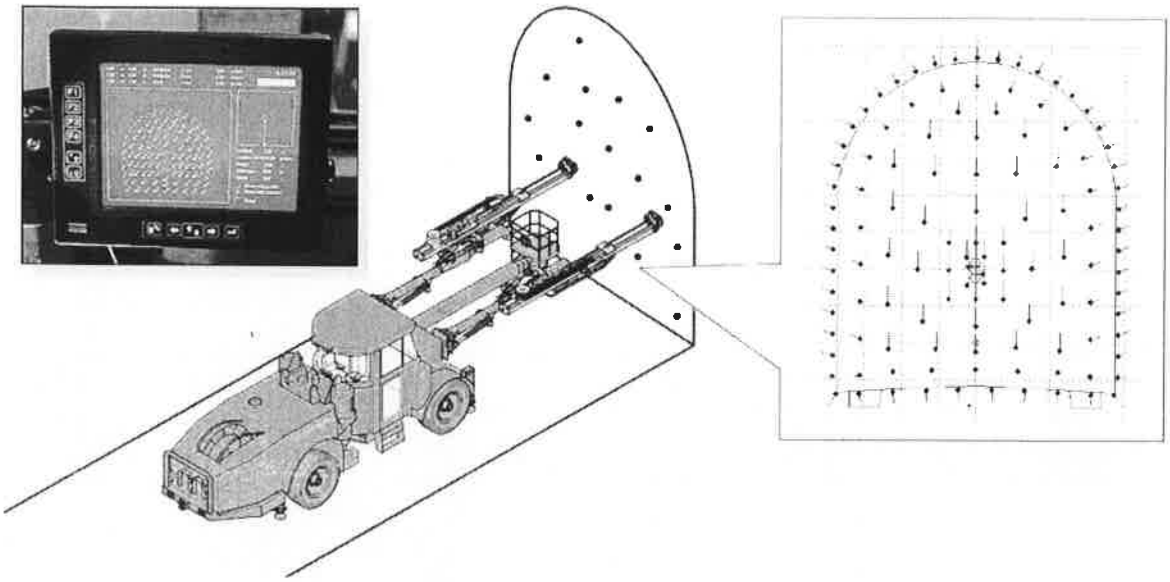


Figure 1: Application of Tunnel Manager®

This software offers various advantages over traditional system of design and implementation. With this Advanced Boom Control (ABC) is achieved. It makes easy boom navigation, good profile, less over break/under break, higher productivity, less time in completion of project, no cost overrun, and of the last project cost savings.

2.2 MEASUREMENT WHILE DRILLING

The technique to extract rock mass properties while drilling is called "Measure While Drilling" (MWD), which can record up to 8 parameters, namely penetration rate, feed force, percussive pressure, rotation pressure, rotation speed, damp pressure, water pressure, and water flow. Today Atlas Copco offers a system for both registration and evaluation while most of the other suppliers provide registered data but do not offer any evaluation. By evaluation of the data we can get an inferred picture of the geological conditions. MWD does

not have much of an application for a face blast, as not many people have the time to do a MWD analysis of a drilled face and then make an adjustment of blast design to take the MWD findings into account. MWD is best employed for probe-hole drilling ahead of a face in order to gain understanding of upcoming geological conditions. This is most useful in tunnelling where we think there could be bad ground – e.g. once bad ground is identified using MWD, the engineers can order pipe-roofing to pre-support the ground before the tunnel face reaches the poor area.
For mining, MWD has applications for long hole production drilling in order to help define ore body boundaries. Hence blast design can be adjusted in order to reduce dilution.

It also reduces the chances of unexpected decrease in pull and over break, hence improves the working safety at the site apart from reducing the extra cost of, loading-hauling, scaling and supporting. This further helps in avoiding the unexpected encounter of difficult rock conditions which can hamper the progress rate or the site safety.

2.3 RIG REMOTE ACCESS

Rig remote access is another feature which is gaining popularity. In this system the rig is connected on line with the mine control room and even can be connected to the manufacturer or head office via internet and diagnosis is done before getting down the mine (Figure 2). The rig can also get an update of drill plans without any delay. It offers the following advantages:

- Production planning– The drill rig is always on line
- The drill rig operator has always access to the latest production planning
- Operator– You do not need to update PC-cards before each shift
- Log files will automatically be saved to the planning department
- There is no need for PC-card handling, what so ever
- Maintenance– Service technician can diagnose the problem before travelling to the drill rig
- Fault diagnoses are easily done on line
- You can plan service based on actual needs

2.4 TUNNEL PROFILER

Automatic tunnel profiler is also available on the drill rigs which offer great advantage over conventional system of profiling (Figure 3). This presents the following advantages:

- Fast profiling, 5 minutes to scan a 65m^2 roof
- Result data, profile and drill log, all combined
- One set-up and navigation only
- Operator is advised about under breaks
- Over break is calculated/presented to the operator
- Reduced need for surveying work
- Scanning while drilling next round and so time is saved
- Change in design of blast round depending on the feedback of profile

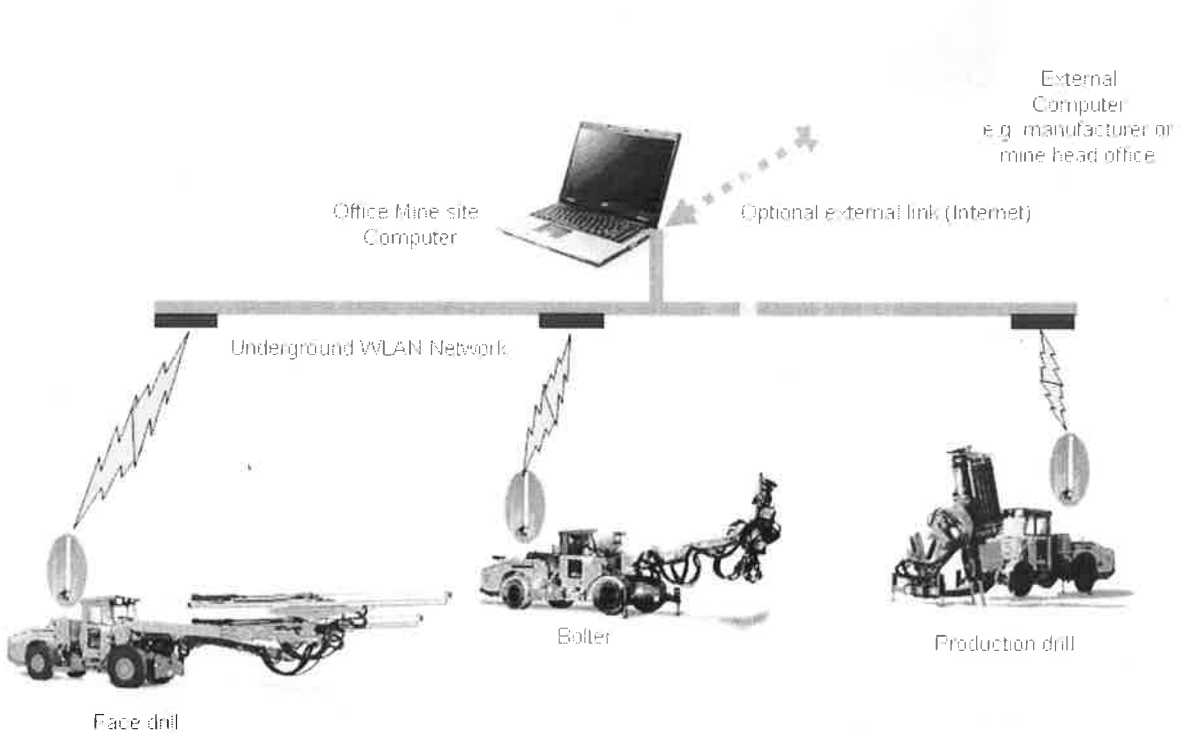


Figure 2: Rig remote access system

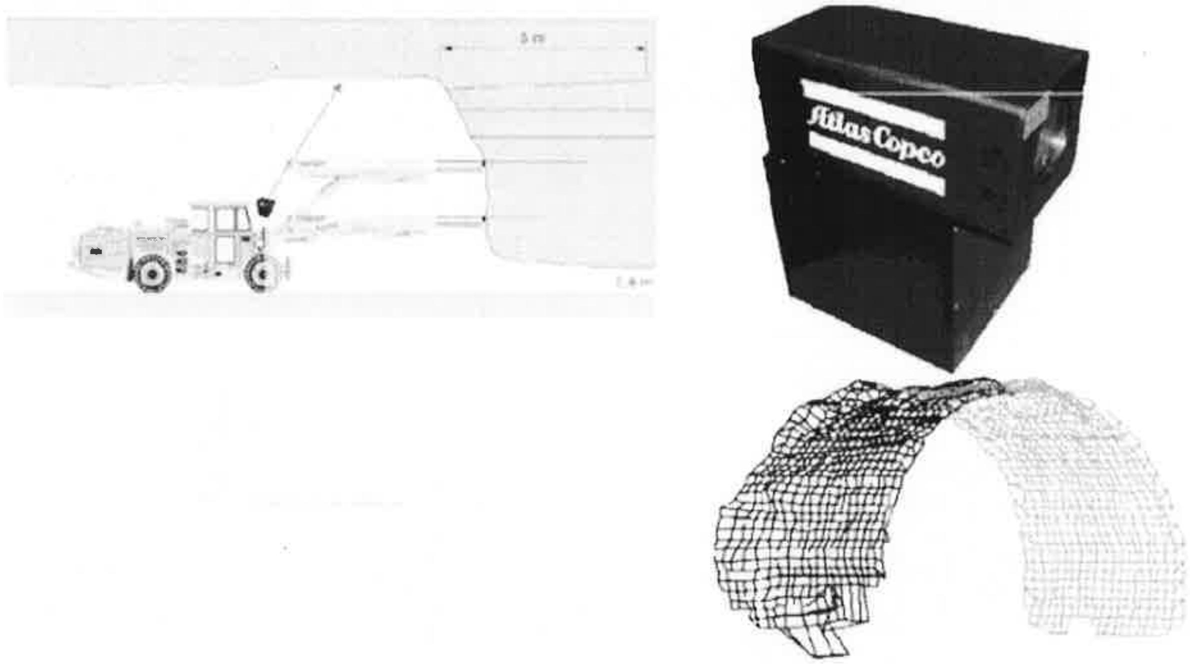


Figure 3: Tunnel Profiler

Globally Drilling and blasting technology has been an area of keen interest and is improving very fast. Products like **digital drilling and drill navigation systems** have given mining companies the space to operate for better productivity and efficiency. Rapid development is buzz word among the mine operators and construction companies. The emphasis is on technological up gradation and automation of drilling and blasting.

3.0 DIGITAL DRILL JUMBO FOR UNDERGROUND MINES/TUNNELS

3.1 TAMROCK has developed TMS JUMBO (Tamrock Super Drilling and Power Class 1-3 boom jumbos) for underground drilling which is in use in mines and tunnels. It is capable of measuring and maintaining at its own the drill feed direction, hole depth and penetration rate. All data are displayed numerically for the selected boom. Boom selection is done using interface keyboard. It has the following components:

1. HARDWARE
--- I/O modules and computer unit for measurement and calculations
2. USER INTERFACE
--- Numerical displays and keyboard for user interface
3. SENSORS
--- Carrier inclinometers
--- Inclinometers for tilt angle
--- Angle sensors for swing angles of boom and drill feed
--- Position of linear movement: drill depth (DDSS version only).
Not available for TTF CF or feeds.

TMS alternatives
1. TMS Dx JUMBO Tilt and look out angles of drill feed
2. TMS DDSSx JUMBO Tilt and look out angles of drill feed, hole depth measurement, penetration rate and
cumulative drilled meters counters + drill "stop and return" at preset hole depth
Note1! "x" stands for number of booms (1---3)

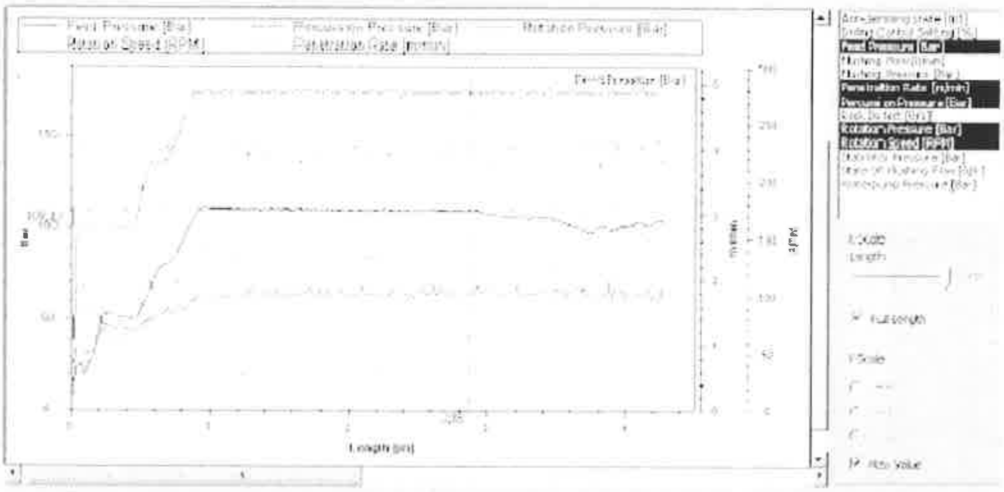
3.2 SANDVIK'S MEASURE WHILE DRILLING (MWD) SOFTWARE (ANALYSIS)

Measure While Drilling Software (Analysis)
The analysis software module makes analyzing MWD. MWD data collection can be turned on in the drill plan design phase for any number of holes or it can be turned on from the drill rig used interface. Data is collected while drilling and stored in to the drill rigs data collection files. The data can be used to analyse the performance of the drill rig or to see the changes in rock condition

Collected MWD parameters
- Anti-jamming state
- Drilling control setting (%)
- Feed pressure (bar)
- Flushing flow (LPM)
- Flushing pressure (bar)
- Penetration rate (m/min)
- Percusion pressure (bar)
- Rock detect state
- Rotation presssure (bar)
- Rotation speed set point (RPM)
- Stabilator pressure (bar)

• State of flushing flow
• Water pump pressure (bar)

MWD data can be visualized as colour coded freely rotated 3D-images. When the same data from numerous holes is drawn in the same picture with the data in the exact coordinates it was measured, a picture of different areas in the rock starts to form. The colour mapping of the vizualization can be modified to highlight selected values in the measurement.



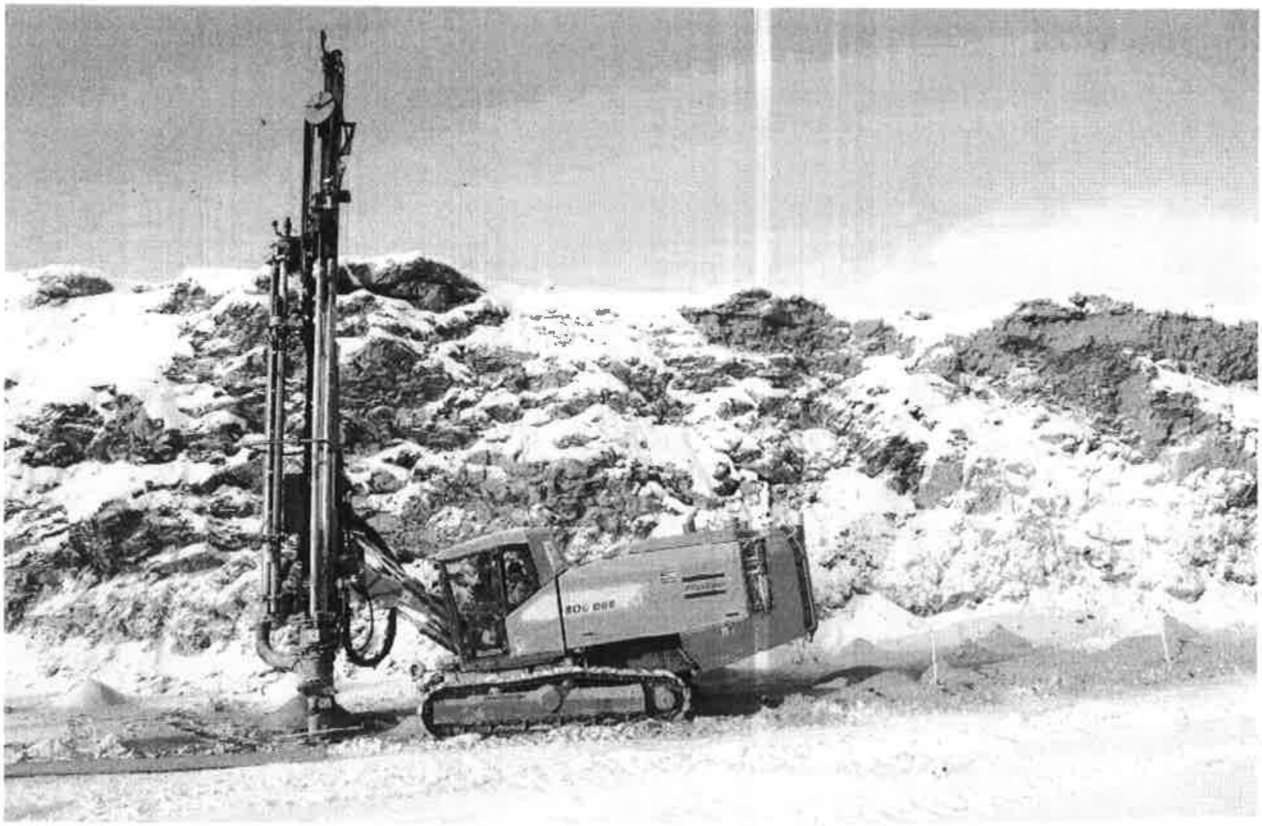
4.0 DIGITAL SURFACE DRILL RIGS

4.1 ATLAS COPCO SMART RIG- ROC D65

Atlas Copco has elevated open pit mining to a new level with the launch of its latest down-the-hole drill rig – the SmartRig ROC D65. This rig is the first of its kind in that it combines all the advantages of the well proven ROC L8 DTH drill rig with the advanced automation and control of the SmartRig family, thereby paving the way for new levels of productivity in surface drilling applications.

Atlas Copco has long and broad experience of its computerized Rig Control System (RCS) that forms the basis of the SmartRig platform. With the SmartRig ROC D65, all the benefits of this system are now also available for open pit miners. RCS controls the entire handling of the drill rig, including everything from the drilling cycle to automatic tube handling and the optional hole navigation system. The system also facilitates the easy transfer of planning and performance data between the rig and the mine office.
The SmartRig ROC D65 is designed for drilling in the 110–203 mm hole range. It uses Secoroc COP 44, 54 or 64 down-the-hole hammers to a maximum depth of 54 metres.The drilling power is provided by an onboard Atlas Copco XRX10 compressor that supplies 30 bar pressure while a Caterpillar C15 engine, rated at 539 hp powers the rig.

One example is that the rig can automatically add and retract drill tubes, which relieves the operators of this tedious task so that they are free to prepare materials on the bench while the rig completes the hole by itself.
The new SmartRig ROC D65 has been undergoing rigorous testing by mining contractor NCC Roads at the Aitik copper mine in northern Sweden where it has completed some 45 000 drillmetres in about six months.



Caption: The SmartRig ROC D65 complements Atlas Copco's range of surface drilling Rigs, offering an advanced drilling solution for every mining and construction application

4.2 ATLAS COPCO'S - ROC Manager

Fast and accurate drilling with consistently good blasting results and the documentation to prove it. What better way to control your costs and grow your reputation! It's all within your grasp with Atlas Copco's ROC Manager.
The ROC Manager software system contains a set of integrated functions for:
Generating and editing drill plans.
Evaluating the results of drilling from data logged by the rig's control system and hole deviation data obtained from a bore-hole probe.
Generating hard-copy reports of drill plans, drilling results and hole deviation data.
ROC Manager runs on a stand alone Windows PC. Drill plans are produced on the PC and transferred to the rig by means of a PC card. Logged drilling results are

transferred back to the PC for analysis in the same way. And the planning and evaluation data for multiple projects across different sites can all be recorded in a single common database.

Benefits

Accurate drill plans,
Better blasting results – more even fragmentation, less overbreak and flat new bench floor
Safer blasting - lower risk of flying rocks
Clear presentation of logged drilling results including, if available, graphical representation of MWD (Measure while Drilling) data that can be used to analyse the kind of rock formation being dealt with and refine the drilling operation accordingly
Savings in time and money leading to better overall economy.

ROC Manager

Drill plans are developed in the office on a PC. They are then copied to a PC card, which is transferred to the drill rig. There the plans are loaded into the rig control system.
The drill rig operator uses the drill plans to drill the holes in each round.
Results, including MWD data if available, are logged by the rig control system, copied to a PC card and transferred back to the offiice for evaluation.
Hole deviations can be measured with a bore-hole probe and stored on a laptop PC. This data can also be transferred to the office PC and integrated with the data logged by the rig.

Several different analysis screens are available. MWD results can be seen in graphical form by clicking on "Graph". Drilling results are analysed offline in the office by specialists and can be used to refine the drill plans for the next rounds.

A number of hard-copy reports can be generated, showing drill plans and results in the form of graphs and tables. These can be useful for the rig operator while rilling, for specialists to evaluate rock conditions and adjust plans accordingly, and as tatus reports for management or customers.

Data for different work sites, such as drill plans, are stored in a hierarchical tree structure. This gives an excellent picture of work done and of work in progress and brings a high degree of order to the operation.

ROHTANG TUNNEL

WORLD HIGHEST MOTORABLE TUNNEL

श्रमेण सर्वम साध्यम

- The **Rohtang Tunnel** is **8.80 km** long bi-directional highway tunnel and is under construction by **Border Roads Organisation (BRO)** in Pir-Panjal ranges of Himalayas to connect the District Kullu with Lahul & Sipiti in Himachal Pradesh.
- It is the highest road tunnel being constructed at an altitude of 10,075 ft above MSL
- Tunnel likely to be opened for traffic in 2019.

Tunnel breakthrough on 15 Oct 2017 in presence of Hon'ble Raksha Mantri Smt Nirmala Sitharaman