ISRM International Symposium 2006

4th Asian Rock Mechanics Symposium

Rock Mechanics in Underground Construction

C. F. Leung • Y. X. Zhou

Editors

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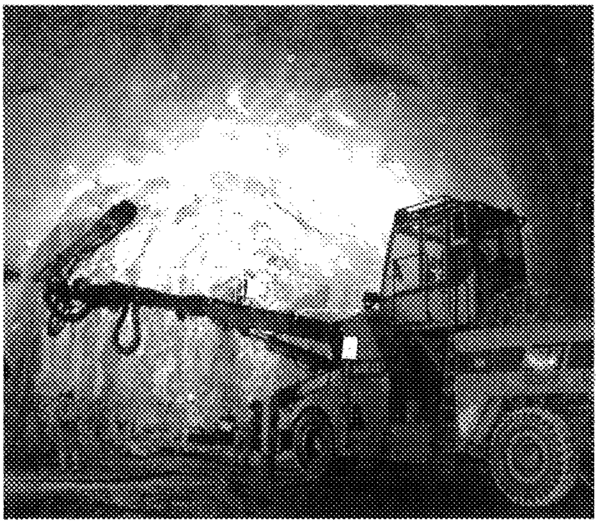
8 – 10 November 2006 Singapore



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PREFACE

The 4th Asian Rock Mechanics Symposium (ARMS) received overwhelming response for its call for papers in early 2006 with about 450 abstracts received by the Organising Committee. After a rigorous selection process, just over 300 papers were finally accepted for the proceeding, a record for ARMS. This is also the first time that the ARMS proceedings volume consists of printed copies of full papers of keynote lectures and extended abstracts of all the technical papers while the full technical papers are provided in a CD-ROM. This has enabled the Organising Committee to accept as many high quality technical papers as possible.

The theme of the Symposium is "Rock Mechanics in Underground Construction". Fittingly all the seven keynote lectures from Asia, Australia, Europe and North America deal with underground rock engineering topics. In fact, about half of the technical papers concern with underground construction such as tunnelling, rock caverns and underground mining. In addition, a large number of the remaining technical papers are directly or indirectly involved with rock mechanics in underground construction. Although the majority of the technical papers are contributed by rock engineers and researchers from Asia, the editors are glad to note that there are considerable number of contributions of high quality technical papers from many countries outside Asia.

The contributions of the technical paper reviewers and the ARMS 2006 award selection committee members are gratefully acknowledged. They play important roles to ensure that the papers in this proceedings volume are of high standard. The editors would like thank the able compilation and thorough checking of the scripts by Ms Chelsea Chin and her colleagues from World Scientific Publishing Company, and the diligent assistance of the staff from the symposium Secretariat, Meeting Matters International. With the efforts of all the above persons, the editors hope that this proceedings volume will serve as a useful reference for the engineers and researchers in rock mechanics and rock engineering.

C. F. Leung
Y. X. Zhou

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KEYNOTE LECTURES

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FORENSIC ENGINEERING FOR UNDERGROUND CONSTRUCTION

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In the context of underground construction, forensic engineering is taken to be the application of engineering principles and methodologies to determine the cause of a performance deficiency, often a collapse, in an excavation, and the reporting of the findings, usually in the form of an expert opinion within the legal system. The procedures that may be used in forensic geotechnical investigations and the interface of the engineer with the legal system are discussed. The application of the principles and methodologies outlined are illustrated through a brief account of the investigation of the collapse of a small part of an excavation in the Lane Cove Tunnel Project, Sydney, Australia, on 2 November, 2005.

Keywords: Accident investigation; failure; forensic engineering; mining; risk; tunnelling; underground construction.

1. Introduction

In common with other areas of geotechnical engineering practice, underground construction involves a number of uncertainties and risks, many of them associated with the inherent variability and unknown properties of the geological materials involved. These and other factors may lead to deficiencies in excavation performance, to the collapse of an excavation and, on occasion, to the loss of life.

It is inevitable that, when an underground failure or collapse has occurred, either in civil engineering construction or in mining, an investigation will be carried out into the causes of the failure. Depending on the nature and severity of the failure, this investigation will be carried out, at least in part, by suitably experienced specialist engineers. In many cases, some form of legal proceedings will follow, either to determine the causes of damage, loss of income or, in some cases, the loss of life, and/or to resolve contractual and responsibility issues and allocate costs. This process will also involve specialist engineers, usually as expert witnesses. The professional engineering work carried out in these cases has come to be described as forensic engineering.

This paper discusses the general nature of forensic engineering and the special issues and difficulties confronting forensic geotechnical engineers, particularly in the investigation of failures or collapses in underground construction and underground mining. The investigation methods used are outlined and the important interface with legal systems is discussed. In the author's experience of forensic investigations in underground civil construction and mining, this legal aspect of the forensic engineer's role is becoming increasingly important and demanding, given the increasing proclivity of some authorities to prosecute engineers and their employers in the courts. Finally, a brief account is given of the author's investigation of the collapse of a small section of an excavation in the Lane Cove Tunnel Project in Sydney, New South Wales, Australia, on 2 November 2005.

2. The General Nature of Forensic Engineering

Technological innovation and advances in engineering have always been attended by failure of one type or another, including the quite spectacular collapse of structures such as bridges and dams (e.g. Lewis, 2004; Petroski, 1985, 1994). More recently, as financial losses and the loss of reputation have increased, personal injury and the loss of life have come to be regraded more seriously than they sometimes were in the past, and society has become generally more litigious, an area of engineering practice known as **forensic engineering** has developed. Forensic engineering now has its own specialist societies, consulting firms, conferences, literature and university courses, and has attracted popular attention through television programs and books (e.g. Lewis, 2004; Wearne, 1999).

A number of definitions of forensic engineering are available in the literature. For example, Specter (1987) defines forensic engineering as "*the art and science of professional practice of those qualified to serve as engineering experts in matters before courts of law or in arbitration proceedings*". Similarly, Noon (2001) defines forensic engineering as "*the application of engineering principles, knowledge, skills, and methodologies to answer questions of fact that may have legal ramifications*". Carper (2000) says that "*the forensic engineer is a professional engineer who deals with the engineering aspects of legal problems. Activities associated with forensic engineering include determination of the physical or technical causes of accidents or failures, preparation of reports, and presentation of testimony or advisory opinions that assist in resolution of related disputes. The forensic engineer may also be asked to render an opinion regarding responsibility for the accident or failure*".

Following Lewis (2003) and Noon (2001), in the context of underground construction, forensic engineering will be taken here to be the application of engineering principles and methodologies to determine the cause of a performance deficiency, usually a collapse, in an excavation, and the reporting of the findings, usually in the form of an expert opinion within the legal system. Some uses of the term forensic engineering do not reflect this involvement with the legal system. However, this legal element is central to the definition, recognition and practice of forensic engineering in some countries (e.g. Australia, USA), and will form an essential part of the discussion presented here.

Forensic engineering as defined above is concerned typically with investigations of failures of constructed facilities; rock falls, excavation collapses and other accidents in mines; fires and explosions; air and rail crashes; aspects of traffic accidents; and failures of consumer products. The more serious events of these types can lead to significant injury and to fatalities as well as to financial loss.

Forensic engineering investigations involve a number of steps. In general, the forensic engineer collects evidence of several types and then carries out analyses, again of various types, to determine the "who, what, where, when, why and how" of the deficient performance or failure of engineered facilities, systems and products, including accidents. A range of formal and less formal procedures may be used to guide the investigations (e.g. Greenspan *et al.*, 1989; Lewis, 2003; Noon, 2001). Figure 1 shows the steps typically used in forensic engineering investigations within a civil engineering context. The legal terminology used is that applying in the USA.

Communicating the results is a vitally important stage of the investigation. This communication may be required, not only to facility owners, contractors and other professional engineers, but also in reports to lawyers and statutory bodies, in expert witness statements in legal proceedings, and in statements to the press and the public. In many cases, the expert's report may be confidential or protected by legal professional privilege. It is probable that only a low percentage of cases for which forensic engineering investigations are undertaken and expert witness reports are prepared, actually reach the courts (e.g. Brookes, 2006). However, the information contained in the reports may be protected by legal privilege and remain confidential so that the results cannot be published and the potentially valuable information that the reports contain never reach the engineering profession.

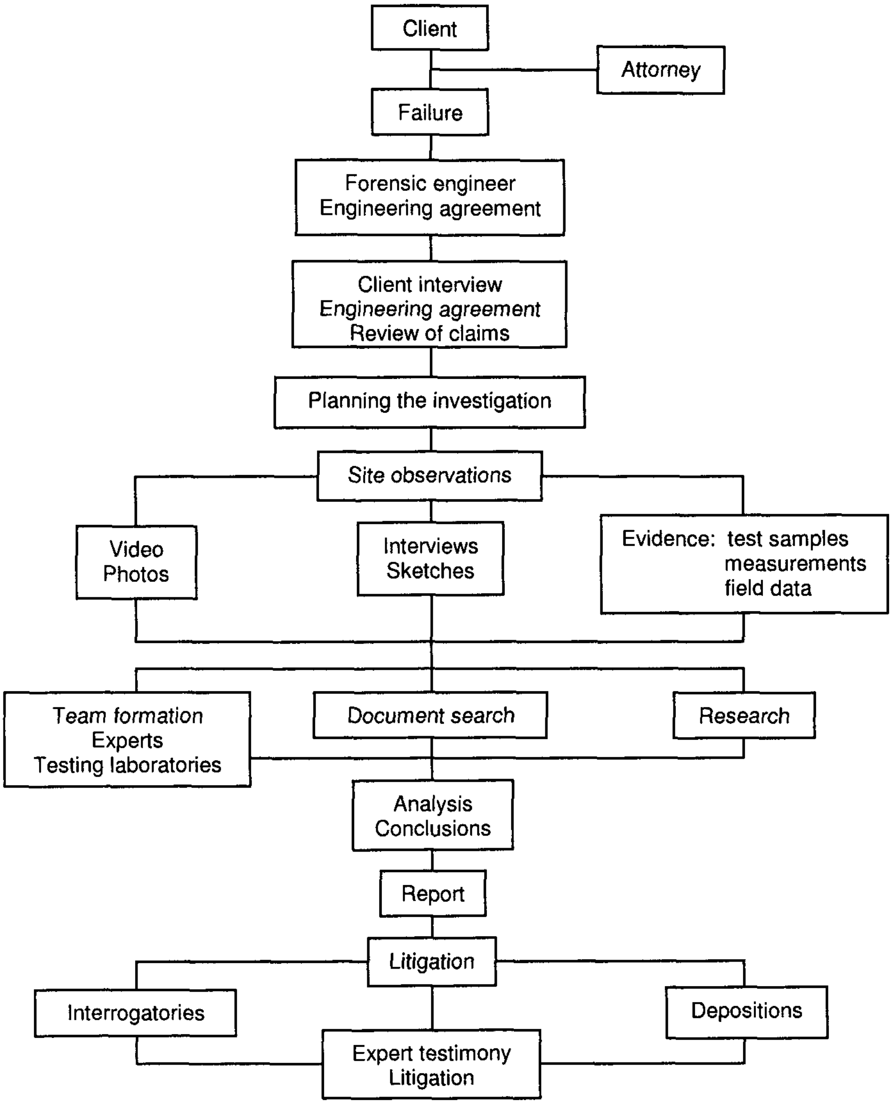

Fig. 1. Flow chart of a typical forensic engineering investigation (after Greenspan *et al.*, 1989).

The important question of what qualifies an engineer to be recognised as an "expert" is discussed in some detail by a number of authorities including Carper (2001), Greenspan *et al.* (1989) and Lewis (2003) who concludes that the key attributes of an expert engineer are "*education, training, experience, skill and knowledge*" and that the engineer must be able to "*perform his or her work accurately, objectively and in a professional manner*". Forensic engineers or engineers serving as expert witnesses have to be especially aware of the ethical practice issues to be discussed in Section 5 below.

3. Forensic Geotechnical Engineering

3.1. *Failures in Geotechnical Engineering*

Engineering in the natural materials found on and under the surface of the Earth has long been fraught with difficulty. Perhaps even more so than in other areas of engineering, there have always been performance deficiencies and failures of geotechnical engineering projects. The most extreme cases, generally arising from dam failures or major landslides, can cause significant damage to property and infrastructure, and the loss of life (e.g. Wearne, 1999). Particular difficulties arise from the inherent spatial variability of soil deposits and rock masses. Errors or omissions having significant engineering consequences can be made in geological interpretations, there can be variations in geotechnical properties over a project site, and apparently minor geological features can have major influences on the performance of engineered structures (e.g. Terzaghi, 1929). Attempts have been made over the last two or three decades to account for some of these factors through probability-based reliability and risk analyses which have now become part of the corpus of geotechnical engineering (e.g. Bea 2006; Christian, 2004; Einstein, 1996; Eskesen *et al.*, 2004). The concept of reliability as "*the likelihood that a system will perform in an acceptable manner*" (Bea, 2006) is important in forensic geotechnical engineering.

It is not surprising, therefore, that studies and analyses of the failures of foundations, slopes, dams and tunnels, for example, have been important in the development of geotechnical engineering research, knowledge and practice. Following Leonards (1982), failure will be taken here to be an "*unacceptable difference between expected and observed performance*". Many failures involve sometimes catastrophic instability which arises when some form of sudden rupture develops. The geotechnical engineering literature is replete with detailed examples and broader studies of the causes of geotechnical failures (e.g. Bea, 2006; Day, 1998; Leonards, 1982; Londe, 1987; Müller, 1968; Osterberg, 1989; Sowers, 1993). Several of these studies include considerations of the influence of human factors on geotechnical engineering failures. For example, Sowers' (1993) study of more than 500 well-documented foundation failures showed that the majority of the failures were due to "*human shortcomings*". Only 12% of the failures studied were attributed to the absence of relevant technical knowledge or solutions.

Generally, forensic geotechnical investigations follow the broad pattern illustrated by Figure 1 (Day, 1998). In many cases, careful and detailed re-investigation of the site is required with detailed geological mapping, drilling, sampling and testing (e.g. Alonso and Gens, 2006a; Skempton and Vaughan, 1993). In order to resolve some cases, similarly careful and detailed analyses of the data, and back-analyses of the problem, are required (e.g. Alonso and Gens, 2006b; Gens and Alonso, 2006; Potts *et al.*, 1990). Although not always falling within the purview of forensic engineering as defined here, similar forensic investigations may also be associated with the repair and/or restitution of historic structures (as in the famous case of the Leaning Tower of Pisa), with geoenvironmental problems, or in the aftermath of natural disasters such as earthquakes. If the results of such investigations can be brought to the attention of the profession through conference presentations or publication, they can make significant contributions to the advancement of geotechnical engineering knowledge (e.g. Londe, 1987).

3.2. *Underground Construction*

Underground construction in soils and rocks can suffer from the same types of errors and uncertainties as those outlined for geotechnical engineering more generally. Although fatalities can occur and severe financial losses may result, the failures or collapses occurring in underground construction do not often have the spectacular or disastrous effects of the major dam failures or landslides referred to above. Tunnelling or the excavation of caverns may be slowed or halted by frequent small or large falls of ground (e.g. Feld and Carper, 1997), squeezing ground conditions (e.g. Hoek, 2001), groundwater problems (Hoek, 2001) including inundation, the sudden development of very large settlements or sinkholes above tunnels or other underground excavations (Shirlaw and Boone, 2005), or the unanticipated and continuing development of excavation deformations (e.g. Stabel and Samani, 2003). Occurrences of the penultimate type have occurred in a large number of projects, including railway tunnels in Singapore (Shirlaw *et al.*, 2003). Such occurrences can have damaging effects on buildings and on surface and near-surface infrastructure, as in the case to be discussed in Section 6 below.

Underground mining in hard or soft rock can suffer from similar problems to those outlined for underground construction. Although the purposes of many of the excavations made and of the operations carried out in underground mining, specifically those associated with the stopes from which the ore is extracted, differ from those in underground construction, modern large-scale underground mining does have associated with it, large numbers of underground infrastructure and transportation excavations which have elements in common with civil engineering excavations. Unfortunately, fatalities arising from broadly geotechnical causes have been all too common in the international mining industry, bringing with them legal proceedings of one type or another and the need for forensic engineering investigations of the general type being discussed here. Rock bursting, which is not unknown in civil construction, has been a particular cause of damage and fatalities in deep, hard rock mining, most notably in the deep level gold mines of South Africa. In addition to collapses underground, underground mining can cause subsidence and disruption to the surface, damaging buildings and infrastructure. As in underground construction, throughout mining history, there have been several major cases of the inundation of mine workings by water or tailings leading to loss of life and of production.

The forensic investigations carried out in these various cases use the general principles and methods discussed elsewhere in this paper. They also require knowledge of the principles of rock mechanics and of the engineering behaviour of rocks and rock masses. De Ambrosis and Kotze (2004) provide an excellent example of the detailed investigation of two large stress-induced roof collapses that occurred in the roof strata of an underground LPG storage facility in Sydney, New South Wales, Australia. The author has had experience of investigations of several of the types of failure identified in this sub-section.

4. Formal Analysis Tools and Investigation Methods

4.1. *Overview*

A wide range of formal investigation methods and analysis tools and approaches are available for use in forensic engineering investigations. Many of them have their basis in hazard identification, risk assessment and risk management or control. Indeed, it can be argued that in underground construction and rock engineering more generally, it is now possible to identify all of the geotechnical hazards likely to have an impact on a project. The forensic engineering task then becomes one of determining which of those hazards contributed to the incident being investigated (Hudson, 2006).

The analytical tools available include a range of specific techniques such as causal analysis, energy/barrier analysis, event trees, fault trees, human error analysis, Petri nets and sequentially timed events plotting (STEP) (e.g. Hadipriono, 2002; Joy, 2004; Kontogiannis *et al.*, 2000). These specific techniques may be incorporated into more integrated analysis methods such as the Incident Cause Analysis Method (ICAM) (Gibb *et al.*, 2004) and overall investigation methods such as the System Safety Accident Investigation (SSAI) approach (Joy, 2004).

These tools and approaches have been developed and applied to accidents and failures of a range of types in a number of industries, including the construction and mining industries. The extent to which they are applied in a given case will depend on the severity or consequences of the incident concerned. Quite often, these tools and approaches may form the basis of the in-house policies and protocols of companies or organizations. In other cases, particularly when major catastrophic events occur, they may be used in accident investigations carried out by external parties. The following outlines of ICAM and SSAI are presented in generic terms rather than in terms that are specific to underground construction.

4.2. *The Incident Cause Analysis Method (ICAM)*

The Incident Cause Analysis Method (ICAM) is an analysis tool that sorts the findings of an investigation into a structured framework. Its fundamental concept is the acceptance of the inevitability of human error. It is based on a conceptual and theoretical approach to the safety of large, complex, socio-technical systems developed by Reason (1997, 2000). This brief account of ICAM is taken directly from that of Gibb *et al.* (2004). The specific objectives of investigations carried out using ICAM are to:

- establish all the relevant and material facts surrounding the event,
- ensure that the investigation is not restricted to the errors and violations of operating personnel,
- identify the underlying or latent causes of the event,
- review the adequacy of existing controls and procedures,
- recommend corrective actions to reduce risk, prevent recurrence and improve operational efficiency,
- detect developing trends that can be analysed to identify specific or recurring problems,
- ensure that it is not the purpose of the investigation to apportion blame or liability, and
- meet relevant statutory requirements for incident investigation and reporting.

Reason (1997) defines ***organisational accidents*** as those in which latent conditions (arising mainly from management decisions, practices or cultural influences) combine adversely with local triggering conditions and with active failures (errors and/or procedural violations) committed by individuals or teams to produce an accident. The Reason Model and ICAM focus on those matters over which management could reasonably have been expected to exercise some control.

The ICAM Model organises incident causal factors into four elements as illustrated in Figure 2. ***Organisational factors*** are the underlying factors which promote the task/environmental conditions that affect performance in the workplace, or allow those conditions to remain unaddressed. They may lie dormant or undetected for some time, and their repercussions may become apparent only when they combine with local conditions and errors to breach the system's defences. ***Task/environmental conditions*** are the task, situational or environmental conditions in existence immediately before, or at the time of, the incident. They are the circumstances under which the errors or violations took place. They can be embedded in the demands of the task, the work environment, individual capabilities and/or human factors. ***Individual or team actions*** are errors or violations of a standard or procedure committed in the presence of a potential hazard that is not properly controlled. They are the actions or omissions that led directly to the incident or accident. ***Absent or failed defences*** fail to provide the required protection to the system against technical or human failures. Both proactive and reactive defences are required to prevent incidents or accidents and to minimise adverse effects if they do occur (Gibb *et al.*, 2004).

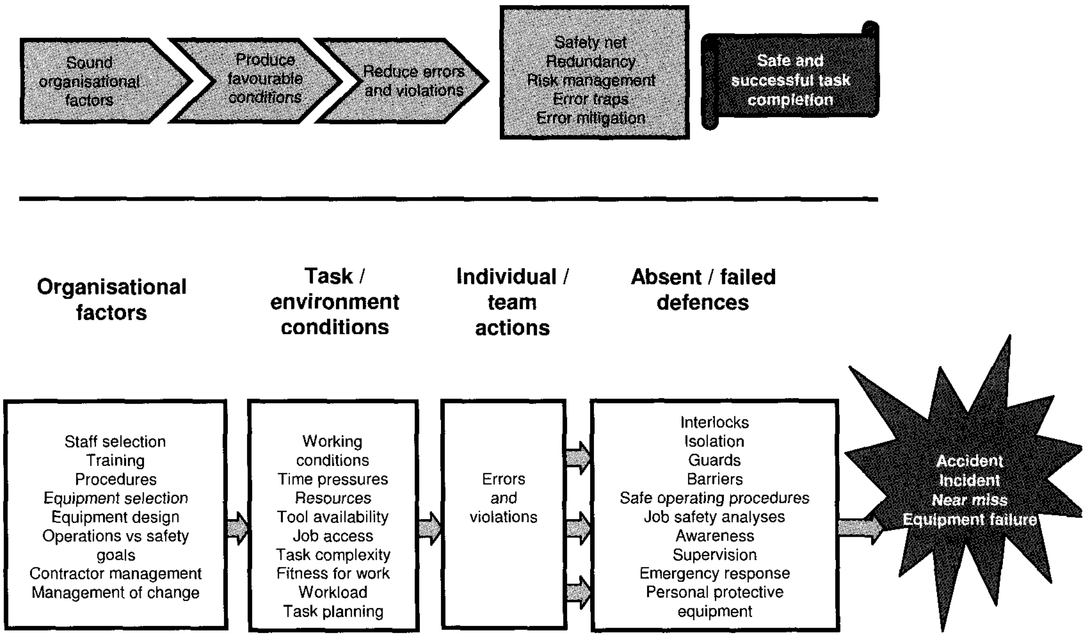

Fig. 2. The ICAM model of accident causation (Gibb *et al.*, 2004).

4.3. *System Safety Accident Investigation (SSAI)*

The System Safety Accident Investigation (SSAI) approach was developed in the 1970s for use in the US nuclear power industry. The techniques have since been modified for use in many high risk industries where fatalities or other catastrophes can occur. This brief account of SSAI is drawn from that given by Joy (2004) who has developed the approach particularly for application to the mining industry.

SSAI provides a systematic and logical process for fact finding in accident investigations and the drawing of conclusions. It imposes an overall discipline on the investigation process, providing a systematic method of identifying what happened and why it happened. SSAI uses several analytical techniques which are usually applied in a specific order:

- ***event and condition charting*** for displaying graphically the events in the accident sequence and the preconditions that affected those events,
- ***fault tree analysis*** for depicting the possible scenarios leading to an event in the accident sequence (where there were no witnesses),

- ***energy/barrier*** analysis to illustrate the unwanted energy flows and barrier inadequacies that contributed to the accident,
- ***human error analysis*** for the systematic examination of any deviations from expected human performance, and
- ***gap analysis*** to provide some insight into why the accident occurred by comparing accident-free conditions with the accident conditions.

Not all of these techniques are necessarily applied in any given accident investigation. They are tools to be used by an accident investigation facilitator. Typically, accidents resulting in or having the potential for loss of life, massive equipment damage, or prolonged system failure, receive a full and extensive investigation. The author has been part of such SSAI investigations in two cases involving fatalities in the Australian mining industry.

5. The Legal Interface

It is axiomatic to the concept and definition of forensic engineering being used here, that the forensic engineer's investigations have a relation to the legal system of the state or country in which the incident took place. The forensic engineer may be required to act as an expert witness in coronial enquiries, in civil or criminal court proceedings, in mediated or arbitrated disputes, or in tribunals. Even in legal cases that may be settled out of court, the forensic engineer will usually be required to prepare an expert witness statement in a prescribed format.

Most court systems issue guidelines for the preparation of expert witness statements, emphasising the need for the expert witness to be objective and impartial in giving opinion evidence (e.g. Department of Constitutional Affairs, UK, 2006; Federal Court of Australia, 2004). In the adversarial court systems in which the author has appeared as an expert witness, cross-examination by barristers can be a challenging, and sometimes harrowing, experience. The Federal Court of Australia's guidelines make provision for the Court to direct the experts retained by the parties to meet in an attempt to reach agreement about matters of expert opinion. Interestingly, the Civil Procedure Rules of the UK now give the Court the power to direct that evidence on a given issue be given by a single joint expert (Department of Constitutional Affairs, UK, 2006).

Professional engineering practice is governed by Codes of Ethics to which engineers subscribe on becoming members of professional engineering organisations. Ethical conduct is especially important in forensic engineering where the stakes can be high and there can be pressure on the forensic engineer to orient his/her investigation, findings and/or expert statement in the interests of one party or another. The *Guidelines for Forensic Engineering Practice* of the American Society of Civil Engineers (Lewis, 2003) define ethical forensic engineering practice as "*the conduct of forensic investigations and providing expert testimony based on sound, comprehensive, and unbiased investigation, and demonstrating exemplary professional conduct and honesty in serving the trier of fact, the public, and clients, as a qualified expert*".

A particular area of litigation often arising in underground construction concerns the consequences of what may be described variously as unforeseen, changed, differing or latent ground conditions. These are ground conditions encountered during construction that it is argued could not reasonably have been foreseen at the time of tendering. Modern risk management techniques and contractual practices (e.g. Eskesen *et al.*, 2004) seek to avoid or minimise litigation arising from this cause. However, the area remains one in which forensic geotechnical engineers often become seriously involved. Gould (1995) notes that, in the USA, progress in dispute resolution has brought with it the increasing involvement of geotechnical engineers in changed-condition claims. Gould (1995) further notes that *"in the present highly competitive heavy-construction market, bidders commonly accept hazards without contingency in their proposal, expecting that "claimsmanship" will turn a marginal job into a profitable one"*.

In some jurisdictions, proceedings in underground construction and mining cases may be taken by government agencies under occupational or work place health and safety legislation. In some states of Australia there appears to be an increasing trend to institute such proceedings against individual professional engineers and against operating companies, contractors and suppliers, for exposing employees to risk in the work place, both in the civil construction and mining industries. Despite the significant advances that have been made in the last decade or more in reducing geotechnically-related lost time injury and fatality rates in Australia's underground mines, Galvin (2005, 2006) has found that seemingly automatic prosecution policies (and some court decisions) are now impacting negatively on the objective of reaching the goal of zero harm because:

- *"Lessons from serious incidents are not being disseminated until some years after because of privilege and other considerations associated with pending charges.*
- *Some organizations and employers are reluctant to encourage near-miss reporting because of concerns that it could be used against them in prosecutions.*
- *It is a major disincentive for young people to seek a management career in the minerals industry."*

6. The Lane Cove Tunnel Project

6.1. *Background*

The Lane Cove Tunnel Project (LCTP) in Sydney, New South Wales, Australia, involves the construction of twin, 3.6 km long, two- and three-lane tunnels together with 3.5 km of bridge and road upgrades to link the M2 motorway at North Ryde with the Gore Hill Freeway, as well as a number of other elements. The twin east-bound and west-bound tunnels run beneath and slightly to the north of Epping Road. Figure 3 shows a schematic diagram of the tunnels in the project.

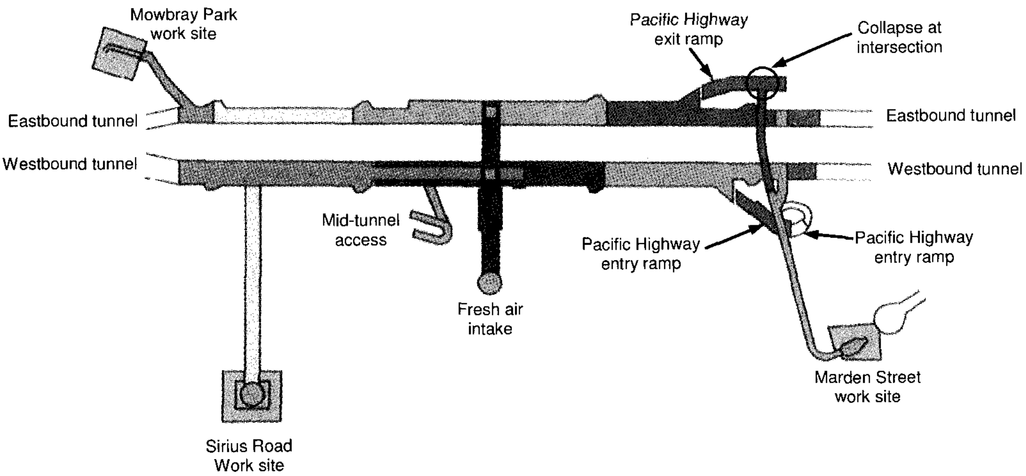

Fig. 3. Tunnel schematic, Lane Cove Tunnel Project (Rozek, 2005).

The New South Wales Roads and Traffic Authority (RTA) engaged the Lane Cove Tunnel Company (LCTC) to design, construct, maintain and operate the tunnel for a period of 33 years. LCTC, in turn, appointed the Thiess John Holland Joint Venture (TJH) to design and construct the project. TJH's contract commenced in December 2003 and tunnel construction in July 2004. The tunnel works are divided into three parts – a western or Mowbray Park section, a central section, and an eastern or Marden Street section. The contract completion date is May 2007.

The majority of the LCTP tunnelling is in the Hawkesbury Sandstone whose properties and engineering behaviour have been the subject of detailed study. Those sections of the tunnelling to be discussed here are excavated in the overlying and generally horizontally-bedded Ashfield Shale, described by Badelow *et al.* (2005) as a sequence of "*mudrocks including siltstone, mudstone or laminate and lesser shale and claystone*". In the process of the investigation, design and construction of a range of excavations and foundations in the Triassic rocks of the Sydney region, local classifications of the sandstones and shales have been developed and applied (e.g. Bertuzzi and Pells, 2002b; Pells *et al.*, 1998). Specific design methods for the excavations in the Sydney rocks and for their support and reinforcement have also been developed and applied with notable success (e.g. Bertuzzi, 2005; Bertuzzi and Pells, 2002a; Pells, 2002; Pells *et al.*, 1991). Advantage was taken of these methods and this experience in the design of the LCTP tunnels (Badelow *et al.*, 2005; Maconochie *et al.*, 2005; Rosek, 2005).

In the early hours of Wednesday, 2 November 2005, subsidence developed above the Pacific Highway Exit Ramp tunnel on design control line MCAA at its junction with the Marden Street ventilation tunnel on design control line MC5B. The road-header and loader working at the location were buried by collapsed material. The subsidence propagated to surface near 11-13 Longueville Road, Lane Cove, and an exit ramp from Longueville Road at the point marked in Figure 3. The subsidence under-mined the front of a building (producing a spectacular effect) and a bored pile retaining wall on the north side of Longueville Road shown in Figure 4. Within a few hours of the collapse, it was decided to fill the cavity with concrete as illustrated in Figure 4 in order to arrest the under-mining and underpin the retaining wall. The incident attracted considerable media attention in Sydney and more widely in Australia.

6.2. *The Investigation*

Two days after the incident, the author was engaged, through Golder Associates Pty Ltd, as an independent expert to investigate and report on the cause(s) of the subsidence. The author visited Sydney in the period 7-11 November 2006 when he carried out the following main activities:

- discussions and interviews with representatives of the contractor, the designer, the geotechnical consultant, and the crew and supervisors working at the collapse site at the time of the incident,
- a surface site visit,
- an underground visit to the site of the collapse and to inspect other tunnels in the Marden Street section of the project,
- study of a wide range of design documents, drawings, construction reports and other documents, and
- giving preliminary consideration to the possible causes of the incident and the preparation of an outline of the report.

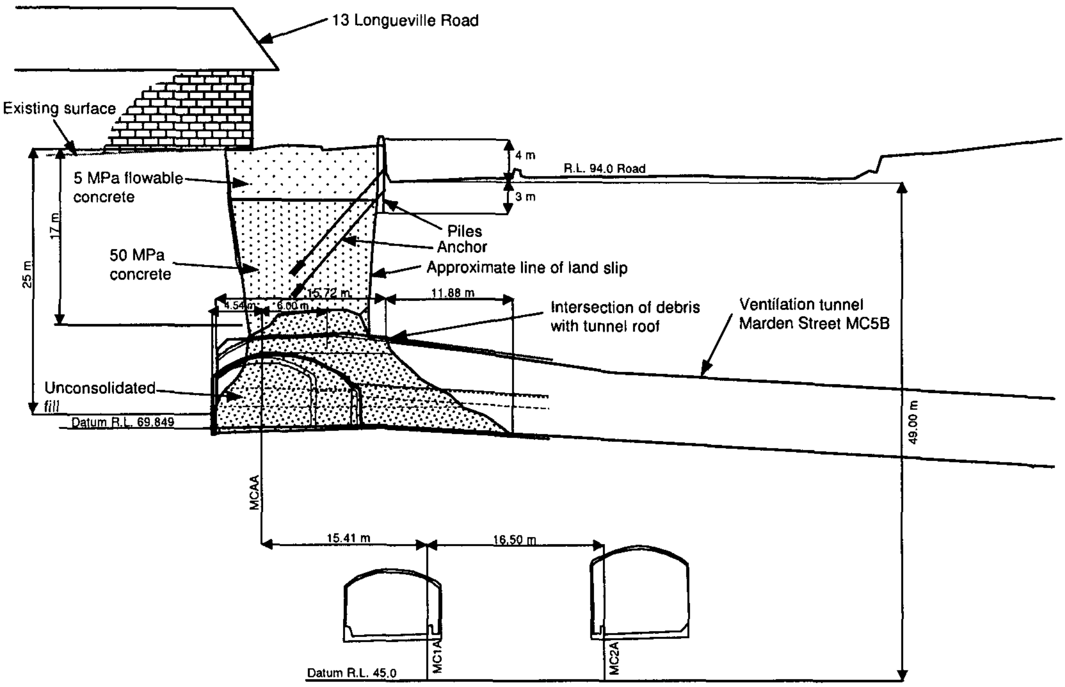

Fig. 4. Vertical section through the Lane Cove Tunnel incident site following the concrete pours.

Following this preliminary analysis of the available information, the following possible contributory factors to the collapse were identified:

- water,
- the weak shale in which the intersection was excavated,
- a low strength, steeply-dipping dolerite dyke crossing the intersection,
- jointing associated with the dyke,
- faulting present at the site,
- the large effective span of the intersection excavation,
- the low depth and nature of the cover,
- the levels of support provided, and
- the proximity of the Longueville Road retaining wall and ground anchors to the roof of the intersection excavation (see Figure 2).

The author then returned to Brisbane to continue his analysis of the information collected during the initial site visit and to prepare his report. During this period, several further documents and items of information were requested and supplied by the contractor and the geotechnical consultant. In the analysis of the incident, formal investigation techniques of the types outlined in Section 4 above were not utilised. However, the issues considered in the SSAI approach were addressed in the investigation, although not in a formal manner. A sequence of events was compiled in tabular rather than diagrammatic format.

The final report (Golder Associates, 2005) was released to the public and attracted some attention in the Sydney media. Other reports of the incident, generally based on the author's report, appeared in the technical press (e.g. Kitching, 2006). A few months later, the WorkCover Authority of New South Wales made public an initial report of its own investigation of the incident carried out pursuant to Section 88 of the NSW *Occupational Health and Safety Act 2000* (WorkCover NSW, 2006).

6.3. *Conclusions Drawn*

The report into the causes of the subsidence (Golder Associates, 2005) drew the following conclusions:

(i) Tunnelling and underground construction are always attended by a number of risks and uncertainties, mainly associated with the inherent variability of the geological structure and mechanical properties of the rock masses in which construction takes place.

(ii) During the excavation of the Marden Street ventilation tunnel, a near-vertical dolerite dyke (or pair of dykes), was intersected at a number of locations. Although dykes are known to exist in the sedimentary rocks of the Sydney region, this particular dyke had not been identified in the pre-construction geotechnical investigation. However, the dyke had been intersected previously during construction in ventilation tunnel MC5A and the main lane tunnels MC1A and MC2A.

(iii) Because of the presence of two sets of orthogonal joints associated with the dyke and other jointing and faulting, the shale rock mass at and near the junction of the MC5B ventilation tunnel and the MCAA exit ramp was of poorer quality than had been anticipated in the design stage.

(iv) The collapse started near the north-west corner of the newly extended down drive of the MCAA exit ramp at about 1:38 am on the morning of 2 November 2005, and rapidly extended across the exit ramp face to the dyke in the crown. The fall propagated across the crown of the junction of the MCAA exit ramp and the MC5B ventilation tunnel to the east or south-east and included the dyke. The collapse propagated to surface in Longueville Road in 10-20 minutes.

(v) The collapse initiated with the fall of rock blocks in the north-west corner of the excavation as a result of unravelling under a lack of lateral and normal restraint.

(vi) The ultimate failure mechanism was progressive, probably consisting of several stages (listed in the report).

(vii) The processes and methodology used in the design of the LCTP tunnels was in accord with best practice in Sydney and elsewhere, and the resulting designs were generally suitable for their purposes.

(viii) In the design stage, no special analysis of the MC5B/MCAA junction was carried out. However, because of the inevitable local variations in the geological and geotechnical conditions, it was recognised that it would be necessary to modify or adapt the initial design, particularly the support provisions, to the conditions actually encountered during construction.

(ix) TJH has in place a series of appropriate and best practice processes for the safe and productive execution of the underground construction works on the LCTP. Some of the documents setting out these processes are models of their kind.

(x) Up to the time of the incident of 2 November 2005, the designs and processes in place had been executed in a highly professional and productive manner by a knowledgeable and dedicated workforce and their supervisors.

(xi) The collapse arose from a combination of factors that were not present together at any other location in the underground works on the project.

(xii) The factors causing the collapse were probably:

- the presence of the dyke providing a persistent, relatively low strength, near vertical discontinuity transecting the roof of the excavation in a strike direction that was closely parallel to the maximum effective span of the junction,
- the presence of orthogonal, closely-spaced jointing associated with the dyke, reducing the already poor mechanical quality of the weathered Ashfield Shale rock mass,
- the presence of faults with orientations such that, in conjunction with the dyke, the joints and the excavations boundaries, they could isolate blocks that were free to fall or slide from the excavation boundaries if not adequately supported,
- a large effective span with relatively low cover to rock head, and
- the level of support existing in the western side of the excavation at the time being inadequate to ensure the excavation's stability given the large effective span, the low rock cover, the presence of persistent vertical discontinuity (the dyke) transecting the excavation, and the poor mechanical properties of the overlying rock mass.

(xiii) Water was not a cause of the collapse.

(xiv) The proximity of the Longueville Road retaining wall and its ground anchors to the crown of the MC5B/MCAA junction excavation may have contributed to the collapse by influencing the loads applied to the rock immediately above the excavation, or by weakening that rock mass, or both.

(xv) The preparation of the best possible longitudinal geological sections and/or progressive geological plans may have been of assistance in projecting conditions ahead of the face as excavation progressed.

7. Concluding Remarks

Forensic investigations of underground construction and mining failures and the associated interactions with the legal system, now represent an important part of the work of many experienced geotechnical engineers. The conduct of these investigations, the drawing of conclusions and the communication of the results impose significant demands on the knowledge and skills of the forensic geotechnical engineer and require the exercise of the highest professional and ethical standards. Forensic geotechnical engineers can play a significant role by explaining publicly and within the legal system, the not widely understood difficulties and risks involved in engineering in the variable natural materials found on and in the crust of the Earth.

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THERMO-MECHANICAL BEHAVIOR OF ROCK MASSES AROUND UNDERGROUND LNG STORAGE CAVERN

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Thermo-mechanical behaviors of rock masses around underground LNG storage cavern are evaluated by analyzing in-situ rock mass responses measured at the Daejeon pilot cavern and a 2-dimensional numerical model of a lined LNG rock pilot cavern with UDEC and PFC2D, with aims to estimate the temperature profile in the containment system including the surrounding rock mass and to study the mechanical response of the rock mass under a thermal loading. The in-situ measured rock mass responses from the operation of the LNG pilot cavern are found in good agreement with the results of the numerical analyses. Overall monitored results from the pilot experiences confirm that construction and operation of underground LNG storage in lined rock caverns are technically feasible from rock mechanical point of view. The Daejeon LNG storage pilot cavern represents a further important step in the validation of technology of lined LNG underground storage. Even though the cavern was small, this project will permit to draw the potential and critical parameters for construction, start up and operation of the new technology in a real scale.

Keywords: Thermo-mechanical behavior; numerical modeling; LNG storage cavern; in-situ rock mass response.

1. Introductions

Some attempts were made in the past to store LNG (Boiling Temperature: -162°C) underground in unlined containments, but were not successful. One of the important problems related to underground LNG storage is the leakage of liquid and gas from the containment system to the rock mass causing tensile stresses due to shrinkage of the rock mass around the caverns (Inada, 1993, Monsen & Barton, 2001). If the storage is unlined and frozen down to -162°C, the rock is cooled and the rock joints start to reopen, a part of the gas flows into the joints and continues cooling inside the rock wall. This opens the joints successively and heavily increases the cooled area and the extent of the cooling front.

To provide a safe and cost-effective solution, a new idea of storing LNG in a lined hard rock cavern has been developed and tested for several years (Amantini & Chanfreau, 2004). This concept consists of protecting the host rock against the cryogenic temperature by using a containment system with a gas tight steel liner and insulation panels.

The pilot project of the lined cavern LNG storage has been carried out in an existing research cavern within KIGAM (Korea Institute of Geoscience and Mineral Resources), Daejeon, Korea. The objectives of this project are to demonstrate the feasibility of the lined rock cavern concept, to examine the overall performance of the storage and to have efficient returns of experience to improve construction and design of industrial scale projects. For safety and practical reasons, liquid nitrogen (LN2, T=-196°C) is used instead of LNG.

In this paper, thermo-mechanical behaviors of rock masses around underground LNG storage cavern are evaluated by analyzing in-situ rock mass responses measured at the Daejeon pilot cavern and a 2-dimensional numerical model of a lined LNG rock pilot cavern is constructed and analyzed in UDEC, which aims to estimate the temperature profile in the containment system including the rock mass, and to study the mechanical response of the rock mass under thermal loading.

2. Geotechnical Monitoring

The roof of the pilot cavern lies at a depth of about 20m below the ground. The inner section of the containment system has a dimension of 3.5 x 3.5m, and a length of 10m. Thickness of the PU insulation panel is 10cm and that of the reinforced concrete lining is 20cm, installed between the rock wall and the containment system. A comprehensive monitoring system was installed to measure temperature, thermo-mechanical responses of the rock and concrete during the implementation of the LNG storage (Lee *et al.*, 2005).

2.1. *Geotechnical monitoring system*

The pilot cavern and its surrounding rock mass are equipped with a comprehensive set of geotechnical instruments to monitor temperature profiles and temperature-induced displacements in the rock around the cavern, reopening of rock joints on cavern surface, load on rock bolts installed, settlement of the ground surface, pore pressure distribution in the rock mass and the variation of the ground water level. All the instruments are equipped with thermal sensors to measure the temperature at the depth they were installed.

Variation of the temperature and displacement were monitored from 6 borehole extensometers with 6 points each. After several commissioning tests, operation started from January 10, 2004 with 2.6m head of LN2 and the drainage system was stopped gradually from June 10, 2004 to July 8, 2004. The filling pipeline was closed on July 10th, and the storage has been fully emptied since August 12, 2004.

Table 1 shows all the geotechnical instruments installed for the pilot cavern, and Figure 1 represents the general arrangement of the instruments in a cross section and a plan view.

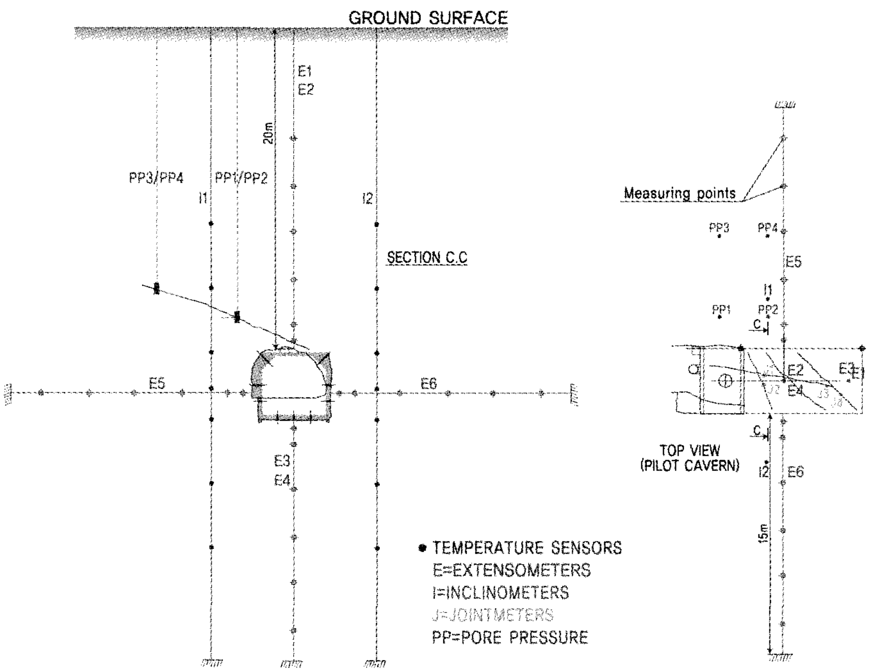


Fig. 1. General arrangement of geotechnical instrument in Daejeon LNG Pilot.

Table 1. The list of geotechnical instruments installed for the pilot.

	Item	Instrument	Model	Quantity	Description
CAVERN	Rock displacement	Multipoint Borehole Extensometer	DISTOFOR	2holes × 6points 2holes × 6points	Vert.(15m) Horiz.(15m)
	Joint behavior	Vibrating Wire Jointmeter	JM-S	5EA	Joint
	Rock bolt axial force	Instrumented Rock Bolt	IRB-750	4EA × 2.5m	Roof
	Concrete stress	Embedded Strain Gage	EM-5	6EA	Con'c
SURFACE	Rock displacement	Multipoint Borehole Extensometer	DISTOFOR	1holes × 6points	Vert.(17.2 & 18.5m)
	Rock horizontal displacement	Digitilt Inclinometer Probe	-	2holes (40m/hole)	Manual reading
	Temperature	Resistance Temperature Detector	SS-5039GK	2holes × 7points	On casing of Inclinometers
	Pore pressure	Vibrating Wire Piezometer	PWS	4holes	vertical

2.2. *Monitoring of thermo-mechanical behavior of rock masses*

2.2.1. *Temperature*

Figure 2 shows temperature profiles around the cavern with all the monitored data at several stages of operation. The stored LN2 level increased very quickly at the initial stage and then maintained at 2.6 m until filling is stopped. Temperature inside the portion of the storage submerged with LN2 was kept constant at -196°C, and temperature on the upper part of the storage tank, filled with the boiled off gas, was about -100 ~ -120°C.

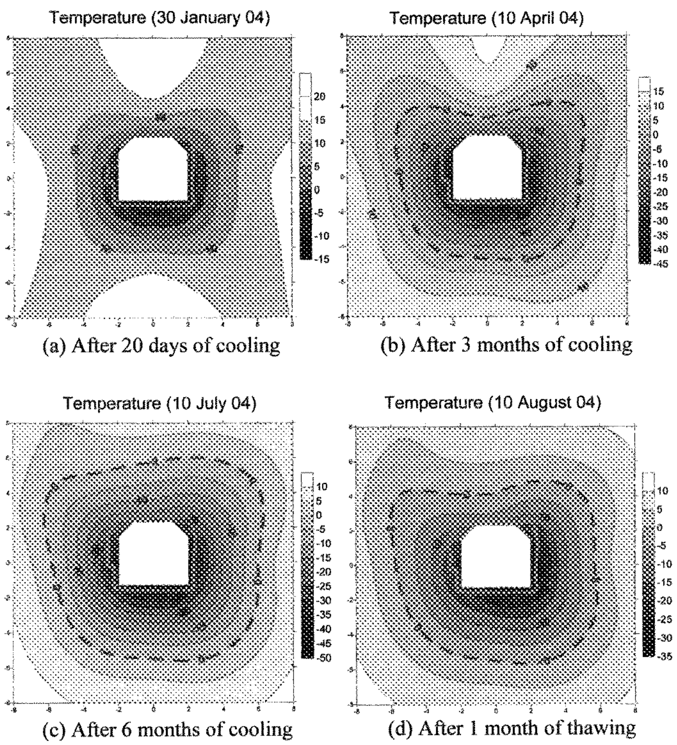


Fig. 2. Temperature profiles around the cavern. Dotted line indicates 0° isotherm.

It can be clearly seen that the cold front propagates through the rock mass. Cooling rate of the rock above the roof of the cavern was slower than that of the sides due to the gaseous space from the boiled off gas. After 6 months of cooling (before LN2 stoppage), the 0°C isotherm propagated up to 4.4m from the cavern floor, 4.0 m from the sidewall and 3.2 m from the roof (Fig. 2c). The 0°C isotherm maintained a similar profile, even after one month of thawing, except above the cavern roof (Fig. 2d). It can be postulated that an ice ring was formed around the cavern wall, which blocked the groundwater movement during this period.

Figure 3 shows the temperature evolution on the rock surface from jointmeters. The initial quick decline of the temperature is stabilized in an asymptotic manner as the time elapsed, and the heat transfer reached steady state. The peak lowest temperature of the cooled rock surface was about -27~-35°C. Compared to the cooling stage, the temperature rose quickly in a linear manner during the thawing stage.

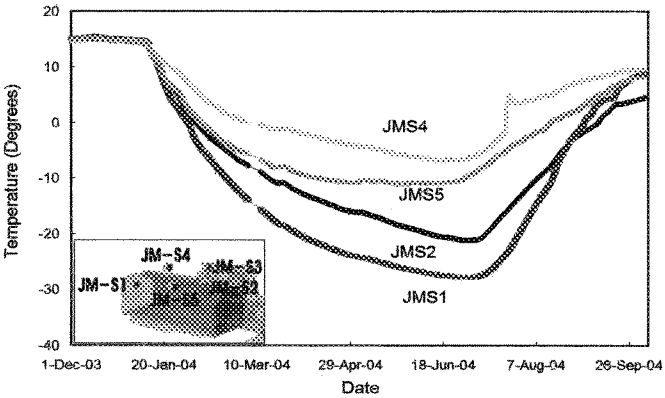
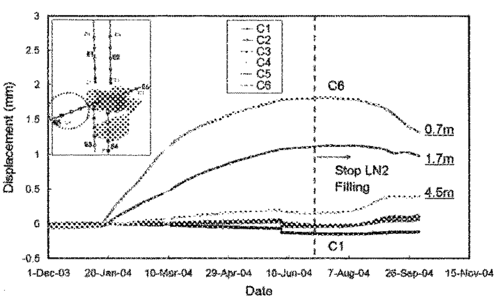


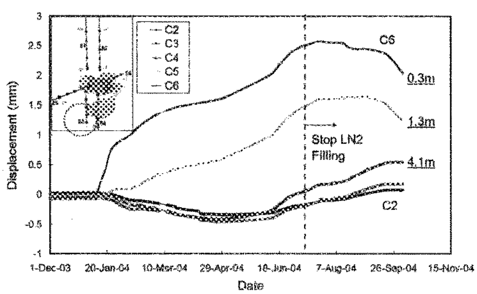
Fig. 3. Temperature history on the rock surface.

2.2.2. *Displacements*

Typical displacements of the rock mass at various points of the extensometers, on the sidewall and the floor, are shown in Figure 4. Positive sign indicates displacement occurring towards the inner rock mass. All displacements occur towards rock mass (positive) due to contraction near the cavern walls, but no significant changes occurred at 4.5 m from the cavern wall. Since all the excavation-induced deformations occur toward the cavern, such thermo-induced deformation will play a positive role in cavern stability. Displacements decreased in a linear manner according to



(a) From sidewall E5



(b) From floor E3

Fig. 4. Displacement changes at various locations from rock surfaces

the depth of rock from the storage wall. From all monitored results, the maximum displacement at the cavern wall was 4~5mm (0.2% of the cavern radius). During the thawing process, the rock displacements do not respond as quickly as compared to the temperature changes.

2.2.3. *Joint separation*

Figure 5 shows a typical joint deformation during cooling and thawing stages. Joint apertures were opened in a linear manner by cooling, though their magnitudes are small; and they were closed as temperature was raised. It is interesting to see that a clear nonlinear mismatch from the overall deformation path is shown near 0°C for both cooling and thawing stages. A reason can be postulated that residual water inside the joints results in locally uneven deformations due to ice formation near 0°C, inducing a temporal unusual behavior.

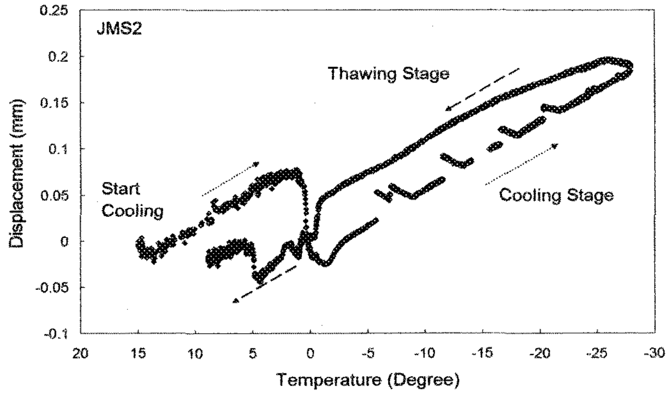


Fig. 5. Hysteresis of joint separation with temperature on the excavation surface during cooling and thawing stages.

2.2.4. *Rock bolt load*

A history of the rockbolt load is shown in Figure 6. Tensile force gradually increases as rock mass is cooled down, but the load disappears quickly during the thawing stage. Tensile force was induced when the cavern wall expanded toward the rock mass as shown in Figure 4. The magnitude of the axial tensile force reached up to 1.3 ton during the cooling stage, which is about one tenth of the pullout load. The rockbolts maintained enough support capability though the temperature on the rock surface was fallen down to -30°C.

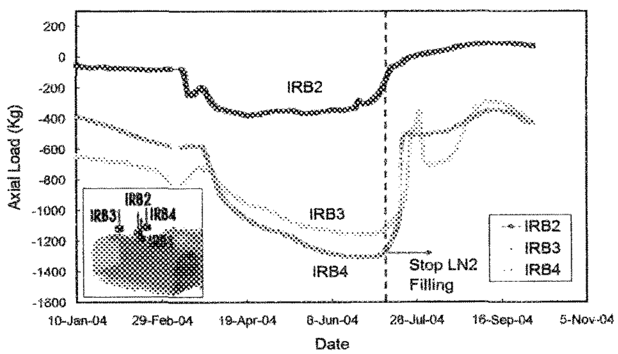


Fig. 6. Change of rockbolt loads during the operation.

2.2.5. *Pore pressure*

The detailed drainage procedure and the resulting hydro-geological phenomena are presented in Lee *et al.* (2003b). In this paper, only the pore pressure changes near the cavern walls are discussed. Typical pore pressure evolutions are shown in Figure 7 by converting pressure to head. Heads of all measured points were kept at 0 m or slightly negative values until drainage stopped. This means that the rock mass near cavern area remained dry during this stage, and drainage efficiency could be confirmed by measuring the pore pressure around the cavern.

After the drainage system stopped on June 10, 2004, water level rose quickly up to 6 m from the cavern roof due to infiltration from surface by rainfall. The pore pressure decreased for one and half months after drainage restarted. During this period, the ice ring could be formed around the cavern wall. Consequently, no artificial water injection was required to resaturate the rock mass for the ice-ring formation because of the natural recharge from rainfall.

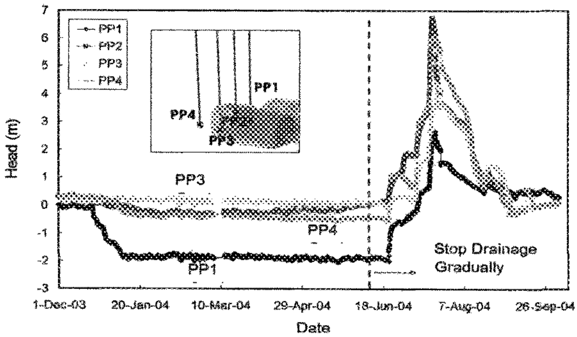


Fig. 7. Pore pressure evolution of rock around the pilot cavern

3. Thermo-Mechanical Modeling

3.1. *Laboratory tests*

Rock type around the cavern is biotite granite. Thermal and mechanical properties of rock, concrete and PU foam are given in table 1. Thermal expansion coefficient of rock is measured at zero degrees Celsius for a core specimen, while the thermal conductivity and the specific heat are obtained by laboratory tests for a thin section. Strength parameters such as friction angle and cohesion are determined by the empirical estimation from Bieniawski's RMR in accordance with the procedure proposed by Serafim and Pereira (1983). Properties of concrete are taken from the standard data for typical un-reinforced concrete in Korea.

Table 1. Thermal and mechanical properties of rock around the pilot cavern.

Materials	Thermal			Mechanical						
	Thermal expansion coefficient (10^{-6}/℃)	Thermal Conductivity (W/m/℃)	Specific Heat (J/Kg/℃)	Density (kg/m^3)	Elastic modulus (GPa)	Poisson ratio	Cohesion (MPa)	Friction angle (deg.)	Tensile strength (MPa)	Dilatancy (deg.)
Rock	6.64	2.63	710	2660	20.3	0.28	0.31	26	8.0	26
Concrete	6.64	2.63	710	2550	23.0	0.25	8.0	30	2.4	30
PU foam	-	0.02	1674	65	0.023	0.20	10000	70	10000	0

3.2. *Concept of analysis*

Combined thermo-mechanical numerical models are adapted and used for investigating relevant mechanisms such as propagation of the cold front, migration of water and ice formation in the host rock mass, and the induced thermo-mechanical stress. Thermal and thermo-mechanical analyses are performed with the aim to estimate temperature profile in the containment structure including the rock mass, and to study the mechanical response of the rock mass under thermal loading as well as the behavior of a rock joint crossing the storage cavity.

The temperature of the cavern wall is decreased in several steps. After the final step, the cooling is stopped, which means that the heat flux through the cavern wall is zero from that time. All the mathematical formulations have been implemented in a computer program.

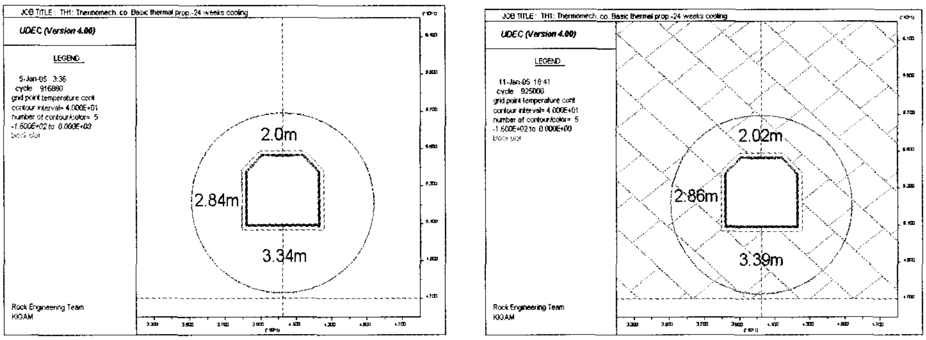
For the thermal analysis, LN2 temperature to be stored at atmospheric pressure (T= -196℃) is set instantaneously to the intrados of the insulation layer. For the thermo-mechanical analysis, numerical model runs were performed after stress initialization and cavern excavation stages to reach the mechanical equilibrium under the uniform and constant temperature. LN2 temperature is set instantaneously to the intrados of the PU foam.

3.3. *Thermal analysis results*

Figure 8 shows the 0°C isothermal profiles around the cavern after 24 weeks of cooling. Same thermal properties are used in each model. As shown in the figure, there is no conspicuous difference with the presence of joints. Cooling rate of the rock below the floor was slower than that of the sides and the roof due to the intervening gaseous space.

From all modeled results of UDEC analyses, the 0°C isotherm propagated up to 2.0~2.9m from the cavern roof, 2.84~3.64m from the sidewall and 3.34~4.3m from the floor. It can be seen that the extent of 0°C isotherm is increased with decreasing thermal properties.

The maximum temperature at rock concrete contact is equal to –27℃ for all the cases. It is likely that the results in all analyses show the same tendency. It should be noted that the maximum temperature at PU foam-concrete contact has been dramatically lowered to about –59℃ after 24 weeks of cooling in the front face of the containment of which concrete is also in outer surface exposed to room air. This part seems to be very sensitive to thermal loading and the mechanical behavior of the concrete. To protect any possible cracking due to too low temperature, it is necessary to utilize a ventilation system for providing room with climate air.



(a) no joint model (Case 1)

(b) two joints set model (Case 2)

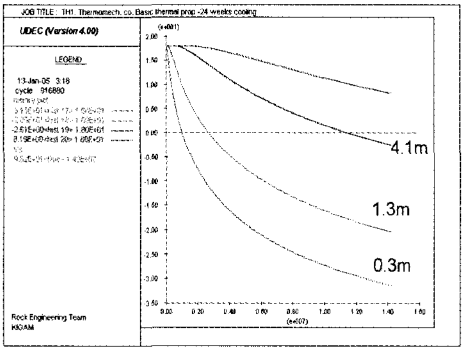
Fig. 8. Contour of Isotherm 0℃ after 24 weeks cooling.

3.4. *Thermo-mechanical analysis results*

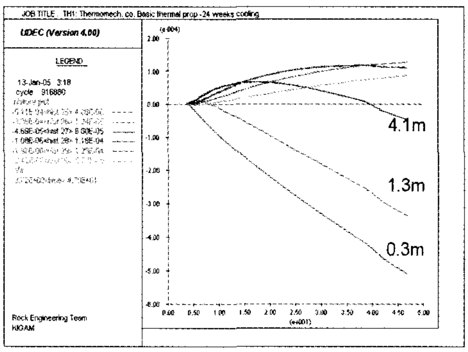
3.4.1. *UDEC modeling*

Figure 9 shows the results of thermo-mechanical analysis after 24 weeks of cooling. Temperature and displacement of the rock mass at various points on the floor are shown in the figure. Initial rapid decline of the temperature has been stabilized in an asymptotic manner as time elapsed (Figure 9(a)). The lowest temperature of rock mass was about -32°C at 0.3m from the cavern floor. It can be inferred that thermal responses of the rock mass around the cryogenic storage cavern could be well predicted by numerical models. Positive sign in Figure 9 (b) indicates displacements occurring toward the storage cavern.

Also Figure 10 shows that while the direction of displacements toward the rock mass (positive) due to contraction near the cavern wall, no significant change occurred at a location over 4.5m from the cavern wall. Since excavation-induced deformation occur toward the cavern, such deformation aspect will play a positive role in cavern stability. Displacements decreased as the depth of rock increased from the cavern wall.



(a) temperature



(b) displacement

Fig. 9. Temperature and displacement changes at various locations from cavern floor.

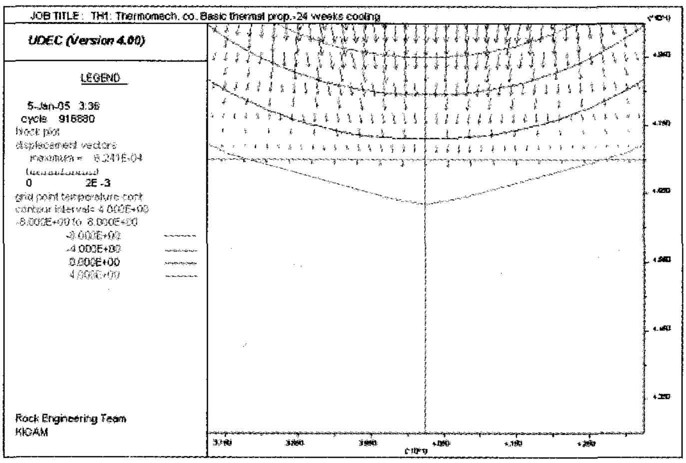


Fig. 10. Temperature contour and displacement vector around the cavern floor.

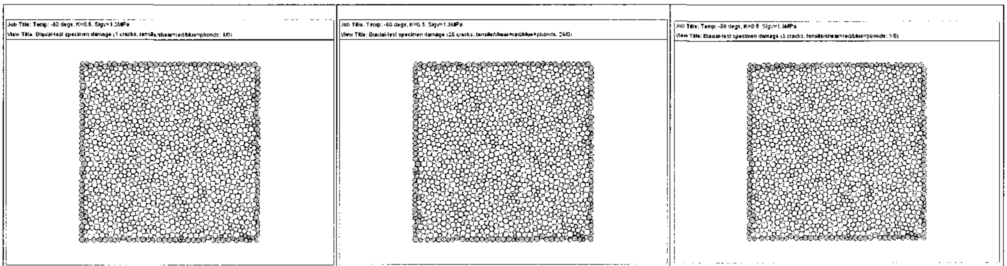
3.4.2. *PFC 2D modeling*

It has been known that when rock is cooled slowly, thermal cracks due to difference of thermal expansion between components of rock are generated although a temperature gradient is not steep(Lee, 1993). From the results of AE experiments on granite by Shell Inc., generation of micro crack was started at about -50℃ (Dahlström, 1992). Therefore a micro crack could be generated when a thermal stress exceeds tensile strength of a rock.

However because of an effect of compensating tensile stresses with thermal stresses when initial stresses (compression) are present, crack generations due to thermal shock would be suppressed. The criterion of LNG storage suggested by Goodall et al (1989) is given as in Eq. (1).

$$\text{In-situ stress} + \text{tensile strength} > \text{thermal stress} \tag{1}$$

Thermo-mechanical analyses are performed by a fracture mechanical approach with a greatly simplified model using PFC 2D. Figure 11 shows the PFC2D model to figure out the fracture mechanism of crack expansion. As shown in Figure 11, fractures are formed and expanded at a lower temperature as a pre-crack exists in a model or its length gets longer. It would be thought that as mechanical properties of cracks is much smaller than those of the surrounding rock, a pre-crack becomes wider easily due to displacements occurred by cooling-down of rock mass, an induced stress is concentrated at the tip of a pre-crack and it causes the crack to expand gradually.



(a) No pre-crack (-80□) | (b) crack length: 0.5m (-60□) | (c) crack length: 1.0m (-50□)

Fig. 11. Fracture patterns according to a pre-crack length (K=0.5, vertical stress=1.3MPa).

Based on the design concept of underground LNG storage, the minimum temperature of the surrounding rock mass is not to drop below -50℃ after 30 years of LNG storage operation. According to this criterion, if a maximum temperature drop is assumed to be -70℃ (initial temperature of rock mass: 20℃), thermal stress in the model is calculated as a tensile stress equivalent to 6.4MPa with 2.62×10^{-6}/℃ of the thermal expansion coefficient, 13.3MPa with 5.78×10^{-6}/℃ of the thermal expansion coefficient. As the temperature drop increases, induced thermal stresses also become larger before fractures are generated. But when the magnitude of induced stresses is decreased steeply below -70℃, fracture initiates and the fractures are distributed throughout the model.

4. Conclusions

In this paper, thermo-mechanical analyses have been performed in two different ways, by the analysis of in-situ rock mass responses measured at the Daejeon pilot cavern and by 2-dimensional numerical modeling to a lined LNG rock pilot cavern with UDEC and PFC 2D. From the results of these analyses, the following conclusions are obtained.

(i) Thermal responses of the rock mass under a very low temperature of about -30°C around the LNG storing lined rock cavern could be well predicted by numerical models due to the absence of water.
(ii) Thermal stress-induced displacements occurred toward the rock masses, which are favorable to the stability aspect of a cavern. Joint separation was small compared to rock displacements, and a linear increase in deformation was observed except near 0°C.
(iii) In-situ thermal expansion of rock was formed 5-11 times larger as compared to the laboratory obtained data, and showed typical nonlinear hysteresis during cooling and thawing stages.
(iv) There was no thermal shock by abrupt cooling near the cavern wall due to insulation barriers.

Cracks could be generated when the thermal stress exceeds the tensile strength of a rock. So the minimum temperature of the surrounding rock mass is not to drop below -50℃ after 30 years of LNG storage operation based on the design concept of the underground LNG storage cavern.

(v) Overall monitored results from the pilot experiences confirm that construction and operation of the underground LNG storage in lined rock caverns are technically feasible from the rock mechanical point of view.

The Daejeon LNG storage pilot cavern represents a further important step in the validation of the technology of lined LNG underground storage. Even though the cavern was small, this project will permit to address the potential and critical points for construction, start up and operation of such a new technology. Besides, the study of different parameters, those were recorded during the operation of the pilot cavern has increased the knowledge of the partners in the behavior of the containment system and its surrounding rock in order to tackle an industrial project in the near future.

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ROCK MECHANICS AND HAZARD CONTROL IN DEEP MINING ENGINEERING IN CHINA

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Recent advances in test and design and computational frameworks of the rock mechanics in mining engineering at great depth are reviewed. It is shown that the main differences for the characteristics of rock mass for deep mining compared with shallow mining are as follows: (1) mining environments, High ground stress, High earth temperature, High karst water pressure, and mining Disturbance (HHHD), (2) five transform features of mechanical characters, (3) four changes of coal mine types and, (4) six typical engineering hazards. With the detailed researches on the nonlinear mechanical characteristics of rock mass in deep mining engineering under complicated geological and mechanical environments, it is shown that the mechanical system is nonlinear for rock mass at depth, but is linear for that at shallow. The classical theory, methods and technology may be partly or most entirely invalid. Accordingly, it is very important to study the basic theory of rock mechanics in deep mining engineering.

Keywords: Deep mining; rock mechanics; HHHD; hazard control.

1. Introduction

The major concern in deep mining engineering is that rock mechanics problems are associated with laneway deformation or stresses of mining structure caused by excavation at great deep. The increased resource consumption and large-scale coal mining at shallow depth makes shallow energy sources be inevitably less and less. Thus the mines are being excavated at greater depth. A challenging issue that is often met in deep mining engineering and design is how to avoid or prevent the occurrence of catastrophic events such as blast-pressure from geological structure field, gas blast, suddenly amplified ground stress, large deformation, rock rheological phenomenon and high underground temperature. Consequently, a comprehensive investigation of deep mining mechanics are of interest on in-situ monitoring, mechanical performance and deformation characteristics, strength, and post-failure and failure mechanism for rock mass which is embedded at deep level. (Kidybinski, 1990; Sellers and Klerck, 2000; and Qian, 2004)

2. Present State of Deep Mining Engineering

From the incomplete statistics, there are more than 80 ore mines at the depth of over 1000m below ground outside China, among which mostly locate golden mines excavated into over 1000m in South Africa, e.g., Anglogold company's western golden mine at the depth of 3700m. Another example is West Driefovten golden mine where plenty of golden ores extend over between 600 and 6000m below ground. In India, there are 3 golden mines with the mining depth of over 2400m, and a total number of 112 stages were excavated from ground surface to 3260m under ground at Championliff. In Russia, there have been 8 iron ore mines with the normal digging depth of 910m and exploiting at the depth of 1570m and even extending to the depth of 2000-2500m in the future at Griverog. Moreover, there are some of nonferrous metal mines also mined at depth of over 1000m in Canada, the United States, and Australia. Most of the countries mined coal started to mine into deep well since the 1960s, e.g., average digging depth had been to 650m before 1960, nearly to 900m in 1987, in Western Germany. Half of coal products in the 80s of last century were produced 600m below ground in the USSR. A summary is shown in Fig. 1 for the trend in the world for the mining depth from 1980 to 2020.

On the other hand, the average speed of incremental mining depth for the major coal mines in China is about 8-12m every year. In east China, the incremental speed of coal well depth has been to 250m every 10-year (He, 2004; Yan, 1996; and Paterson, 1958). Some mines in China is being excavated under deep mining, e.g., for the coal mining, the mining depths have been to 1197m, 1159m, 1100m, 1059m, 1055m, 1008m, and 1000m, at Caitun in Shenyang, Zhaogezhuang at Kailuan, Zhangxiaolou in Xuzhou, Guanshan at Beipiao, Sun village at Xinwen, Mentougou and Changguang in Beijing, respectively; for the metal mining, the mining depth is being between 900 and 1000m at Hongtaoshan copper mine, and 2 vertical wells with the depth of over 1000m have been built at Tongguashan copper mine, and an iron ore mine horizontally extended to 750m and vertically digged into the depth of 1000m was designed at Gongchangling, the golden ores distributed from the pit mouth to the depth of 1050m at Jiapigou, and 38 stages were excavated from the vertical depth of over 850m. Furthermore, there will be more mines such as Shouwangfen copper mine, Fankou pludbum mine, Jinshan nickle mine, and Rushan golden mine to be excavated soon. At the same time, metal and nonferrous mines in China will be excavated from the depth of 1000 to 2000m below ground level. To compare with the deep mining engineering of other countries, a statistical figure of development trend for average mining depth of major coal mines in China is presented in Fig. 2.

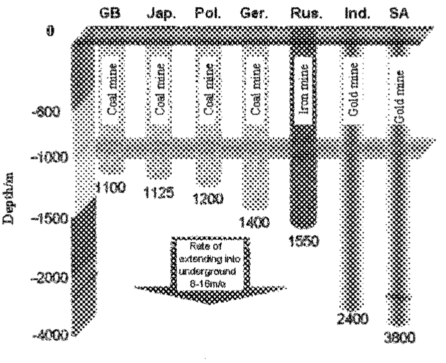

Fig. 1. Present situation of deep mining engineering in the world.

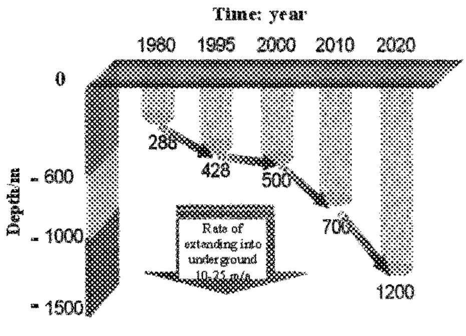

Fig. 2. Developing trends of average mining depth of major coal mines in China.

3. The State-of-the-Art of Researches on Deep Mining

In the early 80-year of last century, some experts proposed to investigate the deep mines where their deformation and stress are of rather complicated and significantly different from the shallow ones. In 1983, one scholar from the USSR suggested that coal mine depth over 1600m, in particular, should be specifically studied. At that time, a huge test-bed that was built in Western Germany was especially utilized to simulate three-dimensional mine-pressure in mine well at great depth. In 1989, a special conference on 'Rock Mechanics in Deep Mines' sponsored by International Society of Rock Mechanics was held in France and its corresponding papers compiled together were also published. In the recent two decades, comprehensive investigations have been conducted, including rock burst forecast, large deformation mechanisms for soft rock, water gust in tunnel and rock burst prevention and cure such as innovating mechanical properties of rocks around tunnel, stress relaxing, anchoring in time, and reasonable construction technique, as well as weak rock prevention and cure like stabilized palm surface, strengthening toe and preventing surface crushing, preventing cracked anchor and lining and shoring, and separately excavated section. Industrial corporations and research institutes cooperate with the government together so as to focus on the study of fundamental mining theory and technical problems in deep mining engineering, in the countries such as the United States, Canada, Australia, and South Africa, where there are some mines with deep mining engineering. A research plan on 'deep mine' that costs 138 million US dollars, by the government together with the universities and the corporations, has been conducted since July 1998, to study safety and economics issue in deep golden mine in South Africa. In Canada, a comprehensive investigation similar to the aforementioned plan of South Africa, which lasted for 10 years and dealt with deep wells, was also performed, including statistical forecast to rock burst and support system of latent softening zone, by the federal and the province and the mining corporations. Furthermore, a similar investigation concerned with deep mine was completed by two universities and one research institute in the USA, and the latter extended to including the identification and difference of earthquake information induced by rock burst from natural earthquake or chemical and nuclear explosion signal sponsored by the Department of Defense. A significant advance on deep mine was also achieved by the University of West Australia.

With the development of national economy, and science and technology in China, the construction requirements of railway and road tunnel under the condition of complex geological environment and the prevention and cure of disaster events due to deep mining motivate to development and improvement of theoretical models and technical methods which have been used to a lot of geotechnical and mine engineering. A number of investigations and practices that involve with design of soft rock support, rock burst prevention, exploration and forecast beyond construction, and construction process using information system, have been conducted by the universities and the research institute in China, e.g., China University of Mining and Technology (CUMT), Central South Uiversity, Northeast University, Chongqing University, Tongji University, and WJTU. For example, predict, forecast, prevention and cure in deep coal mining engineering by CUMT, tunnel optimization and its stability by WHGM, rock burst mechanism in deep mine well over 1000m below ground and its control technique by MSU, and earthquake induced by mining at Laohutai mine in Fushun, all together have merit in the design of mining engineering against hazard control. In this paper, the main issues of rock mechanics and design in deep mining engineering that are the major concerns of researchers and mine engineers will be discussed.

3.1. *Deformation mechanism of rock mass at deep depth*

(i) Brittle-ductile transition of rock mass mechanism in deep strata:

Performances of post-peak strength are varied under different confining pressures of rock mass. A salient characteristics of rock mass is that its performance strongly relies on the variation of confining pressure, i.e., brittle cracking and ductile failure corresponding to lower and higher confining pressure, respectively. Since the pioneering work on experimental tests of granite under the different confining pressures by von Karman (1911), a number of experimental studies on rock mechanics have contributed to the understanding of the effect confining pressure on the mechanical behavior of rock samples. It is also demonstrated using marble tests under the condition of room temperature by Paterson (1958) that one of the fundamental characteristics for marble is also consistent of the brittle-ductile (BD) transition with the increase of confining pressure. The similar phenomenon as the aforementioned was also observed through the series tests by Mogi (1965; 1966) who pointed out the transferable property of rock being in general dependent on the rock strength. However, such conclusion of BD transition associated with rock strength was also obtained by Paterson, but both marble and granite still behave their brittle characteristics even under the confining pressure of 1000 MPa or more (Paterson, 1978), whereas some in-situ observations show that such great hard granitic diorite under the action of geological stress field is capable of enduring large deformation like ductile material.

Different strain values correspond with the rock failure phases under the actions of different confining pressure. However, it is commonly without or with small permanent (or plastic) deformation accompanied with the brittle failure of rock, whereas larger permanent deformation often accompanied with the ductile failure of rock. Therefore, both Heard and Singh referred the strain value of rock at failure as a threshold between brittle and ductile (Heard, 1960; Singh et al., 1989).

After analyzing of the experimental test data that consists of 101 groups of sandstone from Asia, Europe, America, and Africa, Kwasniewski in details studied the rule of BD transition and consequently presented a threshold criterion of BD transition for rock, as well as systematically observed transitional behavior of the sandstone during the BD transition (Kwasniewski, 1989). Chen and Huang also pointed out from the viewpoint of rock physics (Chen and Huang, 2001) that the rock from the transitional state of BD usually behaves like inevitable twofold, i.e., brittle and ductile.

A lot of viewpoints that are from the lithosphere dynamics about the threshold criterion of BD transition of rock have been widely accepted. It is believed that BD transition can occur with the increase of depth while the values of confining pressure and temperature of rock are at certain levels. However, transferable depth associated with rock properties certainly exists. Meissner and Ranalli believed that rock deformation should be at the ductile state if friction strength is equal to creep strength of rock (Meissner and Kusznir, 1987; Ranalli and Murphy, 1987). Shimada presented various predictive curves from fracture strength of rocks among the lithosphere zone to determine the strength of rock in strata (Shimada, 1993). Sibson ever found that high stress level at the range of threshold depth in the upper crust can be relaxed a lot in the process of BD transition (Sibson, 1977; Sibson, 1980).

In a word, brittle-ductile transition is a special characteristic of rock deformation under high temperature and high confining pressure. If the failure occurs with little or no permanent deformation for shallow rock mass that is under low pressure, the failure in deep rock mass frequently accompanies large plastic deformation at high confining pressure. Current issues of BD transition for rock are mostly focused on judge rule of BD transition. Less work has dealt with the physical mechanism of BD transition, so that there is no acceptable theory or model which can be applied to explain that mechanism of BD transition associated with intrinsical properties of rock.

(ii) Rheological characteristics of rock in deep strata

Time effect that evidently behaves like rheology or creep is significant to rock mass at high confining pressure in deep strata. Stability and creep effect of confined rock mass surrounding to nuclear waste repository were analyzed during the investigation of nuclear waste disposal (Blacic, 1981; Pusch, 1993). It is generally believed that larger creep is not possible to intact hard rock. However, it was shown in the South Africa that time-dependent behavior of deep level tabular excavations in hard rock is also strong (Malan, 1999; Malan, 2002). A very simple factor of bearing that is a ratio of rock mass strength with respect to field stress was proposed by Muirwood, which was used to embody the rheological characteristics of wall rock in tunnel (Muirwood, 1972). Then, such applicable range of the factor was discussed by Barla (Barla, 1995). After investigating a considerable quantity of tunnels in soft rock in Japan (Aydan et al, 1996), it was found that the values of bearing factors of the wall rock in tunnels where rock creep was evident are all smaller than 2, which was derived from the typical weak rocks such as marlite, tuff, shale, and pink gritstone, in the depth range of being not more than 400m underground. It is still not known whether this conclusion is available to hard rock in deep mining.

It is found that time-dependent behavior of wall rock at high confining pressure is considerable evident through a lot of observations in ultra-deep golden mining engineering where the biggest shrinking ratio between two walls even approached to 500mm/month, in South Africa (Malan, 1999; 2002; Malan and Basson, 1998; Malan and Spottiswoode, 1997). Thus it is extremely difficult to anchor and line in the process of ultra-deep excavations.

If rock is acted on at high confining pressure together with other disadvantage effect, its creep effect will be more prominent. Such creep effect is ubiquitous in nuclear waste disposal. For example, strength of hard granite can be terribly reduced by the micro-cracking effect and the underground water induced stress corrosion (Kwon et al, 2000). Accordingly, the strength reduction of the rock mass will threaten the stability of near field surrounding to the nuclear waste repository. Additionally, creep phenomenon in rock is also associated with micro-cracking induced rock denudation. In Sweden, the depth of rock denudation will approach to be 3m, according to the observation recordings of candidate sites at Forsmark for nuclear waste repository and the estimation from long-term creep criterion (Pusch, 1993).

(iii) Bulk expansion at deep rock

Phenomenon of bulk expansion at uniaxial compression test was first observed by Bridgman (Bridgman, 1949), which appears before failure of rock sample. Similar phenomenon was also found under the condition of confining pressure by Matsushima (Matsushima, 1960), but then other finding was given that expanding value of rock bulk decreases with the increase of confining pressure. Further the experimental tests by Kwasniewski (Kwasniewski, 1989) showed that, at low confining pressure, rock bulk expanded under the peak strength due to the internal micro-crack opened, at high confining pressure, on the other hand, phenomenon of rock bulk expanded is not obvious or disappearing.

3.2. *Strength and failure of rock at depth*

It is shown that in general rock strength is stronger with the increase of depth (Zhou, 1990; Li, 1996). For example, the proportion of rock with strength being 21-40MPa is reduced to 24% from 30% with the mine depth increased from 600m to 800-1000m. In contrary, the proportion of rock with strength being 81-100MPa is then increased to 24.5% from 5.5% and the rock within this range of strength is more brittle and easier to burst.

A nonlinear strength criterion of rock mass that was derived from the extremely high lateral stress (700 MPa) was proposed by Singh (Singh et al, 1989), based on a mass of experimental data. From the observation of experiments, granite strength is low under the condition of different confining pressure and temperature 200-280℃. Therefore, a model of Christmas tree that has been used to reasonably explain the seismogenic zones in the crust for strength of crust structure was presented by Shimada and Liu (Shimada and Liu, 1999).

With the increase of mining depth, mechanism of rock failure is supposed to be varied, i.e., brittle failure in shallow rock controlled by its fracture toughness transitting to crack growth failure in deep mining controlled by its lateral stress. In other words, failure mechanism of rock in deep mining indeed should be quasi-static instead of dynamic in shallow mining, as well as latent mechanical response of ductibility in deep mining instead of brittle response in shallow (Cleary, 1989). On the contrary, some people believed that most rock failure in deep mining exhibits dynamical feature, i.e., so-called rock burst or mining seismology (Gibowicz and Kijko, 1994).

In deep mining, it is not only rock burst occurrence clearly related to, but rock burst strength and mining seismology also dependent on velocity of strata motion (Ortlepp, 1993). Accordingly, displacement and motion velocity of terrane now are widely referred to as two important parameters so as to be utilized for predicting rock burst. One of the reasons of rock burst, in addition, may be of deep mining and falling, as well as weak strata activation of geological structure. Even the stress redistribution near to geological structure probably leads to foreshocks in the field of mining. Thus rock burst is much more complicatedly distributed in either space or time, in deep mining. It is noted that the time series constituted by rock burst event do not obey Gaussian distribution (Marcak, 1997).

3.3. *Fragmentation of rock at depth*

It is easy to excavate and explode in deep hard rock in deep mining, which indeed means that high stress in the field of underground engineering plays a key role in rock failure. Accordingly, how to effectively prevent and control disaster event induced by high stress and at the same time how to take full advantage of fragmentation for rock under high stress, stress wave and stress field, is an inevitable challenging issue in deep mining nowadays.

In the recent decades, a series of investigations that include the behavior of rock at high stress and the dynamical response and characteristics of rock induced by dynamical loads respectively have been conducted. For example, most of the experimental tests by triaxial instrument have been performed, involving the physical characteristics and the failure of rock at high stress. On one hand, theoretical modeling and numerical simulation for rock constitutive and fracture failure mechanism based on meso-mechanics and fracture damage theory were utilized to explain the essential phenomenon of impact pressure and rockburst (Tang et al, 2000; Feng and Wang, 1998; Xie, 1990). On the other hand, Hopkinson bar and light gas gun were taken as impact apparatus to study dynamic response and dynamic parameters at high strain rate for rocks. From the viewpoint of stress wave theory, a series of dynamic failure laws for rocks were obtained considering modeling and numerical simulation for dynamic constitutive, propagation of stress wave in rock, energy dissipation, and interface effect for rock (Kirzhner and Rosenhouse, 2000; Chen et al, 2000; Li and Gu, 1994; Li et al, 2000 ; Zhang et al, 2000). To my best knowledge, dynamic behavior and fragmentation mechanism for rocks at high stress are even not seriously considered (Gu, 2001; Li and Gu, 2002). Only was little theoretical analysis placed on the stress fluctuations in brittle fracture composed with high stress (Dyskin, 1999), for brittle material.

4. Geological Mechanics for Rock Mass in Deep Mining Engineering

Deep rock mass is one type of complex material with characteristic of modern geological environment, which implies to the characteristics of long-term geological evolution and full trail of formulation and reformulation and can be shown in Fig. 3.

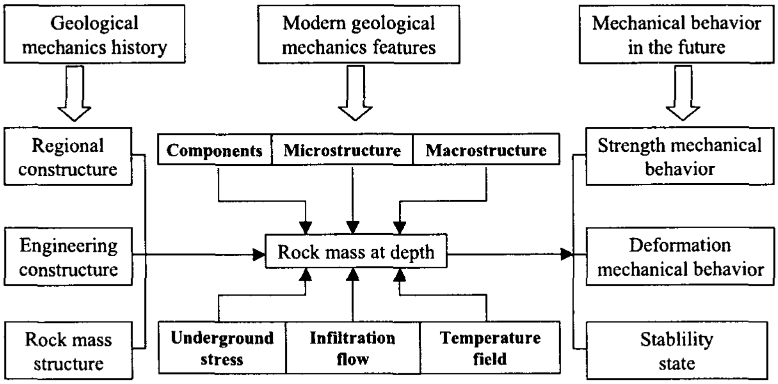

Fig. 3. Characteristics of geological mechanics of rock mass at depth.

The nonlinear phenomena such as impacting pressure, rock burst, sudden gas leakage, creep, and water-emitted, attributes to the especial environment of physical geography and the complicated stress field, for rock mass at great depth. As a result, the significant thing is not the linear elastic theory which can generally be used to simulate the physical process for shallow rock mass, but the nonlinear mechanics of stability and stable control which are difficult to be treated by the classical continuum mechanics. Table 1 presents the typical comparison of physical features between deep mines and shallow mines.

5. Mechanical Characteristics of Rock Mass in Deep Mining

5.1. *Mining environment (HHHD)*

As the mentioned earlier, the main features of mining environment at great depth is high ground stress, high earth temperature, high water pressure and mining disturbance (HHHD). They are determined by the especial geological environment of rock mass at great depth and will be further discussed in details.

(i) High underground stress: Only is the vertical stress ordinarily produced by the gravity field of geological media more than the compressive strength (>20 MPa) of engineering rock mass. Needless to say, concentration stress induced by engineering excavation is much more than that of the compressive strength (>40 MPa). Simultaneously, it is also shown in the stress data of observation in strata that rock mass is formed in the process of long-term geology evolution and is with leaving behind the trace of constitution motion of remote antiquity, among which two stress fields involving constitution and residual exist, and the superposition of two fields cumulates to be in the state of high stress which is a singular field of stress for the rock masses at great depth. According to the measurement in South Africa, underground stress is about 95-135 MPa for the rock masses at the range of 3500-5000m. It is inconceivable to be excavated at so high underground stress.

(ii) High underground temperature: From the thermometer measurement, the further from ground the rock mass is, the higher the underground temperature is. Gradient of underground temperature is commonly about 30-50 ℃/km, and is conventionally equal to 30 ℃/km, but it sometimes approaches to 200 ℃/km near to fault or local abnormal zone with good conductivity. Meanwhile, the mechanical and deformable behaviors of rock mass at the condition of super-conventional temperature are considerably different from at the conventional condition. Variation of underground temperature makes rock mass broken into pieces. For example, temperature altered with 1 ℃ within rock mass is corresponding to the change of underground stress, 0.4-0.5 MP. Accordingly, the underground stress varied with the underground temperature rising significantly influences the mechanical behavior of engineering rock mass.

(iii) High dissolution hydraulic pressure: With the rising of underground stress and temperature, dissolution hydraulic pressure also becomes bigger. For example, the dissolution hydraulic pressure nearly equals to 7 MPa or so, while the mining depth is more than 1000m. Consequently, it makes mine be even easier occurring to flood.

Mining disturbance here is mainly produced by intensive excavation and tunneling at depth. While mining at great depth, most of the tunnels are acted not only by high underground stress, but also by strong supporting pressure distributed within large zone of cyclic excavation. Accordingly, the pressure surrounding to tunnel that is influenced by mining is as many or decade times as the original pressure of rock mass not being disturbed. One of the concrete examples is that ordinary hard rock mass at shallow mine may exclusively behave as the aforementioned characteristics such as large deformation, high underground pressure, and hard to support if the same rock mass situated at great depth. Moreover, it is noted that rock mass at shallow mine is usual at the state of elasticity whereas rock mass at great depth may be at the state of plasticity. In other words, compressive and shear stress induced by the original stress field at inequiaxial pressure might try to be beyond rock mass strength, although it indeed is impossible.

5.2. *Mechanical transitional characteristics of rock mass in deep mining*

Since the synthetized influence of HHHD action is very strong in deep mining, geological mechanical environment around the wall rock mass of mine engineering is evidently varied in deep mine comparing with those of shallow one. As a result, the wall rock mass exhibits especial mechanical features (He, 2004), presented in the following.

(i) Complex stress field around the wall rock mass

Surrounding wall rock mass of tunnel can often be divided to three parts involving with relaxation, plastic, and elastic zone, respectively, in which their constitutive relationship can be derived from the elasto-plastic theory of continuum mechanics (Fu, 1995). However, it is shown that expanded zone and compressed zone or so-called broken zone and unbroken zone can be alternatively appeared in the surrounding wall rock mass of tunnel at great depth, and their zone increment is geometric proportional to their width, which is also regarded as the phenomenon of zone broken by Shemyakin. Some in-situ measurements also presented a composite character of tension-compression in the deformable zone around the wall rock mass at great depth (Fang, 1996). Thus we say that a more complex stress field exists at great depth.

(ii) Large deformation and strong rheological characteristics of wall rock

As the discussed earlier, there are two complete different kinds of rock deformation in deep strata. One is the behavior of persistent strong rheology of rock masses, which is not only the larger deformation, but also obvious 'time-dependent' (Jiang et al, 2004; Zhai and Li, 1995), just as there are some of laneways where the bulging has approached to several decameters for 20 years. It is found that the rheology phenomena of wall rock is much more evident in the deep golden mines in South Africa, through systematic studies, in which the maximum speed of motion surrounding rock is near to 500 mm per month (Malan, 1999; 2002; Malan and Spottiswoode, 1997). The other is that rock masses do not have any evident deformation at all, but these rock masses are considerably cracked. According to the classical concept of rock mass failure, this kind of rock mass does not have any ability of bearing at all. It is interested, however, that indeed these rock masses are still able to work and restabilize (Zhai and Li, 1995; Jing, 2001). Some laneways are disposed within the creaking rocks (or coal) by considering this feature of rock cracked, e.g., mined-out tunneling.

Table 1. Mechanical characteristics of rock mass in deep mines compared with those of shallow mines.

Material	**Forces**	**Theory**	**Energy field**	**Loading process**	**Superposition principle**	**Design method in engineering**
Brick	Weight	Linear	Conservation	Independent	Available	Parameter design
Rock mass in shallow mine	Weight & low stress	Linear	Conservation	Independent	Available	
Rock mass in deep mine	Weight & high stress	Nonlinear	Dissipation	Dependent	Unavailable	Design based on nonlinear mechanics

(iii) Catastrophic effect of dynamical response

The failure process of shallow rock mass is progressive, and it has a clear signal of failure (e.g., large deformation). But in the deep strata, the process of dynamical response is almost always abrupt, and is without any signal. This dynamical response is also associated with the characteristics of strong impacting devastation which phenomenologically behaves as sudden destabilization induced collapse in large-scale on the roof of the laneway or the surrounding wall rock mass.

(iv) Brittle-ductile transition of rock mass at depth

As the test researches show (Paterson 1958; 1978; Singh et al, 1989; Kwasniewski, 1989), the rock masses are with various post-peak behaviors under different confining pressure, so that the final strain at rock mass failure is not a constant. For example, the failure of rock mass in the shallow mining mainly behaves as a type of brittle formulation, with little or without any permanent (plastic) deformation. In the deep mining, however, the rock mass under the action of HHHD is indeed with the character of post-peak strength that may transit from brittle to ductile under the high confining pressure. Usually the permanent deformation is large when the rock mass failures. Because of these, as the depth increases, the ductile mechanism of the rock mass in the deep strata has been activated instead of the brittle response in the upper strata (Cleary, 1989).

5.3. ***Six features of mine hazard***

Since the significant thing is the evident difference of mechanical performances of rock mass between at shallow mine and at deep one and further the storage environment for the rock mass at depth is complex, hazard event that happens to occur in deep mining, such as rock burst, water gusting, large-scale compression suddenly acted on the roof, and the disability of mined-out area, is more severe in damage scope, more frequent in frequency, and more complex in hazard mechanism, compared with the hazard in shallow mining. The main features are in details presented as follows.

(i) The frequency and the strength of rock burst are all obviously stronger

From the statistic data, rock burst usually happens in the hard terrane (or coal) where it is stronger and thicker. The main factor of rock burst consists of the condition of roof and floor for the coal layer, the in-situ stress of rock mass, the depth of embedment, the physical-mechanical performance of the coal layer, the thickness, the rake angle, and so on. It is shown by the statistical data that, though the records of rock burst ever occurred in the hard coal bed which was at very shallow, e.g., the depth being less than 100m and even being taken from 30-50m, generally speaking, rock burst is closely associated with digging depth, i.e., with the increase of mining depth, the number, the strength, and the scale of rock burst will increase too.

(ii) The amplified confined stress in the quarry

As the depth is increasing, compressive stress by the self weight of overlying rock mass is enhanced and tectonic stress is also strengthened. It is accordingly induced that the violent deformation of surrounding wall rock mass is yielded and further the tunnel and the quarry lose their stability as well as the hazard shock bump is readily occurred, with lots of trouble in the roof plate management.

(iii) More serious water gusting

Since the catastrophic water gusting event induced by the collapse of karst mass at Fangezhuang mine at Kailuan on 2nd June 1984, there are a series of large-scale water gusting of Ordovician limestone karst one after another occurred to various places, including Yangzhuang in Huaibei, Xin'an in Yima, Wutong at Fengfeng, Renlou in the north of Anhui Province, Zhangji in Xuzhou, in China. From the preliminary estimation, the economic losses for these hazard events were over 2.7 billion RMB and at the same time some negative effects were generated to geological environment.

(iv) Large deformation of surrounding wall rock mass and regional failure

The purpose of deep support system is to make an effort keeping the integrity of wall rock mass and preventing from the movement of broken rock mass, as such doing in shallow mining. During deep mining, the confined stress by self weight of overlay rock mass is progressively enhanced, at the same time, for the rock tectonics at depth is fully developed and the tectonic stress is much more remarkable, the pressure of laneway wall rock mass is large and the cost of the support system is naturally risen. From the statistics of the mining industry in China, the cost of the support system in mining engineering now is 2.4 times as much as 10 years ago, and further the rebuilt laneways take up 40 percent of the total drivage amount.

In addition, the phenomenon of generally accelerating deformation can be observed when the surrounding wall rock mass in the shallow mine approaches to failure, from which engineer can give a forecast of rock mass failure appeared to be in the local field. Contrarily, before failure, the wall rock mass at great depth seems to be nearly stable, and also the failure precursor is extra-ordinarily obscure, these make it hardly predict the failure of wall rock mass. Thus, the failure of wall rock mass in deep mining often appears intensively and is with the characteristics of regionality, e.g., the large-scale collapse of roof.

(v) High underground temperature and bad working environment

In the condition of deep mining, high underground temperature is in the order of several decade in temperature, e.g., the average temperature of 30-40 ^{0}C at the depth of 1000m in Russia being, even specific temperature of 52 ^{0}C. At the depth of 3000 meter in South Africa, the underground temperature is about 70 ^{0}C. As a result, it is impossible for the miners to efficiently work with the perilous environment and even not to normally operate.

(vi) Large amount of gas surging

Another significant thing is the content of gas rapidly accumulated with the increase of mining depth, where gas hazard is frequently occurred. In the recent years, there are 70 percent of the coal accidents caused by gas surge and blast that resulted in over 10 casualties taken place in the exploiting depth of over 600m in China. On the other hand, coal layer at depth is usually accompanied by the higher temperature field, which can readily self-ignite and further activize fire and gas blast in the mine.

5.4. *Ten theoretical problems*

Compared with the shallow mining, we can summarize that 10 theoretical issues in deep mining closely associated with the complex geological-mechanical environment should be noted, including the following aspects: (1) Mechanical performance of deep engineering rock mass, (2) Continuity of deep engineering rock mass, (3) Constitutive relationship and parameter determination of deep engineering rock mass, (4) Strength determination of deep engineering rock mass, (5) Failure criterion of deep engineering rock mass, (6) Structure uniqueness of deep rock mass, (7) Large deformation of deep engineering rock mass, (8) Design method based on nonlinear mechanics for deep engineering rock mass, (9) Load calculation of the deep tunnel rock mass, and (10) Stability control and technique of deep engineering rock mass.

6. Major Research of Deep Mining Engineering in the Future

Although a lot of research work has been done, only part of knowledge concerned with deep mining engineering can be realized nowadays. In the other word, the past researches that emphasized particularly on technology and specific project are not enough to describe the nonlinear mechanism of rock mass system coupled with HHHD from the fundamental viewpoint of physical mechanics. The reason for this is that the special geological-mechanical environment induced by HHHD strongly influences the nonlinear mechanical behavior of deep engineering rock mass where traditional theory and technique such as theory of elasticity are partial or completely not available. Accordingly, it urgently calls for the basic research on the deep mining (at the depth of 1000-1500m) for the energy strategy in China.

6.1. *Rock mechanics in the deep mining engineering*

We here again emphasize that the complex geological-mechanical environment induced by HHHD leads to the evident differences of the mechanical behavior and the hazard character between at shallow and at deep mining. Naturally, the traditional theories based on shallow mining with three disadvantages are not available to investigate the problems we will face in deep mining.

(i) Strength criterion determined

In shallow mining, for lower underground stress, engineering rock mass strength can generally be derived by the test of rock block, which is determined by a rock sample loaded until its failure. On the other hand, for higher underground stress, the characteristics of strength variation due to the change of stress state at some direction after excavating in deep mining, is a complex type of stress composed by tension and compression instead of a single tension or compression. In other words, both unloading and loading are simultaneously produced at the radial and the tangential directions. Therefore, it is not possible to simply derive from the test strength of rock mass for the engineering rock mass, but a composite strength criterion of tension-compression should be proposed, so as to accommodate the basic features of deep mining.

(ii) Stability control model

In shallow mining, for lower underground stress, the surrounding wall rock mass usually cannot be broken after excavating. Thus primary support is enough to maintain the safety stability of tunnel. On the other hand, the surrounding wall rock mass in deep mining after excavating will be destroyed under the action of high confined pressure over the strength of engineering rock mass. At the time, simple support cannot meet the requirement of the safety stability of deep mining. However, its safety stability needs to be maintained through at least two secondary or more support. It is also noted that the stability control model based on shallow mining is not valid at depth whereas an alternative stability control model should be proposed to simulate the secondary or multi- support in deep mining engineering.

(iii) Design theory

The traditional linear design can work in the stability control in shallow mining for the simple mechanical environment. However, in deep mining, single linear design cannot be also applied to maintain the stability of deep mine rock mass. Corresponding to the nonlinear stability control mentioned-above, we should consider an advanced method such as secondary or multi-times of stability control design closely associated with the nonlinear large deformation theory.

6.2. *Major research requirements in the future*

Against the rock mechanics problems in deep mining engineering, the focus of the future research should be concentrated on the basic characteristics of deep rock mechanics, the stability control of deep mining engineering, the damage control of ground environment, and the basic theory of synthesized mining in the deep thick coal layer.

6.2.1. *Concept of depth mining and its classification system*

In spite of paying much more attention to the rock mechanical problems rooted in deep mining, the definitions of 'depth' and 'deep mining engineering' are disagreed in the literatures and their concepts and classification standard have not been accepted commonly. The indefinite concept or criterion negatively affects further development and communication of theory and technology in this scientific field. The urgent affairs, accordingly, are how to scientifically define these concepts such as depth, classification system, and appraisal index, according to the special geological-mechanical environment of HHHR, so as to motivate the basic research on the mechanics of rock mass in deep mining engineering.

6.2.2. *One-Disturbance Research on the fundamental properties of deep rock mass*

The complex geological-mechanical environment induced by HHHD not only leads to thorough changing of the microstructure and the behavior of rock mass and the response of engineering rock mass, but also often suddenly causes unpredicted catastrophic events in deep mining. Therefore, under the long-term action of HHH, how to describe and explain the special mechanical behavior of deep rock mass by excavation and disturbance is the key scientific issue what we face in deep resource exploiting. In particular, we should pay much more attention on the cause of the evolution mechanism of high underground stress field, the mechanical stress field coupling with multi-physical processes, the method how to determine the tension-compression compound strength under the situation of complex stress, and the mechanism of catastrophic evolution.

6.2.3. *Stability research of deep mining engineering*

Different from general ground engineering that does not allow entering the state of plastic failure, the stability of deep mining engineering deals with a complex mechanical mechanism that mainly focus on the interaction between the surrounding cracked wall rock mass and the support system, including the interaction mechanism between the local roof structure and the support after the roof failure by excavating disturbance, the mechanisms of secondary stability interaction between the surrounding wall rock mass with plastic large deformation and the support system. At the same time, because of the nestification and coupling between mining stress field and tunneling stress field, complex three-dimensional stress field is yielded, in which the distribution of mining stress, the multi-dimensional and dynamic space-time rule of mining space, and the scope and stress peak of the bearing part all are considerably varied. Based on the advanced analysis of the space-time distribution of deep quarry and mining stress in deep mining field and wall rock mass, together with the investigation of nonlinear mechanical feature in deep rock mass, a nonlinear mechanical control strategy that can take into account the integrated effects between mining field and tunnel should be presented.

6.2.4. *Hazard mechanism research in deep mining engineering and its strategy*

It is also important for us to understand stable and unstable deformation characteristics of deep rock mass under the action of multi-physical processes such as high underground stress, underground water, gas, and temperature. Then the failure state and conversion mechanism and condition and law for rock mass at depth, also need to be further studied. The character of frequency and the strength of hazard under the action of coupling multi-physical processes needs to be known, so as to explain the hazard process and the hazard induced mechanism and present a feasible strategy for dealing with rock burst, water gusting, coal and gas blast, large-scale pressure acted on the roof, and unstable mined-out zone.

6.2.5. *Basic research on the thick seam and full-mechanized caving at great depth*

Thick seam and full-mechanized top-coal caving is an international leading technique in mining engineering. It is a key issue to determine caving possibility of deep thick coal seam while using the full-mechanized caving, based on the investigation of basic mechanical characteristic of engineering rock mass combined with considering the complicated geological-mechanical environment of HHHD. Meanwhile, how to efficiently model thick seam and full-mechanized caving and how to determine optimized engineering parameters are the foundation of full-mechanized caving technique applied to mine deep resource, taking account of the change of tunnel stress distribution in time-space and the mechanism of thick seam and full-mechanized caving under the condition of complex stress field.

7. Conclusions

The coal resource at great depth is still the main part of energy resource storage in the 21st century at least in China, and corresponding to this situation it is necessary to do the fundamental work of rock mechanics in deep mining engineering. In the complex geological-mechanical environment of HHHD, the challenging issue is how to treat geological hazards such as rock burst, water gust, and gas blast in deep mining engineering, from two aspects of hazard mechanism and hazard control technique. Different from the ground engineering where its plastic failure is not allowed, the rock mass engineering at great depth is indeed commonly failed due to excavation, but these broken rock masses could again be at the state of stability by interaction with supporting system. Therefore, a comprehensive investigation of the rock mechanics in deep mining engineering are not only to develop the reliable theoretical model to simulate and explain the complex mechanism of multi-field coupling, but also to guarantee the sustainable development of the national economics and implement the security strategy from the viewpoint of energy resource.

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ROCK MECHANICS CONSIDERATIONS FOR CONSTRUCTION OF DEEP TUNNELS IN BRITTLE ROCK

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Tunnelling in brittle rock at depth poses unique problems as stress-driven failure processes often dominate the tunnel behaviour. Such failure processes can lead to gradual ravelling or to violent, strainbursting modes of instability that cause difficult conditions for tunnel construction, whether advanced by TBM or by blasting.

The author's experience with brittle failing rock in deep mining and Alpine tunnelling was previously presented in keynote lectures at GeoEng 2000 (Kaiser *et al.* 2000), summarizing a decade of collaborative research work on brittle rock failure, at the Rockburst and Seismicity in Mines Symposium (Kaiser 2005), introducing new means of complex data interpretation in seismically active mines, and at GEAT'06 (Kaiser 2006), focusing on recent experiences from deep Alpine tunnelling. This article briefly summarizes findings from previous studies, presents recent developments and highlights implications that are of practical importance with respect to constructability. Guidelines for support selection in brittle failing ground are given, and brittle failure processes at the tunnel face are briefly analyzed.

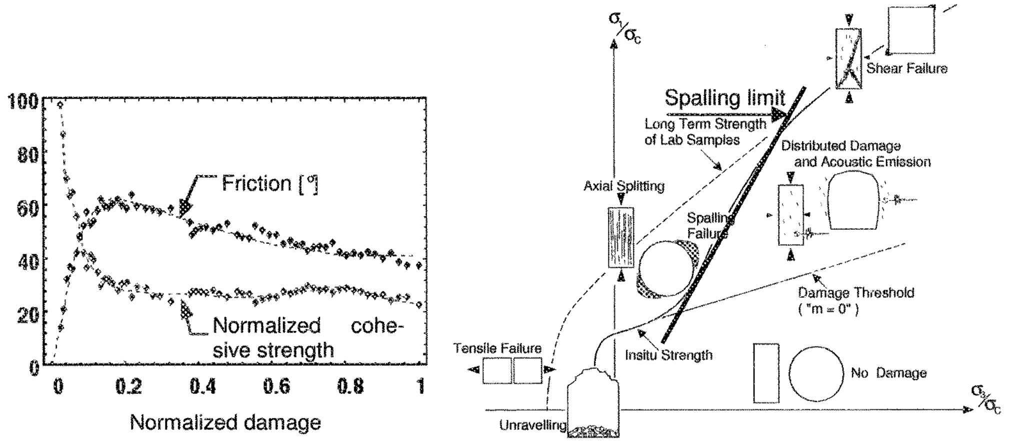
Keywords: Brittle failure, constructability, tunnelling, ground control, rock support, rock bursting.

1. Introduction

The theme of this conference is "rock mechanics in underground construction" which presumably involves rock mass characterization incl. in-situ stresses, design and construction; each component posing its own challenges. Contrary to other fields of science, in rock engineering when dealing with a natural material, it is difficult to obtain representative parameters for design, in part because of scale effects or because of the heterogeneous, discontinuous nature of rock. The issue of obtaining design parameters for design of structures in or on rock was eloquently discussed by Hoek (1999) in his lecture on "putting numbers to geology".

Difficulties in designing underground excavations are also often experienced because constitutive laws in numerical models do not necessarily reflect the actual behaviour of rock. This is particularly true when dealing with brittle failure where the fundamental paradigm of the Mohr-Coulomb yield criteria $\tau = c + \sigma_n \tan \phi$, relating the shear strength τ to a cohesion c and a simultaneously acting frictional resistance $\sigma_n \tan \phi$ is not valid (Martin 1997, Martin *et al.* 1999, Kaiser *et al.* 2000). As intact rock is strained, cohesive bonds fail and the frictional resistance develops at different rates and as brittle rock "disintegrates" (Fig. 1.a). Furthermore, damage initiation and propagation occurs at different stress thresholds (Diederichs 2003) and the propagation of tensile fractures depends on the level of confinement (as established by Hoek 1968 and used to explain brittle failure by his collaborators (Kaiser *et al.* 2000)). A bilinear failure criterion (Fig. 1.b) captures this dependence on confinement for materials that are prone to spalling. Brittle failure by tensile spalling occurs if the stress path moves above the damage threshold and to the left of the spalling limit.

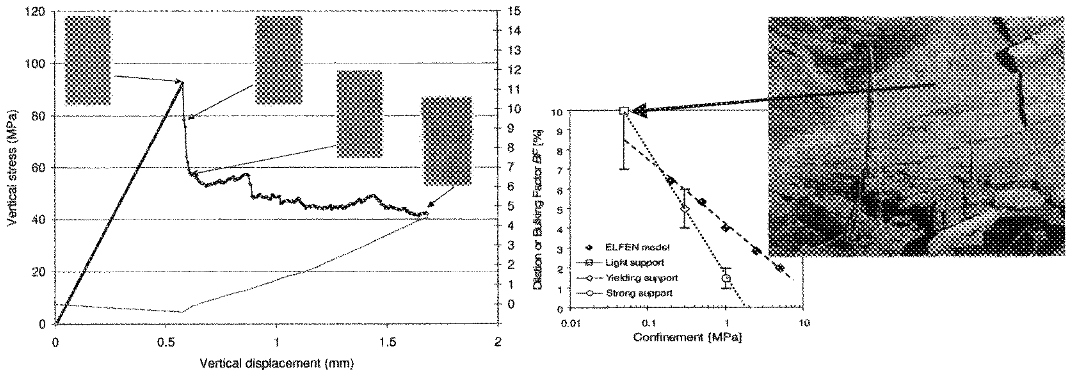
Beyond these thresholds, the volumetric deformation characteristic of brittle failing rock changes drastically as geometric incompatibilities between fractured rock fragments lead to high dilation (Fig. 2 (photo insert)). Most interestingly, while the depth of failure is essentially independent of the support pressure, the dilation of the broken rock is very sensitive to confinement and thus to the rock support characteristic or support pressure (Fig. 2).



(a) (b)

Fig. 1. (a) Cohesion and frictional strength development as a function of plastic strain or damage (Martin 1997); (b) bilinear failure criteria (Kaiser *et al.* 2000) showing damage initiation threshold and spalling limit.

The development of dilation during the spalling of a triaxially loaded sample confined by 1MPa is shown in Fig. 2.a (Numerical simulation using ELFEN code). After peak, the dilation steadily increases to 4.5% at a total strain of roughly 3-times the elastic strain at peak. This dilation is however highly dependent on the confinement as illustrated by Fig. 2.b. For typical support pressures of <1MPa, the predicted dilation increases rapidly to values in the range of 7 to 10% at <0.1MPa. This is consistent with bulking factors *BF* derived from field measurements (Kaiser *et al.* 1996) showing >30% for unconfined floors and 1 to 10% depending on support type (Fig. 2.b).



(a) (b)

Fig. 2. Photo insert shows rock fractured by dynamic loading (rockburst) with tensile fracturing and related volume increase (bulking); (a) simulated stress-strain and dilation of brittle failing rock using ELFEN (Cai 2006; unpublished report); (b) dilation as a function of confinement (dashed line: ELFEN model; dotted line: data from Kaiser *et al.* (1996).

As a consequence, serious deficiencies in predicted rock mass behaviour and resulting support designs must be expected when numerical models with constitutive models developed to simulate soil behaviour are applied to predict the behaviour of brittle rock. Even if underground excavations are designed based on continuum or with more sophisticated discontinuum models (UDEC, 3DEC, PFC, etc.), difficulties are encountered because of serious limitations in modelling the transition from continuum to discontinuum behaviour. While advanced fracture mechanics codes (such as ELFEN) may eventually help to overcome these limitations, at present, more often then not, excavations and ground control measures are designed based on models with these serious limitations. More importantly, the practical implications of the transition from continuum to discontinuum behaviour are often not considered when selecting ground control methods (TBM shields, rock support, etc.). As a result, construction difficulties, such as drilling and grouting bolts in fractured rock, stabilizing ravelling rock behind a vibrating open TBM, setting ribs on irregular surfaces due to overbreak, slow the construction progress and lead to equipment abuse.

For good constructability, understanding the "post-peak" behaviour of a rock mass is often of critical importance. If rock tends to unravel, it matters what the block size of the fractured rock mass are, what the available time to install support is, and whether the rock fails in a gradual or violent manner. By observing failure processes, it is possible to make predictions about the anticipated behaviour of tunnels and to anticipate situations that may lead to construction difficulties. Unfortunately, costly mistakes are often made because of a lack of understanding the actual rock mass behaviour under stress, over reliance on analyses or experiences in low stress environments.

In the author's opinion, the behaviour of highly stressed, brittle failing rock is often not anticipated and thus not fully integrated into the design. Many lessons have been learned in recent years with respect to the behaviour of deep tunnels in brittle ground and it is the goal of this keynote to provide an improved framework for future tunnelling in hard rock at depth.

1.1. *Robust engineering*

No matter how good the site investigation program and the engineering design analyses are, when tunnelling at depth in complex geological settings, it will not likely be possible to predict the exact response of the rock: it will depend on the location, extent and depth of spalling, the impact of support on rock mass dilation, etc. In such situations, it is necessary to adopt a robust engineering approach that focuses on the construction process and ensures that all construction tools work well (Kaiser 2005). The need for robust tunnelling designs is not widely recognized and practiced as Teuscher (2005) reminded the engineers of the AlpTransit project "... geology comprises all aspects of describing the roc mass, including its geomechanical behaviour" ... (stress, strength, etc.) ... "these parameters are the factors which determine the contractor's performance ...". This comment reflects recent experiences showing that a good design is a robust design that can actually be implemented by the contractor without duress, in a timely and cost-effective manner.

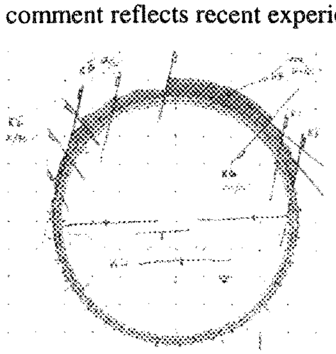


Fig. 3. Shallow stress-induced overbreak in tunnel excavated by TBM in moderately jointed rock.

Alternatively, the challenge of tunnelling within a framework of uncertainty could be managed by adopting an observational design/build approach (Peck 1969). Unfortunately, contractual arrangements are seldom conducive to adapt techniques in a truly responsive manner and, more often than not, lead to undesired delays and cost overruns. A qualitative, observational approach is particularly justified for deep tunnels in brittle rock. As Peck indicated, a measurement-based, quantitative approach though is often not applicable when brittle failure processes predominate.

Robust engineering in brittle rock means to design for rock mass degradation processes that negatively affect constructability.

In brittle failing rock where stress-driven failure leads to a zone of fractured rock near an excavation (e.g., Fig. 3), engineering for constructability basically involves three aspects: (1) retention of broken rock near an excavation; (2) control of deformations due to the bulking of fractured rock, and (3) dissipation of energy if failure occurs in a violent manner.

These three aspects of broken rock management are reviewed here and the reader is referred to Kaiser *et al.* (1996, 2000, 2005) and Kaiser (2005) for details on related subjects.

2. Anticipating Rock Degradation

2.1. *Rock mass degradation and stress-driven rock "fragmentation"*

Rock mass behaviour and failure modes near underground excavations can be grouped according to stress level *SL* and degree of natural jointing (Hoek *et al.* 1995). In strong hard rock, structurally controlled failure processes dominate at shallow to intermediate stress, while stress-driven behaviour dominates if the stress level is sufficient to fail intact rock. If the rock mass surrounding the tunnel is heavily fractured, it tends to increase in volume and cause large wall deformations. Thus, deformation control measures are required to manage the ground.

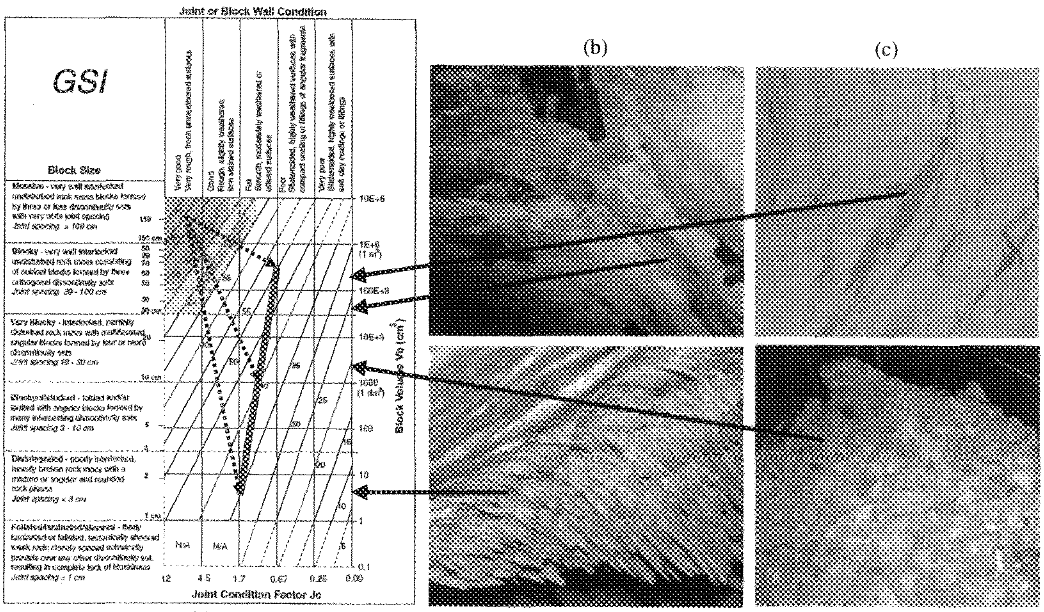
At intermediate stress, typically when σ_{max} > ~0.8 *UCS* [a] or *SL* > 0.8, intact rock fracturing starts to overlap with structurally-controlled processes. From a constructability perspective, however, while stress-driven fracturing may not add much load to the support system, it may increase overbreak and thus the development of "loose ground". In addition, strain-bursting in massive to moderately jointed ground may lead to elevated risk levels (worker safety concerns).

At high stress, typically at *SL* > ~1.15 in strong hard rock, deep-seated stress-driven failure dominates. The rock mass surrounding the tunnel is heavily fractured, tends to increase in volume, and deformation control measures such as rock reinforcements are required.

This keynote relates to conditions encountered at intermediate to high stress, in massive to moderately jointed ground (ground starting with *GSI* > 65; Fig. 4.a).

Stress-driven rock "fragmentation" is a tensile fracturing process (Kaiser *et al.* 2000, Diederichs 2003) in low confinement zones that creates broken rock of varying fragment size and shape. For example, as illustrated by Fig. 4.b, a massive granitic rock spalled to platy shapes with typical aspect ratios of 2:100:100mm to as high as 50:1000:1000, and to similar block sizes and shapes with aspect ratios as high as 300:1500:>2000 at the Bedretto tunnel (not shown) after long-term wall degradation. When rockbursts are involved, the block sizes typically vary from dm^3 to <m^3 size with aspect ratios from cubic to platy. This degradation is plotted on the *GSI* chart (Cai *et al.* 2006; Fig. 4.a), showing that degradation transforms a rock mass of *GSI* >65 to damaged, fractured ground with 35< *GSI* <55. Most importantly, a continuum or tight discontinuum, often with non-persistent joints, is transformed to a "loose" discontinuum with often continuous and open fractures as illustrated by the photos in Fig. 4.b to e.

[a] $\sigma_{max} = 3\sigma_1 - \sigma_3$ is the maximum tangential stress at the wall of a circular opening in elastic ground, and UCS = σ_c is the laboratory uniaxial compressive strength of the intact rock. The stress level *SL* is defined as σ_{max} /UCS.



(a) (d) (e)

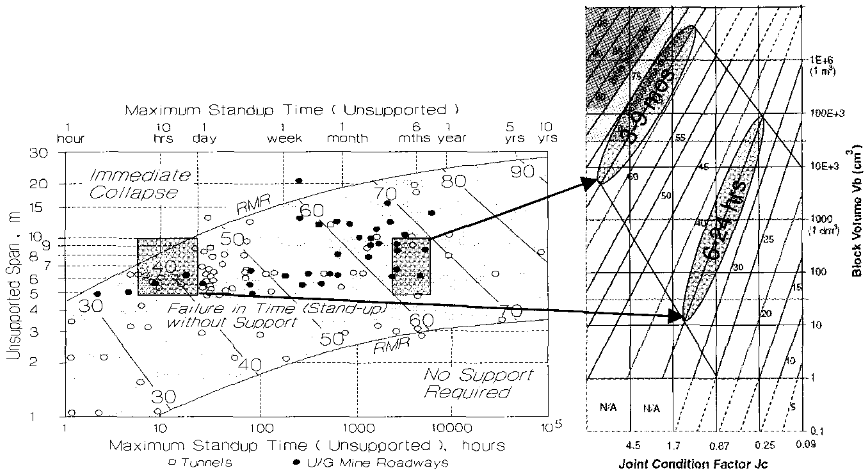
Fig. 4. (a) GSI-chart showing degradation paths for ultimate block sizes of 1.5, 10 and 70cm edge lengths; (b) spalling in back of Piora tunnel, Switzerland (image ~2m wide); (c) Lötschberg tunnel wall (slab ~ 30cm thick); (d) Spalling in notch at URL, AECL, Canada (image ~1m wide); (e) fragmented fall of ground due to rockburst (image ~2.5m wide).

2.2. *Ravelling of stress-fractured rock*

When rock is excavated, the stress path, though in a contorted manner (Diederichs *et al.* 2004), eventually ends in a nearly unconfined state (in the spalling zone of Fig. 1.b for brittle rocks). According to Bieniawski's stand-up time chart (Fig. 5), for a 5 to 10m wide unsupported tunnel, nearly permanent stability with stand-up times of several months to a year can be achieved when $RMR = 65\pm5$ or $GSI = 60\pm5$ and acceptable short-term stand-up times for constructability of say 6-24 hours can be achieved when $RMR = 40\pm5$ or $GSI = 35\pm5$. Thus, rock mass degradation and stress-driven fracturing can bring a stable, self-supporting rock mass to the brink of ravelling (compare Fig. 4.a and Fig. 5.b).

According to Fig. 5.b, reasonable stand-up times and thus sufficient time to install support should be achievable if the joint spacing is >3cm with fair to good and >50cm for poor to fair joint condition[b]. Any process that degrades the rock mass to a worse state tends to lead to constructability problems as the fractured rock near the excavation will not be self-stabilizing (e.g., Fig. 3). The spacing limits given in Fig. 5.b, are actually somewhat optimistic, as a 10m wide excavation in jointed rock with a joint spacing of 3 to 10cm would tend to ravel immediately after blasting or when emerging from the shield of an open TBM. Nevertheless, these charts indicate how much rock mass degradation, due to stress, can be tolerated before serious construction problems must be anticipated.

[b] RMR joint spacing rating classes tend to be higher than indicated in quantified *GSI* chart: ratings for jointing factor are 20, 15, 10, 8, 5 for spacing of >200 to 60 to 20 to 6 to <6cm, respectively.



(a) (b)

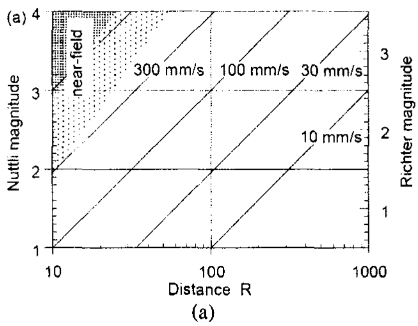
Fig. 5. (a) Stand-up time graph after Bieniawski (1989; from Hutchinson and Diederichs 1996) showing short- and long-term stability ranges for 5 to 10m wide excavations; (b) GSI chart with corresponding characteristics for 6-24 hours and 3-9 months unsupported stand-up time.

Other factors often dominate instability in the zone between short- and long-term stability (35<GSI<65). Of particular importance is the degree of stress relaxation (Diederichs and Kaiser 1999)) and the level of post-peak straining (Martin (1997) and Cai *et al.* (2006).

2.3. *Depth of failure*

One unique, from a constructability perspective very important, observation is that while stresses induce instability of the excavation wall and related ravelling of the fractured rock, this process is mostly self-stabilizing (Fig. 3). Even if the excavation is unsupported, this process does not propagate beyond a finite depth of failure d_f unless the damage-causing stress changes (e.g., due to mining-induced stress changes). For this reason, it is possible to determine the depth of spalling, normalized to the tunnel radius a, by the now well established semi-empirical relationship:

$$\frac{d_f}{a} = C_1 \frac{\sigma_{\max}}{\sigma_c} - C_2 = C_1 SL - C_2 \qquad (1)$$



ppv [m/s]	C_1	Mean SL_o for $d_f = 0$
static	1.37	0.42
0.5	1.54 - 1.74	0.35
1	1.79 - 2.08	0.29
2	2.17 - 2.86	0.23
3	2.63 - 3.64	0.18

(a) (b)

Fig. 6. Anticipated peak particle velocities for 50% confidence (average scaling law parameters: $a^* = 0.5$ and $C^* = 0.1\,m^2/s$; Kaiser *et al.* 1996); (b) C_1 for dynamic depth of failure determination, and mean stress level SL_0 for $d_f = 0$.

Based on an extensive database of tunnels in varying geological settings, Martin *et al.* (1999) established the two constants as $C_1 = 1.25$ and $C_2 = 0.51 \pm 0.1$. Kaiser *et al.* (1996) showed that the constant C_1 depends on the level of dynamic stress increment due to a remote seismic event and found, supported by numerical modeling, that C_1 can be related to the induced ground motion (peak particle velocity *ppv*). Values for C_1 are listed in Fig. 6.b for $C_2 = 0.57$. Other factors such as the frequency content of the seismic wave must be taken into account (for details refer to Kaiser *et al.* 1996). The anticipated ground motion depends on the event magnitude M_L, the distance R from the event and other parameters (see caption) as illustrated by Fig. 6.a. For example, for a stress level of $SL = 0.65$, the d_f/a under static conditions of about 0.32 would increase to about $d_f/a = 0.50$ when affected by a seismic event causing a *ppv* of 0.5m/s.

2.3.1. *Variability of depth of failure*

When examining brittle failures underground, it is evident that spalling is often localized and that the lateral extent and depth of failure varies greatly as illustrated by Fig. 7.a. This is not surprising. Even if the two constants C_1 and C_2 were truly constant, i.e., independent of the rock's proneness to spalling (Diederichs 2004), the *UCS* and in-situ stress, and thus σ_{max}, are variable.

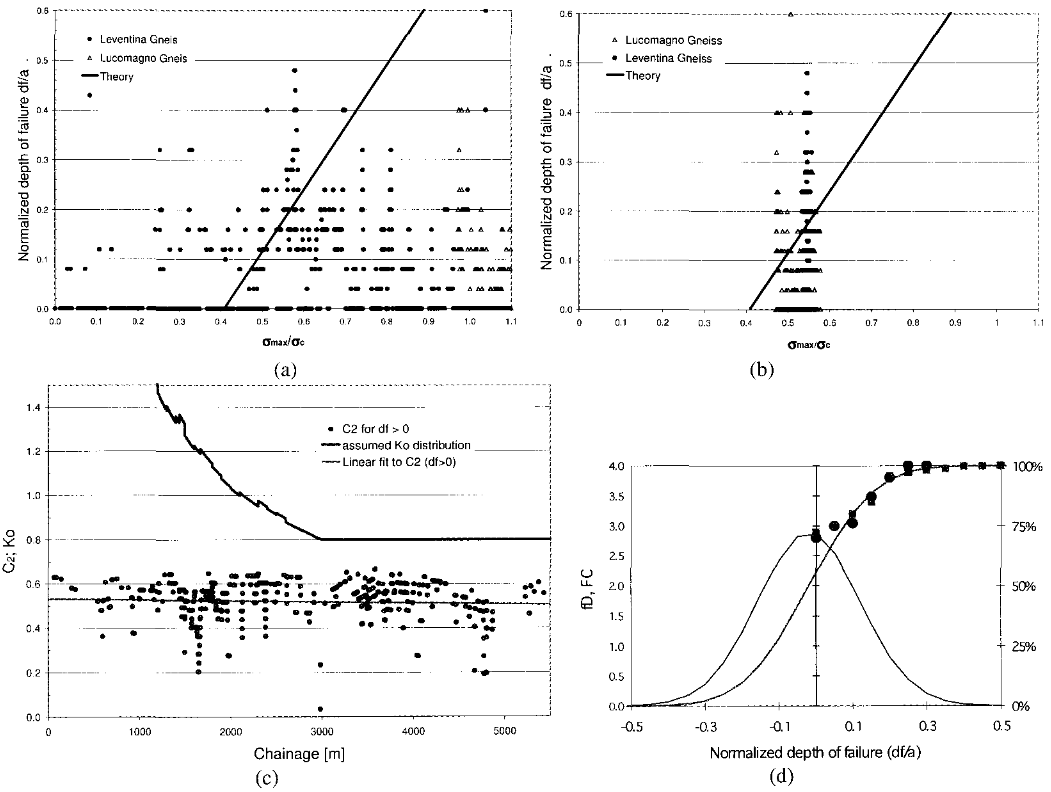


Fig. 7. (a) Normalized depth of failure records from Piora tunnel, Switzerland, assuming constant *UCS*, K_0 and overburden stress; (b) same assuming constant *UCS*, K_0-profile of shown in (c) and overburden stress; constant C_2 back-calculated assuming $C_1 = 1.25$ for measured d_f locations; (d) corresponding cumulative distribution curve with coefficients of variation = 17,15, 10% for C_2, *UCS*, and stress, respectively, compared with data from Piora tunnel (squares) and from Lötschberg tunnel (circles).

After processing the data by assuming a variable stress field (K_0-profile of Fig. 7.c is based on assumption of a linearly decreasing horizontal stress from 22MPa at the portal to 32MPa at chainage 2900 (interface between Leventina and Lucomagno Gneiss), much of the clustering is removed (Fig. 7.b) but d_f/a is still highly variable. Unfortunately, no reliable measurements are available to confirm the assumed stress profile. However, when back-calculating C_2 (assuming C_1 = 1.25), a consistent pattern emerges (Fig. 7.c). The mean value of C_2 is more or less constant over the full length of the tunnel at 0.52 (CoV = 17%).

The *UCS* typically shows coefficients of variation *CoV* of about 25%. The in-situ stress may be highly variable in heterogeneous rock formations (e.g., near dykes) or relatively uniform in massive homogeneous rock. The anticipated effect on d_f/a of a *CoV* of 15% for *UCS*, 10% for stress, and 17% for C_2, is shown in Fig. 7.d (Rosenbleuth 1981). Even though the mean d_f/a is close to zero for the assumed parameters, a maximum of 0.45 may be reached, and, most importantly from a constructability perspective, it follows from the cumulative probability curve, that about 40% of the tunnel should be expected to show stress-driven instabilities. When compared with the data of the Lötschberg tunnel (Fig. 7.d), it can be seen that the observed distributions in both tunnels are almost identical and agree well with the predicted base on Piora data. Both tunnels were excavated with an open TBM and in comparable rock formations.

Equation (1) predicts the mean depth of failure reasonably well if the relevant input parameters are well defined. When combined with standard statistical means and estimates of the *CoV*s, the anticipated frequency of occurrence as well as the range of depth of failure can be obtained to guide excavation method and support selection.

2.4. *Anticipated demands on support due to brittle rock failure*

For support selection to control *static* stress-driven failure, three aspects can now be evaluated:

- the depth of potential unravelling can be constrained (e.g., d_f/a = 0 to 0.45 over 40% of the tunnel) and means to manage this ravelling potential can be chosen.
- the anticipated load demand can be estimated (e.g., at $d_f * \gamma$ = 0 to 0.03MPa for the Piora tunnel) and the static factor of FS_L = load capacity/demand can be established.
- the anticipated deformation demand can be estimated (e.g., at $d_f * BF$ (see above; BF = 1 to 10% with *CoV* of ~25% depending on support type). For the Piora tunnel wall displacements of 11±6 mm, if well reinforced, to 110±60mm, if allowed to loosen with very light support, would be anticipated. Since the bulking is very sensitive to the applied support pressure, the anticipated deformations can be easily managed with a tight retaining system, e.g., early shotcrete.

The required bolt length is equal to the maximum d_f plus a safe anchor length. For split set bolts, the pick-up length (d_f) must be deducted from the bolt length to assess the holding capacity of the remaining anchor length. Other failure modes such as structurally controlled failures, must be assessed separately.

For support selection to control stress-driven failure under *dynamic loading*, dynamic deepening of the failure zone as well as the shakedown and energy release potential must be considered.

- The dynamic deepening and thus the resulting potential depth of ravelling can be estimated for a design event magnitude as a function of the distance R to a potential event (e.g., Fig. 6.a for the assumed scaling law; $log(R\ ppv) = ML + (1.5{\pm}0.15){+}2\ logC^*$ with M_L = local Richter magnitude and constant C^* depending on static stress drop and confidence level (Kaiser *et al.* 1996)). For the example presented above with a *ppv* of 0.5m/s, the mean static d_f/a = 0.32 would be expected to deepen to a mean dynamic d_f/a = ~0.5, with a maximum around 1.2 (Fig. 8.a). Thus, dynamic fracturing could locally deepen to as much as 3m with a corresponding static load increase to 0.08MPa and a wall deformation of between 30 and 300mm due to bulking depending on the effectiveness of the support. As shown by Fig. 8.a, the potential for no failure at that location would be near zero.

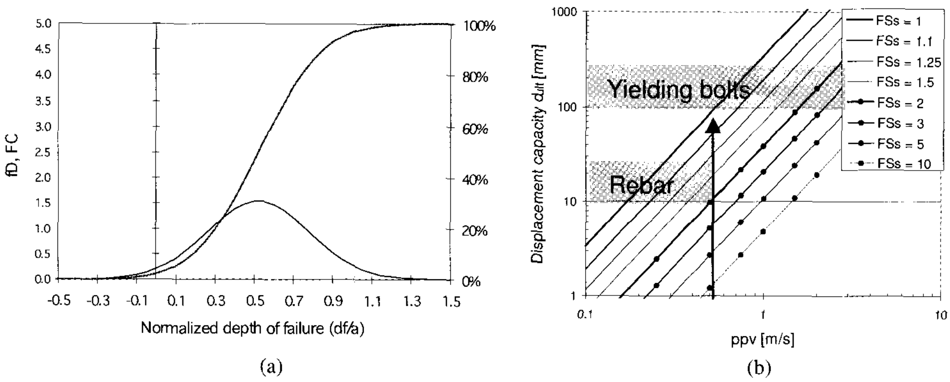


Fig. 8. (a) Dynamic depth of failure distribution for ppv = 0.5m/s and conditions of Fig. 7; (b) ultimate support displacement capacity requirements as a function of static factor of safety FS_s (before shaking).

- In addition to the dynamic deepening of d_f, the ground motion can lead to a shakedown failure of the broken rock. According to Kaiser et al. (1996), the survival limit of a support system depends on its static factor of safety FS_s (before shaking), the ground motion ppv and the ultimate displacement capacity d_{ult} of the support:

$$FS_s = 0.87\left[\left(\frac{ppv^2}{2gd_{ult}}\right)+1\right] \quad \text{or} \quad d_{ult} = \frac{ppv^2}{2g\left(\frac{FS_s}{0.87}-1\right)} \tag{2}$$

The static factor of safety is the FS after dynamic deepening of the depth of failure zone. For ppv = 0.5m/s, standard grouted bolts (rebar) would survive if this factor of safety was higher than 1.25 (Fig. 8.b) or yielding bolts with a displacement capacity of d_{ult} >80mm would be required to (just) survive the event.

- The deepening process of the failure zone may occur incrementally by spalling of thin slabs (Fig. 4.c) or in a violent manner due to the sudden release of energy stored in larger slabs. When energy is released the spalling process is called strainbursting. As Salamon (1984) showed, the energy release is largest when rock is "excavated" in a single step and zero when excavated in infinitesimally small steps. Thus, the worst case exists when the entire depth of failure zone is created in a single event. However, the energy released and therefore the load on the rock support depends on the amount of energy dissipated by friction in the fragmented rock. It is difficult if not impossible to undertake accurate energy release calculations. Aglawe (1999) found that for a 4m wide circular tunnel in 200MPa rock, the maximum releasable energy was 0.05MJ/m^2 at d_f/a =0.5. Considering that burst-resistant supports can at best dissipate 0.05MJ/m^2, it follows that violent failure of slabs exceeding d_f = 1m would destroy the best possible burst-resistant support system. Furthermore, since the energy dissipation capacity of standard, non-yielding supports is ≤0.01MJ/m^2, it follows that the support limit of a previously unloaded support (FS_s=∞) is reached when a strainburst involves a normalized depth of failure in the range: <0.25 < d_f <<1.25m for a = 2.5m. In other words, when strainbursting is anticipated or encountered, constructive means should be used to induce incremental spalling.

3. Face Stability

The stress fracturing and bulking process illustrated above (Fig. 4) is visible only because the rock near the damage location collapsed and provided insight into the disintegrated rock mass. At the tunnel face, broken rock is either excavated during a blast or hidden ahead of a TBM.

Because of the constrained convergence pattern near the face, causing stress concentrations at the edge of the face and thus face parallel loading, the same processes as observed in the tunnel wall can happen at the tunnel face. The anticipated rock mass behaviour of a highly strained tunnel face was interpreted by Kaiser (2006). As illustrated by Fig. 9, two processes lead to face instability: (i) surface parallel spalling (black zones in Fig. 9.b), and (ii) structurally controlled instability mechanisms (gray zones in Fig. 9.b) in response to the opening of joints by the inward bulging of the face. These processes differentially strain the rock in the face and lead to a degradation that produces "blocky ground". Due to this stress-induced fracturing, the block size is generally of predominantly platy shape (Fig. 9.d).

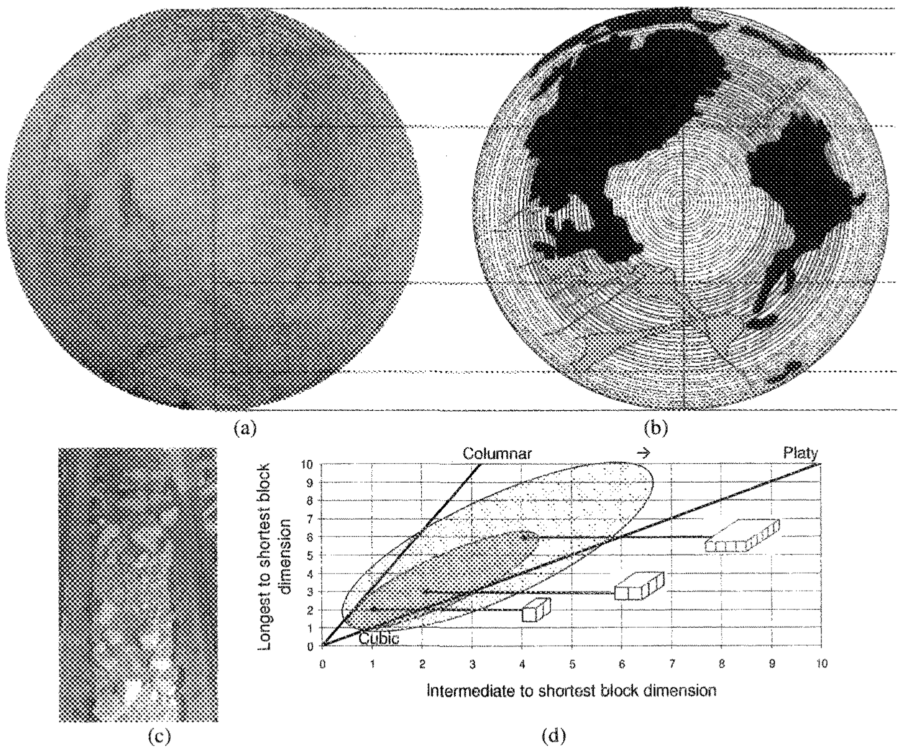


Fig. 9. (a) Photo and (b) sketch of spalling zones of tunnel face in massive to moderately jointed rock (Weh and Bertholet 2005); (c) "blocky rock" rather than typical chips on conveyer; (d) observed, predominantly platy block shapes (shaded zones contain 65 and 99% of data).

There is now ample field evidence in support of the face disintegration process described above and this can be numerically modeled using the brittle failure criterion introduced earlier (Kaiser 2006). Stress-driven spalling processes with or without inherent weaknesses can lead to unstable "blocky" tunnel faces. While pre-existing joint sets may enhance the process of face degradation, the creation of a fracture set that is sub-parallel to the face is the primary cause for the widely observed face instability problems evident in large tunnels at depth. Kaiser (2006) also showed that the observed depth of face failure d_{ff} increases rapidly once an initiation threshold at overburden depth z/UCS ~ 4 is exceeded (z in m; UCS in MPa). At a stress level of z/UCS = 5 to 7, d_{ff}/a typically reaches 0.1 to 0.2. Most importantly, Kaiser (2006) showed that face instability may occur at lower stress levels than when wall instability is first encountered.

From a constructability perspective, this finding is of great practical importance. It clearly demonstrates that the tunnel face can be more prone to stress-induced failure than the tunnel wall. Thus TBMs will have to act as rock breakers rather than disc cutters, leading to additional wear of disks and cutter head as well as additional forces on the TBM, and may cause unexpected material handling problems.

This finding is also not in accord with some of the implicit principles behind current tunnelling standards or contractual arrangements (support and excavation classes). Contrary to the commonly encountered situation where wall instabilities slows the tunnelling progress, in this situation, face instability may slow progress even though the walls are stable and may need little structural support.

4. Conclusion

Constructability of tunnels at depth is often hampered by ravelling of stress-fractured rock near the excavation walls, backs and at the tunnel face. These ground control problems are seldom anticipated or only recognized too late when it is not possible to adjust construction techniques easily without causing delays and extra costs. With semi-empirical means or by modeling with appropriate brittle failure criteria, it is now possible to anticipate the lateral and longitudinal extent of instabilities, the depth of failure as well as the associated loads and deformations. Hence, excavation techniques and support systems can be tailored to manage brittle failing ground in a cost- and time-effective manner. Highly stressed tunnel faces should be shaped in a convex shape to remove potentially unstable rock before it can cause a serious safety hazard to workers near the tunnel face.

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DEVELOPMENT AND APPLICATION OF DISCONTINUOUS DEFORMATION ANALYSIS

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DDA (Discontinuous Deformation Analysis) originally developed by G.H. Shi is an extension of FEM in dynamic analysis. The DDA simulations have been used in some practical problems; especially for discontinuous rock masses and demands are extended to more complex simulations in rock engineering fields. However this method is not widely accepted because of insufficient information and some unknown parameters in the method. Several modifications and improvements are necessary to use the original DDA in practical application. In this paper we discussed the structure of DDA, what the key point of DDA for easy understanding is, how many parameters are used in DDA and how they are determined. Damping effects (physical damping, algorithmic damping, quasi-static and dynamic damping in DDA, time increment) are the most important parameters should be discussed in detail. Some examples of DDA application will be shown. Tunnel excavation in discontinuous rock masses, rock slope stability in earthquake conditions and 3 dimensional rock fall problems etc. are examined. These examples will clearly demonstrate the potential applicability of DDA to rock engineering problems.

Keywords: Discontinuous Deformation Analysis; discontinuous rock masses; simulation.

1. Introduction

Japan is susceptible to numerous types of natural disasters, such as rock falls and landslides induced by earthquakes and intensive rains, resulting in roads, railways and public houses on slopes being damaged and closed. Now about 25000 slopes are identified to be potentially unstable. Ministry of construction and transport announced that these unstable slopes should be taken care as soon as possible to establish the safe and comfortable country. In order to protect against unstable slopes, it is very important to take prior countermeasures. Some government agencies intended to change their conventional safety codes for rock slope and rock fall and to include modern numerical methods. DDA is one of the candidate tools to analyze rock slope stability problems and to use for designing such countermeasure structures (Ishikawa *et al.*, 2004, Hamasaki *et al.*, 2004, Osada *et al.*, 2004, Wu *et al.*, 2003 and Wu *et al.*, 2004). However, in DDA analyses, how we can determine necessary parameters is not clear yet and engineers are still skeptical to use DDA in design works.

Figure 1 shows an example of DDA analyses (Monma *et al.*, 2004). This rock slope failed in March, 1999 and its falling process was recorded by a video camera. DDA analysis was done to estimate the failure mechanism, but we encountered the difficulties of determining many parameters involved in the DDA calculation. Some parameters are obtained by various tests, but some are determined by try and error process. Final results of calculation gave us a lot of hints how the combination of blocks interactively moved (Wu *et al.*, 2005).

Parameters used in DDA are rock mass densities, elastic moduli of rock blocks, the penalty coefficients, viscosity (damping) of blocks, joint friction coefficient, velocity energy ratio, time increment, allowable displacement in a time increment and so on. Who can determine these parameters with confidence?

Here, we will discuss the feature of DDA in relation to the basic equations with respect to rock-fall phenomenon and some new parameters to be considered in an earthquake dynamic problem. In this discussion, we simply focused on the relationship between numerical stability, calculation precision and the parameter characteristics in DDA calculations. Although we pick up a rock-fall phenomenon as an example, the discussion can be extended to general DDA analysis (Wu *et al.*, 2003).

Fig. 1(a). Rock slope before failure.

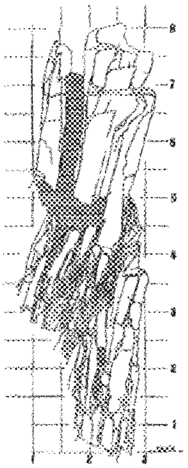


Fig. 1(b). Sketch of jointed rock mass.

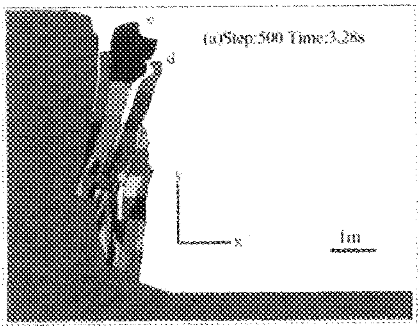


Fig. 1(c). DDA model at 3.28 sec from the beginning.

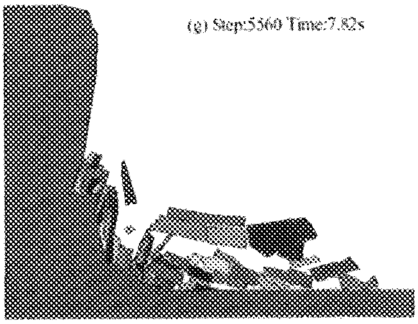


Fig. 1(d). DDA model: Slope failed at 7.82sec.

2. DDA Parameters

In order to get an accurate solution for a complex rock condition in DDA analyses, we should know how the equation behaves and how the DDA parameters interact in the problems.

Here we discussed the parameters for DDA analysis as follows.

(1) Damping effects and viscosity
(2) Energy loss before and after contact or collision
(3) Concerning the balance of numerical stability in the kinematic equation
(4) Viscous damping boundary for dynamic earthquake analysis

2.1. *Kinematic equations of motion (Basic Equation)*

DDA (Shi, 1984) is formulated from Eq. (1) using the kinematic equations based on Hamilton's principle and minimized potential energy expressed by:

$$M\ddot{u} + C\dot{u} + Ku = F \tag{1}$$

where, M: mass matrix, C: viscosity matrix, K: stiffness matrix, F: external force vector, $\ddot{u}$: acceleration, $\dot{u}$: velocity, and u: displacement of the center of a block.

Although the original DDA do not include viscosity term, the viscosity matrix of a body C in Eq. (1) can be rewritten as follows in terms of viscosity η_e and mass matrix M:

$$C = \eta_e M \tag{2}$$

The physical meaning of viscosity η_e is the damping of the rock and rock mass itself and also is extended to other viscous effects.

The kinematic equation of motion Eq. (1) is solved by Newmark's β and γ method by using parameters β = 0.5 and γ = 1.0, and the algebraic equation for the increase in displacement is solved for each time increment by the following three equations (Hatzor *et al.*, 2004):

$$\tilde{K} \cdot \Delta u = \tilde{F} \tag{3}$$

with
$$\tilde{K} = \frac{2}{\Delta t^2} M + \frac{2\eta}{\Delta t} M + \frac{\rho^c}{\rho^0} [K_e + K_s] \tag{4}$$

$$\tilde{F} = a(\frac{2}{\Delta t} M \cdot \dot{u}) + (\Delta F - \sum \int \sigma dv) - M \alpha(t) \tag{5}$$

where, Δu: incremental displacement, K_e: stiffness matrix of linear term, K_s: initial stress matrix caused by rigid rotation, and $\alpha(t)$: time history of earthquake acceleration, ρ^0 is mass density before deformation and ρ^c is mass density after deformation. Usually $\frac{\rho^c}{\rho^0} \cong 1$ in small deformation of rock blocks and K_s is neglected.

2.2. *Discussion on damping effects*

2.2.1. *Parameters for quasi static and dynamic problems*

Here, our discussions focused on Eq. (4) and Eq. (5). The original DDA uses the parameter "a (k01 in other literatures)" in Eq. (5) and it is defined as "a=0" for quasi-static problems and as "a=1.0" for dynamic problems. In the case of earthquake or rock fall dynamic analyses, we empirically have known that the parameter "a" could be set in the range from 0.90 to 0.99 to achieve a stable solution (Tsesarsky, 1994).

As an example, here we assume a=0.95, then,

$$\{\frac{2}{\Delta t^2} M + \frac{0.05 \times 2}{\Delta t} M + [K_e + K_s]\} \cdot \Delta u = 0.95(\frac{2}{\Delta t} M \cdot \dot{u}) + (\Delta F - \sum \int \sigma dv) - M \alpha(t) \tag{6}$$

During discussions among us and Prof. Hatzor, he pointed out that the result of Eq. (6) is equivalent to introducing a 5% viscous damping defined as η_e in Eq. (2). To check this idea, let's make calculation.

$$\{\frac{0.05\times 2}{\Delta t}M\}\cdot\Delta u = 0.05\{\frac{2}{\Delta t}M\}\cdot\Delta u = 0.05(\frac{2}{\Delta t}M\cdot\frac{\Delta t}{2}\dot{u}) = 0.05(M)\cdot\dot{u} \tag{7}$$

As a result, this equation is equivalent to defining $\eta_e = 0.05$ in Eq. (2).

However, the above definition can only be used on a homogeneous model. In inhomogeneous or more complex models, our choice is to use Eq. (4) with $\eta_e = 0.05$.

2.2.2. *Viscous damping*

Eq. (1) include viscous matrix C to represent various damping effects such as damping of the rock and rock mass itself and also air resistance around the rock surface when it flies. Chen *et al.* (Ohnishi *et al.*, 1996) and Shinji *et al.* (Shinji *et al.*, 1997) proposed to use viscous damping to consider energy losses during rock fall. Energy loss may be caused by rock crushing, rock plastic deformation, drug force of grass and woods on the ground, etc. However, there is no general criterion to determine the viscous coefficient. The selection of the coefficient can be done by trial and error procedure or back-analysis of experiments.

2.2.3. *Newmarkalgorithmic damping*

According to the previous studies (Wang *et al.*, 1996), the time integration method in DDA is one of the Newmark-type approaches, and Newmark-type equations can be expressed as follows:

$$\text{Let } A = \ddot{u} \text{ and } V = \dot{u}$$

$$\Delta u^{(n)} = V^{(n)}\Delta t^{(n)} + \frac{(\Delta t^{(n)})^2}{2}(1-2\beta)A^{(n)} + (\Delta t^{(n)})^2\beta A^{(n+1)} \tag{8a}$$

$$V^{(n+1)} = V^{(n)} + (1-\gamma)A^{(n)}\Delta t^{(n)} + \Delta t^{(n)}\gamma A^{(n+1)} \tag{8b}$$

where "A(n + 1)" is the acceleration vector at time step n + 1, the parameters "β" and "γ" define the variation of acceleration over the time step and determine the stability and accuracy characteristics of the method. With the Newmark-type approach, the requirement for unconditional stability is satisfied when $2\beta \geq \gamma \geq 0.5$.

The time integration scheme in DDA introduces $\beta = 0.5$ and $\gamma = 1.0$; this is equivalent to take the acceleration at the end of the time step to be constant over the time step. This approach confirms that DDA approach is implicit and unconditionally stable.

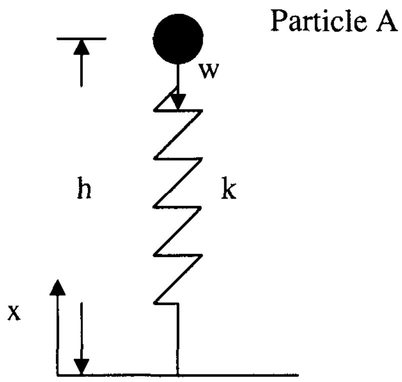
Supposed that one rigid particle with mass "M" and weight "w" is located at the height of "h" above the ground level, and the spring with stiffness "k" always connects the particle to the ground shown in Figure 2. The initial velocity of the problem is 0. No damper is concerned in the system, and the time integration method to calculate the particle behavior is the same as DDA. With the gravity load, the particle should do SHM (simple harmonic motion) from its equilibrium position without any energy losses. If –x0 is the equilibrium extension at which the force of spring and gravity balance, then

$$w = -Mg = kx_0 \tag{9}$$

For any extension x, the net force on the falling particle is

$$F = -Mg - kx = -k(x - x_0) = -kx' \tag{10}$$

where, $x'=x-x_0$ is the displacement from the equilibrium position.



According to Newton's second law (F=MA) applied to the particle, which means

$$A = -\frac{k}{M}x' \tag{11}$$

Since the acceleration $A=d^2x'/dt^2$, equation (11) can be rewritten as

$$\frac{d^2x'}{dt^2} + \frac{k}{M}x' = 0 \tag{12}$$

Fig. 2. Simple Particle-Spring Connecting System.

Equation (12) is the same form as the simple harmonic motion with an angular frequency

$$\omega = \sqrt{\frac{k}{M}} \tag{13}$$

Hence, the displacement from equilibrium can be given as

$$x(t) = (Amp)\sin(\omega t + \phi) = (h + x_0)\sin(\omega t + \phi) \tag{14}$$

where "Amp" is the amplitude of the oscillation, and "ϕ" is the phase constant.

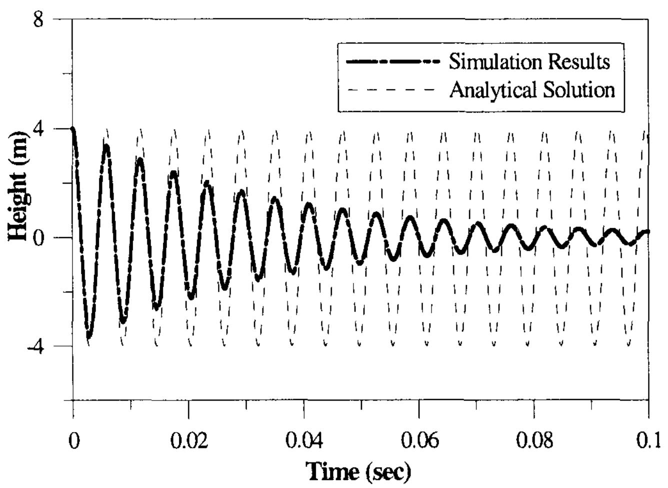


Fig. 3. Simulation Results with Time Delta Equals to 0.0001 sec.

By introducing the numerical method analyzing the same problem with weight (w) equals to 26.0 kN, the height (h) is 4 m, the spring stiffness (k) is 3000 MN/m, and the time interval equals to 0.0001 sec, the simulation result in Figure 3 shows the particle track with "damping", and the total potential energy (including, potential energy, kinetic energy and the spring energy) decreases. In addition, the total energy decreases exponentially with time, and the relation between total energy and the time can be expressed as follow:

$$TotalEnergy = E_0 \cdot e^{-\lambda t} \tag{15}$$

where, "E_0" is the initial total energy, "t" is the time, and "λ" is the coefficient of damping .

As shown in Fig. 4, it is clear that the "coefficient of damping" is related to not only the spring stiffness but also the time interval (Wu, 2003). These results indicate the existence of algorithmic damping in the numerical analysis when the system is "particle-spring connected", which was pointed out by many researchers (Ishikawa *et al.*, 2003). In addition, the coefficient of damping increases with the time increment and spring stiffness in the computations.

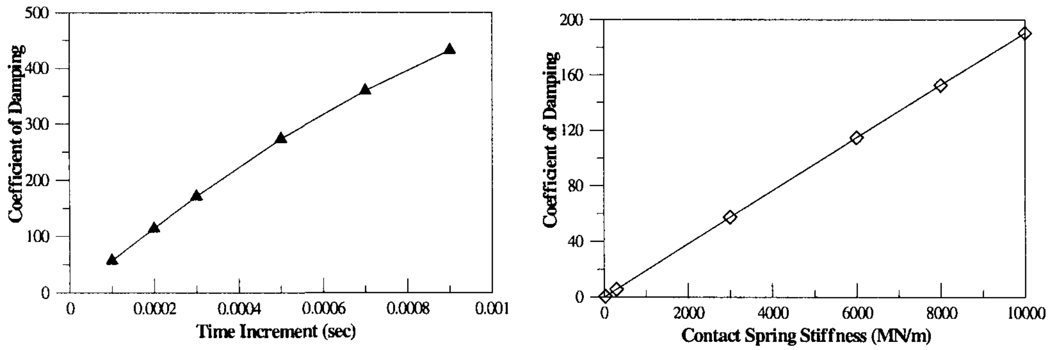


Fig. 4. Relationship between Coefficient of Damping and Time Increment Contact Spring Stiffness.

2.3. *Characteristics of the numerical stability of kinematic equation matrices (Effect of time increment)*

We discuss the characteristics of the numerical stability of the global matrix as Eq. (4).

$$\tilde{K} = \frac{2}{\Delta t^2}M + \frac{2\eta}{\Delta t}M + \frac{\rho^c}{\rho^0}[K_e + K_s] \tag{16}$$

The characteristics of matrix K are expressed as an Eigen problem:

$$[(\frac{2}{\Delta t^2}M + \frac{2\eta}{\Delta t}M + \frac{\rho^c}{\rho^0}[K_e + K_s]) - \lambda I]\{\phi\} = \lambda\{\phi\} \tag{17}$$

where, λ : Eigen value of the matrix [A], { ϕ }: Eigen vectors.

The solution condition for the calculation except zero solution is defined by:

$$\det[(\frac{2}{\Delta t^2}M + \frac{2\eta}{\Delta t}M + \frac{\rho^c}{\rho^0}[K_e + K_s]) - \lambda I] = 0 \tag{18}$$

The inverse matrix of K is expressed by Eigen values and Eigen vectors.

$$[\frac{2}{\Delta t^2}M+\frac{2\eta}{\Delta t}M+\frac{\rho^c}{\rho^0}[K_e+K_s]]^{-1}=\sum_{i=1}^{n}(\frac{1}{\lambda_i})\phi_i\phi_i^T \tag{19}$$

The cancellation of significant digits depends on the spectral radius as the condition number of matrix [A] defined in Eq. (20).

$$Cond[A]=\frac{\lambda_{\max}}{\lambda_{\min}} \tag{20}$$

where, $\lambda_{\max}$: the maximum Eigen value of the matrix [A], $\lambda_{\min}$: the minimum Eigen value of the matrix [A].

In the case of a condition number under 1,000, the numerical solution is stable in experience. Poor conditions of the matrix K depend on the time increment.

$$[\frac{2}{\Delta t^2}M+\frac{2\eta}{\Delta t}M+\frac{\rho^c}{\rho^0}[K_e+K_s]]=[A] \tag{21}$$

The existence condition of a solution to matrix [A] is positive definite and the condition is defined by Eq. (22).

$$\det[A]\neq 0 \tag{22}$$

One factor inducing instability in matrix [A] is selecting a very small time increment Δt for instance. As a result, the first term on the left hand side of Eq. (21) becomes disproportionately large while the third term on the right hand side of Eq. (21) becomes relatively small, and the determinant of the matrix [A] is close to zero. Thereupon, the numerical condition of matrix [A] becomes unstable as shown in Eq. (23).

$$\frac{\rho^c}{\rho^0}[K_e+K_s]\leq\frac{2}{\Delta t^2}M+\frac{2\eta}{\Delta t}M \tag{23}$$

For simplicity, we assume the stiffness matrix as follows:

$$\frac{\rho^c}{\rho^0}=1.0,\ \ [K_e+K_s]\cong[K] \tag{24}$$

Eq. (23) is given by:

$$[K]\leq\frac{2}{\Delta t^2}M \tag{25}$$

As the conditions of matrix [A] become close to Eq. (25), the potential for numerical instability increases. For simplicity, if Eq. (25) is shown one dimensionally as Eq. (26), the time increment of the lower limit can be estimated.

$$\Delta t \le \sqrt{\frac{2M}{K}} \tag{26}$$

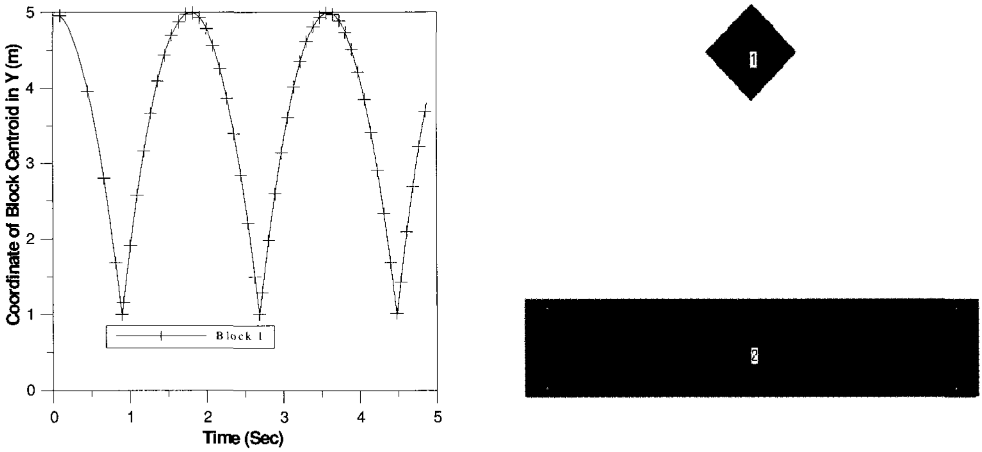


Fig. 5. Block drop and bouncing analysis.

If we use a smaller time increment than Eq. (26), the characteristics of the stiffness matrix [K] are loosened and the solutions become unstable. For example, if we assume the rocks elastic modulus is 1 GPa (1,000,000 kN/m^2), the area of the block is 1 m^2, the unit mass is 25 kN/m^3, then the time increment is calculated by Eq. (27).

$$\Delta t \le \sqrt{\frac{2\times 25}{1{,}000{,}000}} = 0.007 \tag{27}$$

The result of Eq. (27) shows that the chosen time increment might affect the optimal value. Hence, the value of the chosen time increment should not be extremely small.

In the discussion with Dr. Wu (Wu, 2003), he pointed out that the algorithmic damping in DDA diminishes as shown in Figure 5 if he takes a very small amount of time increment. However, as indicated in the Eqs. (26) and (27), too small time increment will cancel the existence of M in Eq. (21) and consequently the algorithmic damping effect disappear. As a conclusion, we can only choose a time increment in a certain range and can not avoid the algorithmic damping if we use Newmark method with $\beta = 0.5$ and $\gamma = 1.0$.

2.4. *Parameters for block contact (selection of the optimal penalty value)*

The potential energy of the contact between discontinuous planes, and is evaluated by the least squares method by using a penalty as follows:

$$\Pi_{PL}^{i,j} = \frac{1}{2}k_N[(u^j - u^i)\cdot n]^2 - \frac{1}{2}k_T[u_T^j - u_T^i]^2 \tag{28}$$

where, k_N: penalty coefficient in the normal direction, k_T: penalty coefficient in the shear direction, a $(u^j - u^i)\cdot n$:mount of penetration in the normal direction, u_T: amount of slip in the shear direction, and n: direction cosine of the contact plane. The global stiffness matrix with Penalty is expressed by:

$$\begin{bmatrix} K_{11} & \cdots & \cdots & \cdots & \cdots & K_{1n} \\ \vdots & \ddots & & & & \vdots \\ \vdots & & (K_{ii}+K_N+K_T) & \cdots & (K_{ij}-K_N-K_T) & \vdots \\ \vdots & & \vdots & \ddots & \vdots & \vdots \\ \vdots & & (K_{ji}-K_N-K_T) & \cdots & (K_{jj}+K_N+K_T) & \vdots \\ K_{n1} & \cdots & \cdots & \cdots & \cdots & K_{nn} \end{bmatrix} \begin{Bmatrix} u_1 \\ \vdots \\ u_i \\ \vdots \\ u_j \\ u_n \end{Bmatrix} = \begin{Bmatrix} f_1 \\ \vdots \\ f_i+(u_i-u_j)(K_N+K_T) \\ \vdots \\ f_j+(u_j-u_i)(K_N+K_T) \\ f_n \end{Bmatrix} \tag{29}$$

In the case of dynamic analysis, especially for rock fall problems, the reaction forces of contact blocks are defined on the right hand side of Eq. (29). If we choose a large value for the penalty coefficient, the reaction force becomes very large, and then the rebound motion of the block is also very large. Accordingly, the penalty coefficient might be chosen as the same value as the elastic modulus of the penetrated block side, which would result in a very large value as defined in Eq. (30).

$$\text{(Elastic modulus)} \times \text{(block area)} / \text{(penalty coefficient)} \tag{30}$$

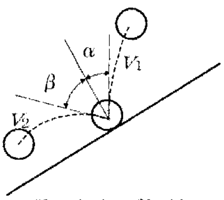
In the event that the value of Eq. (30) is less than 1/1000, this condition is equivalent to a small block or a small elastic modulus of the materials. In this state, the cancellation of significant digits results on the left hand side of Eq. (29) between the block stiffness and penalty, hence, the characteristics of the block stiffness are dispersed and the solution diverges. Therefore, we should also select the optimal penalty value.

2.5. *Restitution coefficient (bouncing phenomenon)*

It is important to have the best possible knowledge of rock-fall trajectories and energies in order to determine accurate risk zoning and construct adequate defense structures near the threatened area (Fukawa *et al.*, 2005 and Yang *et al.*, 2004). The ground's restitution coefficient is evaluated according to the ration between the total energy of the falling block before and after its impact on the ground.

Motion of a rock at bouncing is mainly observed to analyze restitution (repulsive) coefficients and velocity energy ratio. In DDA rock-fall simulation, mass density, elastic modulus and Poisson's ratio are used as input parameters. In addition, viscous damping coefficient and velocity energy ratio to control the bouncing motion and rolling of rock-fall are required in DDA. Energies are assessed by measuring both rotational and translational block velocities, slope inclination and sizes of the block at impact.

When a block drops on a ground, it bounces and settles with energy loss. This energy loss phenomenon can be modeled by Restitution coefficient (Rn: normal direction, Rt: tangential) and velocity energy ratio (γ) as shown in Figure 6 that are defined on the basis of the repulsion between falling rock and slopes. Since the velocity is a vector, it is direction dependent. As discussed above (Section 2.2.), this energy loss may be modeled by viscous effects.



V_1:velocity of incidence
V_2:velocity of reflection
α:angle of incidence
β:angle of reflection

$$\gamma = \frac{1/2\, mV_2^2}{1/2\, mV_1^2} = \frac{V_2^2}{V_1^2}$$

$$Rn = \frac{V_2 \cos\beta}{V_1 \cos\alpha}$$

$$Rt = \frac{V_2 \sin\beta}{V_1 \sin\alpha}$$

Fig. 6. Definitions for velocity energy ratio.

Many experiments of rock-fall in-situ have been reported and their parameters have changed on each site (Ohnishi *et al.*, 1994). The magnitudes of change for this parameter have yet been unknown. We still are under investigation with recorded data for falling rocks in field experiments (Hagiwara *et al.*, 2004, Ma *et al.*, 2004, Sasaki *et al.*, 2002, Sasaki *et al.*, 2004, Shimauchi *et al.*, 2004 and Wu *et al.*, 2005).

2.6. *(Additional discussion) viscous damping at block contact*

It is an important issue to discuss whether DDA can simulate earthquake response of (rock) structures like as FEM. In FEM analysis it is conventional to prevent the reflection of seismic waves from the boundary to the soil mass in the analysis area by using dash-pot damping elements as shown in Figure 7. We therefore introduce Voigt-type viscous damping elements between blocks as shown in Figure 8.

DDA was formulated by defining penalty parameters for the normal direction K_n and the shear direction K_s. we also introduced parameters for the normal damping η_{pn} and the shear damping η_{ps} based on the original DDA theory. In the case of Voigt-type damping, the penetration d and the velocity strain $d/\Delta t$ are expressed by:

$$f_t = f_p + f_\eta = pd + \eta_p \cdot \dot{d} = pd + \eta_p \frac{d}{\Delta t} \tag{31}$$

The total reaction force of the penetration f_t is expressed as the summation of the force by penalty f_p and the force by dash pot f_η as follows:

$$f_t = f_p + f_\eta = pd + \frac{\eta_p}{\Delta t} d = (p + \frac{\eta_p}{\Delta t})d \tag{32}$$

The potential energy of the penetration between blocks considering contact viscous damping is expressed by:

$$\Pi_{p\eta} = f_t d = \frac{1}{2}(p + \frac{\eta_p}{\Delta t})d^2 \tag{33}$$

As a result, the penalty value is expressed by the function of the penalty, the viscose dumping coefficient, the penetration displacement and time increment as shown in Eq. (33). This formula automatically introduces the influence of the velocity of penetration, and this scheme can save the surplus penetration.

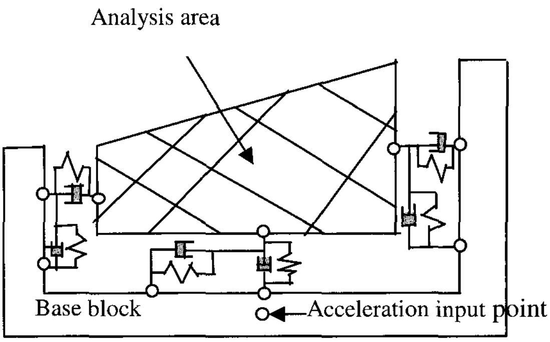


Fig. 7. Boundary conditions for earthquake response analysis.

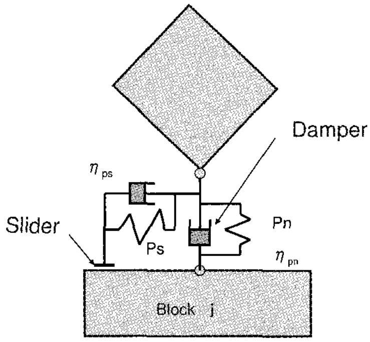


Fig. 8. Voigt-type viscous damping of friction.

3. Conclusion

The Japanese research group has been working on studies of the DDA and Manifold method in terms of its application to actual problems since 1991. Some of the discussion results in our group were shown in this paper. Many problems are still left unsolved when we try to use DDA in practical application.

Here, we introduced a discussion of the parameters for calculation with a focus on damping effects, selection of the optimal penalty value, range of time increment, understanding of bouncing or collision energy loss, and viscous boundary. Many practical problems are examined by Japanese research groups in view of these parameters. Yet we can not find the best estimate of DDA parameters to be used in calculation. We should be careful that DDA must be used to keep in mind the characteristic behavior of DDA.

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THE ROLE OF ON-SITE ENGINEERING IN UNDERGROUND PROJECTS

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In the planning and design phases of underground structures, the information on geological setup, the rock mass structure and characteristics necessarily is incomplete and inaccurate. To allow for a safe and economical construction, a continuous updating of the ground model and an adjustment of the construction methods to the actual site conditions is required. For a smooth construction process, the conditions ahead of the face have to be predicted, and the ground surrounding the tunnel characterized. Based on this updated model the ground behaviour can be assessed, and the final layout of the construction determined. The expected interaction between ground and support (system behaviour) forms the basis for the monitoring program and the safety management plan, which includes warning and alarm criteria.

As all decisions on site have to be made quickly, data acquisition, processing and analysis have to be well organized. Highly qualified and experienced geotechnical personnel, as well as appropriate site organization and contractual conditions are required to allow for a short reaction time to changing conditions.

A number of tools and methods have been developed, which contribute to a more reliable assessment of rock mass structure and behaviour, which again enables a more precise determination of excavation and support methods. Digital stereo photos allow a precise evaluation of the rock mass structure, while advanced software for the evaluation of displacement monitoring data and prediction of displacements assists in predicting and controlling the performance of the underground structure. Up to date methods of monitoring data evaluation and interpretation will be demonstrated with the help of case histories.

A key issue is the accurate prediction of the displacements in their development and final magnitude. Appropriate software can support engineers in assessing displacements and stresses of tunnel supports.

The experience of on-site personnel in general is limited, and may not cover specific problems encountered on site. In the past experts had to be brought to the site and briefed on the conditions to solve such problems. This is slow and inefficient, as the level of information might be not sufficient or time consuming to upgrade. With the Internet nowadays an exchange of information is easy, allowing experts to give a profound advice, even if they are not on site.

Keywords: Tunnel design, monitoring, face mapping, on-site-engineering.

1. Introduction

Even with a good geological and geotechnical investigation and an up to date design, the adjustment of excavation and support to the local conditions has to be done on site in order to achieve an economical and safe tunnel construction. The uncertainties in the ground model increase with increased overburden and the complexity of the geological conditions. Considerable effort and expertise is required to continuously update the ground model, predict ground conditions ahead of the face, identify possible failure modes, determine appropriate excavation and support methods, and predict and verify the system behavior. The increased information gathered during construction allows a more precise ground characterization, and thus an optimal adjustment of the construction method to the ground behavior and required system behavior.

Many serious problems during tunneling arise from so called unexpected geological conditions. This may involve the late detection of faults and fault zones, but also the inflow of ground water. To minimize damages and losses due to such conditions, efficient and continuous site engineering is required.

To allow successful implementation of an observational approach, several technical and organizational conditions have to be fulfilled. First of all consistent procedures for the investigation, ground characterization, and design have to be devised (OeGG, 2001). Then the range of possible ground behaviors have to be assessed and the limits of acceptable system behaviors defined. On site the implementation of a monitoring program targeted to the expected behavior must be implemented in an appropriate density. Processing, evaluation and interpretation of monitoring results has to be done sufficiently rapidly to allow mitigation measures to be implemented in time. Last, but not least, the site organization shall allow an efficient decision making process and a rapid implementation of required measures.

2. The Role of On-Site Engineering

Although the general nature of the ground may be known prior to construction, an accurate prediction of the internal structure is impossible. Thus the ground model has to be continuously improved during construction. Monitoring and data collection have to focus on the specific problems associated with project. An important part of the on site activities of geologists and geotechnical engineers is the prediction of the ground conditions in a representative volume ahead of the tunnel face and around the tunnel. Only if a relatively accurate model exists, can appropriate excavation and support methods be selected, and the expected system behavior predicted. A second very important task is the monitoring of the system behavior after excavation, and the assessment of its compliance with the prediction.

2.1. *Geological tasks*

On many sites the site geologist is used only to document the actual geological conditions. This usually is done in the form of face maps, with a later compilation into longitudinal sections and plan views. This may help in defending or supporting claims, but is not sufficient to allow for a reasonable adaptation of the construction to the actual conditions. To fulfill the requirements of an observational approach, the geologist has to continuously update the geological model, incorporating the observations on site. As many of the decisions during construction have to be made prior to the excavation, like round length, overexcavation, lining thickness, etc. the geologist also has to predict the ground conditions ahead of the face and in a representative volume around the tunnel. To enhance the accuracy of the prediction, a continuous observation of trends of certain parameters is required. For efficient data management and evaluation data base systems with advanced statistical and probabilistic features can be used (Liu et al. 1999).

2.2. *Geotechnical tasks*

The information gathered by the site geologist is further processed by the geotechnical engineer, forming the basis for decisions on construction method, monitoring layout and reading frequency, to name a few tasks only. To allow decisions to be taken in time, all data recording and evaluation has to be done quasi in real time, and the relevant data have to be always available to all parties involved in the construction. Internet based information platforms can be used for that purpose, allowing also off-site experts to keep track with the information flow. Based on the geological model, the geotechnical engineer has to update the ground model by assigning properties to the geological features. Then the ground behavior (ground reaction on excavation without support) for the section ahead is evaluated, possible failure modes identified, and excavation and support methods assigned. To support the geotechnical modeling, the monitoring results of the previously excavated sections can be used. In a next step, the system behavior (combined behavior from ground and construction measures) is predicted and compared to the requirements, like serviceability, compliance with limitations (subsidence, vibrations, etc). Based on the recommendation of the geotechnical engineer the Engineer under consideration of contractual aspects fixes the construction measures. In case those deviate from those recommended by the geotechnical engineer, the expected system behavior has to be re-evaluated.

The geotechnical engineer also has to determine the monitoring layout and program, which should be targeted to capture the expected behavior. Once the expected system behavior is determined and the monitoring conducted, the observed behavior is compared to the predicted one. Deviations from the normal or predicted behavior have to be assessed, and in case of unacceptable developments mitigation measures proposed. Warning and alarm criteria and respective mitigation measures are laid down in a geotechnical safety management plan, jointly developed by the designer and geotechnical engineer on site.

3. Tools to Assist in Data Collection and Evaluation

3.1. *Prediction of ground conditions*

Predicting the ground conditions can be separated into two parts, the geological modeling and the geotechnical prediction, which is mainly based on evaluating and interpreting displacement monitoring data. A close co-operation between the disciplines is required to be able to produce a reliable prediction of the ground conditions ahead.

Basis for the geological modeling ahead of the face in general is the observation of trends of structures, recorded in the excavated section. Traditional manual face mapping increasingly is supported by up to date 3D image systems (Gaich et al. 2004, 2005). Figure 1 shows an example of a 3D model of a tunnel face with measurements of discontinuity orientations taken from the

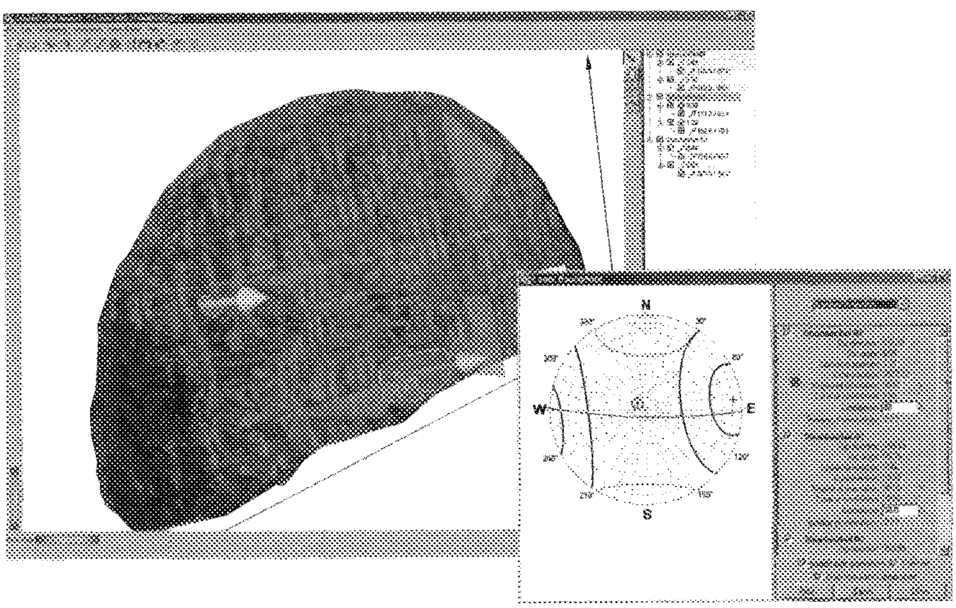

Fig. 1. 3D image of a tunnel face with measurements of discontinuity positions and orientations, and statistical evaluation of discontinuity data with JointMetrix3D Analyst.

image. In this way an unbiased evaluation of the geological situation is possible. In contrast to hand sketches, and discontinuity orientation measurements with the compass, with the images the information is complete and accurate, as the images are calibrated and metric. Orientations of joints can be measured from joint planes or joint traces. The evaluation software also offers options to measure bedding thicknesses, joint bridges, areas and distances.

A series of evaluated images of successive excavation faces allows predicting the rock mass structure and quality of a representative volume ahead of the face and around the tunnel by extrapolation. This, combined with the structural geological evaluation can lead to a pretty reliable geological model, forming the basis for predicting ground behavior, which again is the basis for the determination of required construction measures.

3.2. *Advanced analysis of displacement monitoring data*

The geological modeling preferably is supplemented by an analysis of the monitored displacements. It has been shown, that the trend of the spatial orientation of the monitored displacement vector can be used to identify changes in the rock mass quality ahead of the tunnel face (Schubert et al. 1995, Steindorfer 1998, Grossauer et al. 2003). Figure 2 shows the results of a series of numerical simulations, where the development of the stresses, displacements, and displacement vector orientations for a tunnel crossing a weak zone are shown.

Figure 3 shows an example of an Alpine tunnel, where the displacement vector orientation trend (L/S) significantly changes already when the face is several tens of meters ahead of a fault zone. At this project, the normal displacement vector orientation in quasi homogeneous ground was in the range of 4° to 9° against the direction of the advance. From about station 1100m a deviation of the vector orientation from the normal range can be observed. The peak of the deviation is reached right at the transition between sound rock and fault zone. With further progress of the excavation through the fault zone, the trend of the displacement vector orientation first tends to the normal range again. When the heading is within the fault zone, the trend of the displacement vector orientation deviates to the opposite side of the normal range, indicating the stiffer rock mass behind the fault zone. For extended fault zones, the displacement vector orientation generally returns to the "normal" range again, until the influence of the boundary to the more competent rock mass is indicated by another deviation. This information can be used to estimate fault zone extensions.

As a general rule it can be stated, that the higher the stiffness contrast between faulted rock and neighboring rock mass, and the longer the fault zone is, the larger is the deviation of the displacement vector orientation from the normal range. It has been shown by Grossauer (2001), that this is valid up to a certain critical length of a fault zone. As for fault zones with an extent of less than about three to five tunnel diameters, a certain arching between the more competent rock masses can be observed, also the displacement magnitude within the fault zone is smaller than in a fault zone with a large extension.

Displaying the spatial displacement vector orientation in stereographic projection, the orientation of faults outside the tunnel profile can be determined with some accuracy. Naturally also the virgin stress field and anisotropy of the rock mass influence the displacement vector orientation. Thus for each project the range of "normal" displacement vector orientation will be somewhat different.

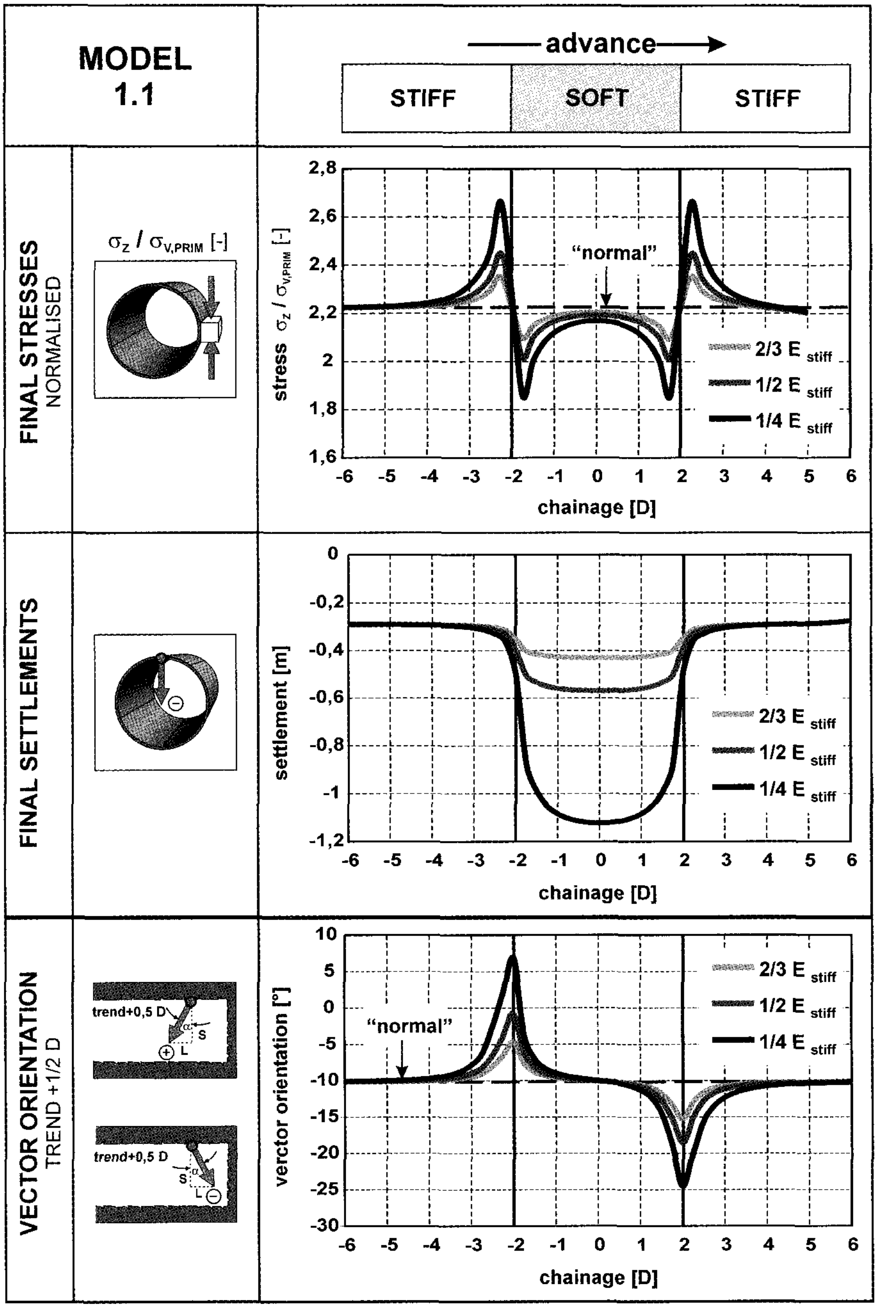

Fig. 2. Distribution of stresses along the sidewall of a tunnel when the tunnel penetrates a weak zone (upper). Development of displacements, when the tunnel penetrates a weak zone (center). Trend of displacement vector orientation (lower) shows a clear deviation from its normal when the heading approaches a zone of different stiffness already a few tunnel diameters ahead of the transition. The different lines show the influence of different stiffness contrasts between soft and stiff rock (Grossauer, 2001).

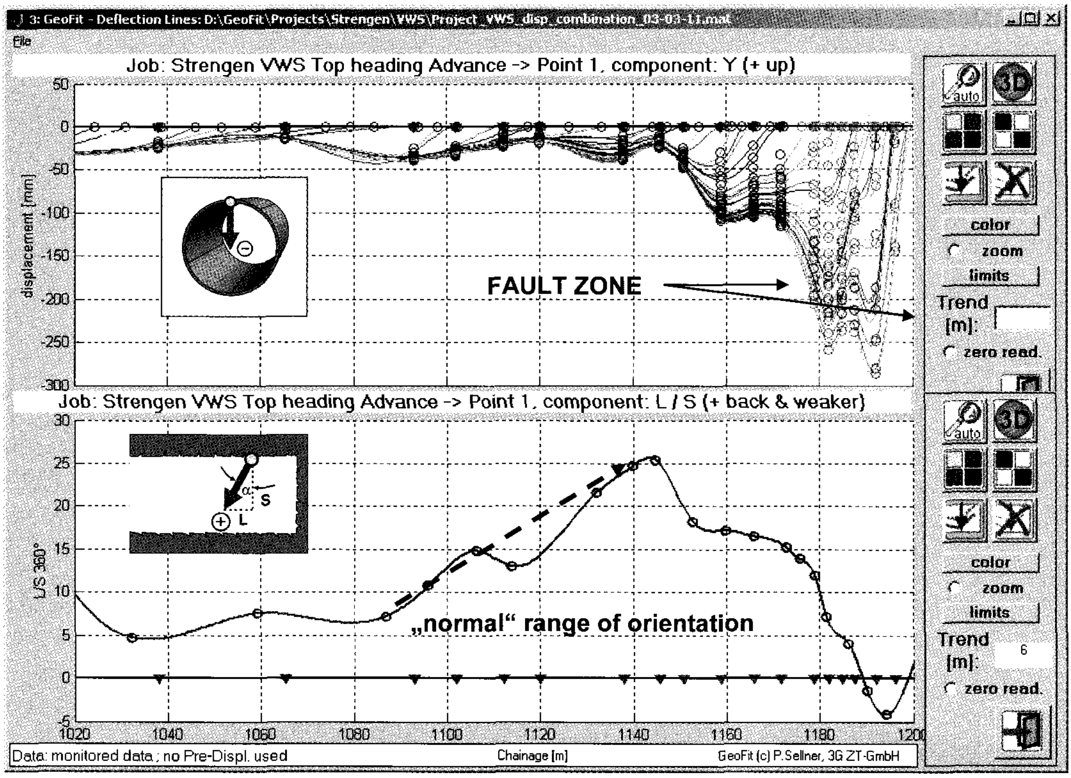

Fig. 3. Deviation of the displacement vector orientation from the “normal” several diameters ahead of a fault zone. When the excavation passes the fault zone the deviation in the opposite direction indicates stiff rock mass ahead again.

3.3. *Prediction of displacements*

Once the geological-geotechnical model for the region ahead of the face has been established, the support and excavation measures can be determined, and the expected displacement development predicted. Sellner (2000), based on research conducted by Sulem et al. (1987) developed software (GeoFit®) allowing the prediction of displacements considering varying excavation sequences, advance rates and different supports. With this tool it is not only possible to predict the development of the displacements and the final displacement magnitude, but also the effect of different supports. Figure 4 shows such a prediction of the displacement development for a shallow tunnel in a tectonic mélange. The excavation was done in a top heading-bench-invert sequence. The shotcrete-rock bolt support is supplemented by a temporary shotcrete invert in the top heading. Based on an assumed construction progress, the development of the displacements for the top heading without temporary invert is predicted (dashed line). Then the temporary invert is added, showing in a decrease of displacements. In a third step the additional displacement caused by the bench and invert excavation are predicted. This approach allows an assessment of the effectiveness of various support types and the influence of the construction sequence on the development of displacements in a very early stage. With some experience, the displacement development can be predicted already a couple of hours after excavation, if readings are taken in a sufficiently short interval. If it for example shows, that displacements would be in an unacceptable range, support can be increased, and the efficiency immediately simulated.

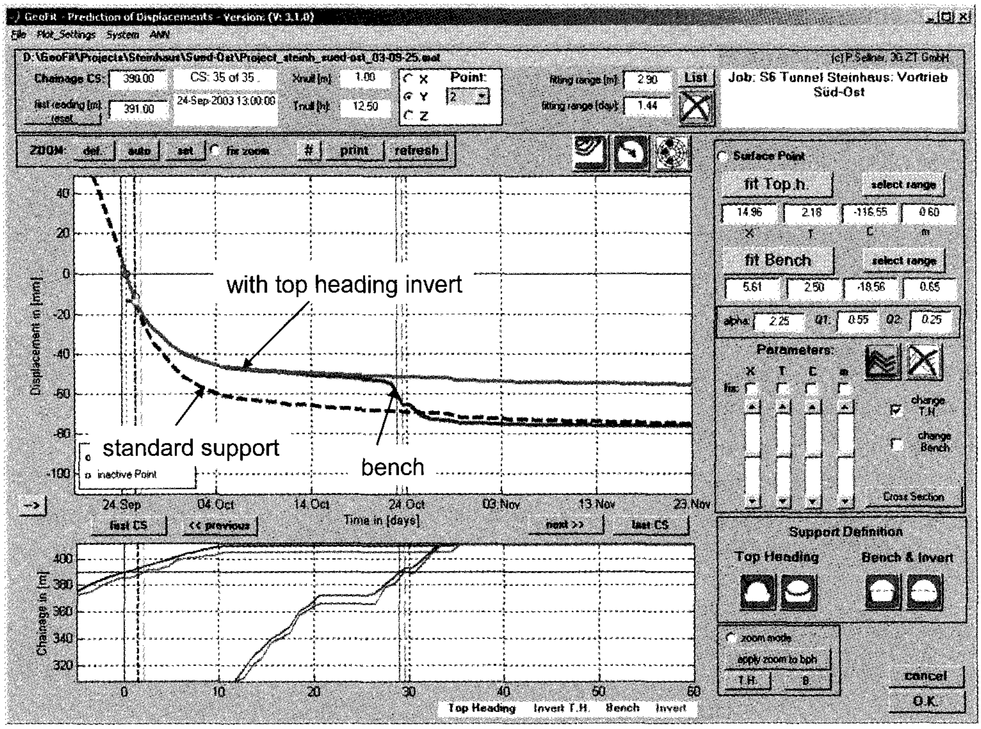

Fig. 4. Predicted development of the displacements for a top heading bench invert excavation. In the top heading a temporary invert is installed.

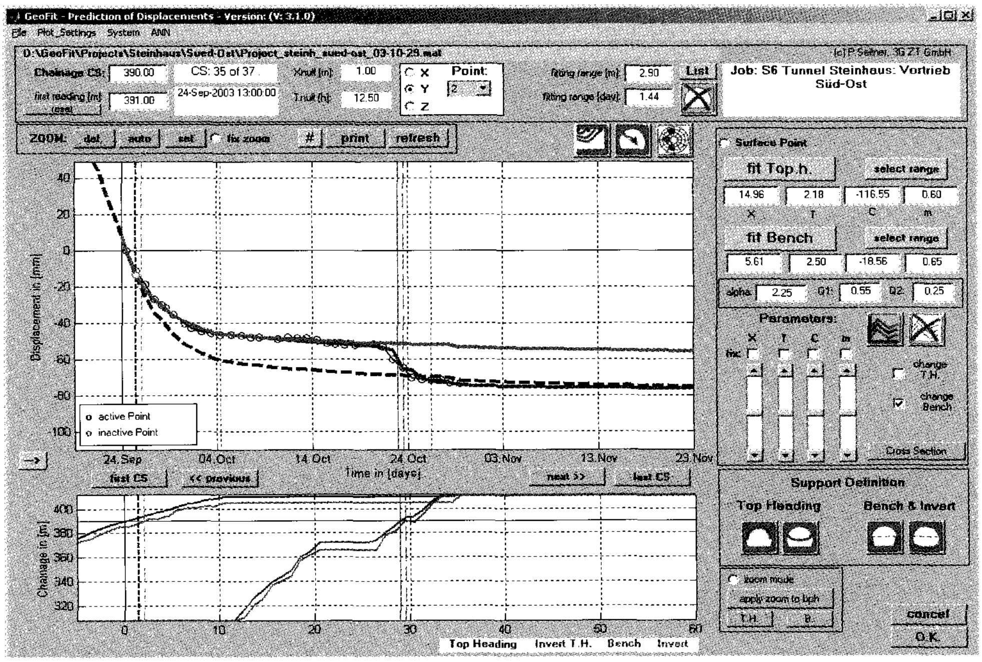

Fig. 5. Comparison of the measured displacements (circles) to the predicted ones (full lines)

During excavation, the measured displacements can be compared to the predicted ones (Figure 5). The big advantage of such a tool compared to traditional plots of the displacement history only is the fact that also for unsteady advance a clear assessment of the normality of the system behavior is possible.

It has been shown in many applications that the empirical formulations used in GeoFit® for the prediction of displacements very well reflect the ground reaction. Deviations from the predicted displacement development thus can be attributed to unusual behavior or failure in the ground – lining-system. A reasonable application of the software however needs quite some experience.

The previous example shows an excavation with a very steady advance rate. In such cases the interpretation of displacement history diagrams is pretty simple, as the displacement rate should continuously decrease with increasing distance between face and measuring section. More challenging is the assessment of the normality of the system behavior in case of an unsteady advance. Figure 6 shows an example, where a weak zone in the ground to the left of the tunnel led to overstressing, showing in a pronounced deviation from the predicted displacement development. If the comparison between predicted and measured displacements is done routinely, such deviations can be easily detected, and mitigation measures implemented in time.

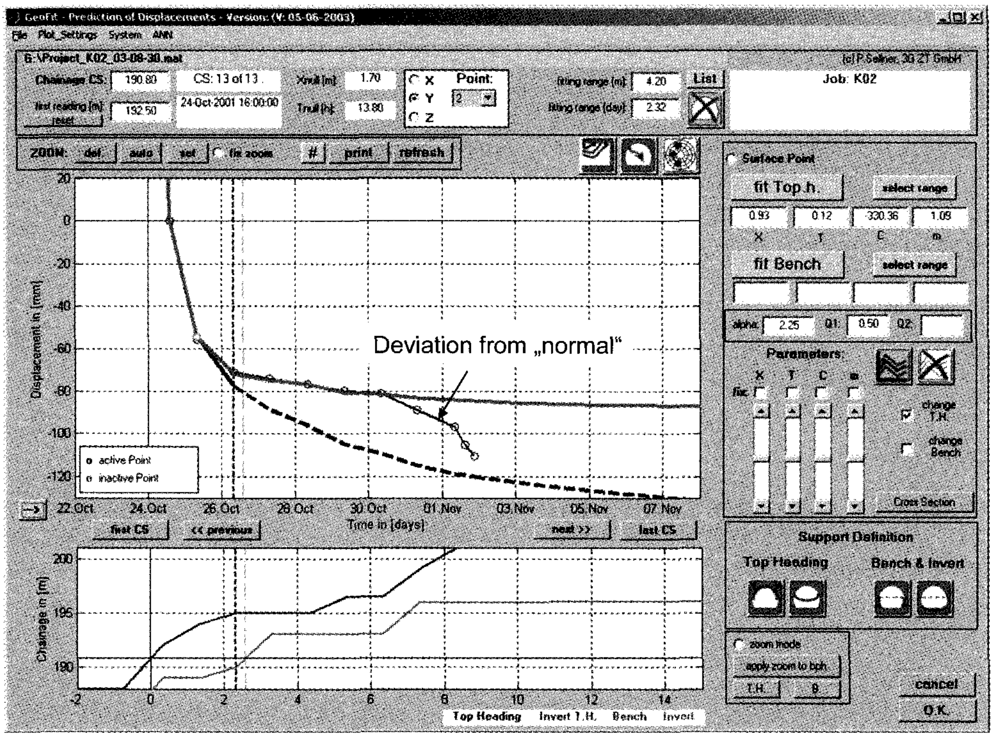

Fig. 6. Deviation from the predicted "normal" behavior, indicating destabilization of the tunnel.

In addition the prediction is used to determine the required overexcavation to allow for the expected displacement without impairing the clearance after stabilization of the tunnel. This in particular is of importance when tunneling in fault zones, as the final displacement magnitude may vary in a wide range, depending on the rock mass structure and quality, and it is well known, that reshaping is extremely expensive.

3.4. *Check of lining stresses*

For shallow tunnels in urban areas usually a pretty stiff lining is used to minimize ground deformations. As such linings tend to fail in a brittle mode at low levels of deformation the evaluation of the displacements only does not provide a reliable indication of the state of stress. The results of 3D optical monitoring can be used to evaluate the strain development. With an appropriate material model, considering time dependent hardening and strength development, as well as the effects of shrink, temperature and creep, the actual stress level in the lining can be evaluated (Schubert, P. 1988, Aldrian 1999, Rokahr *et a*l. 2002, Hellmich *et al.* 1999, Macht 2002, Tunnel:Monitor, 2006).

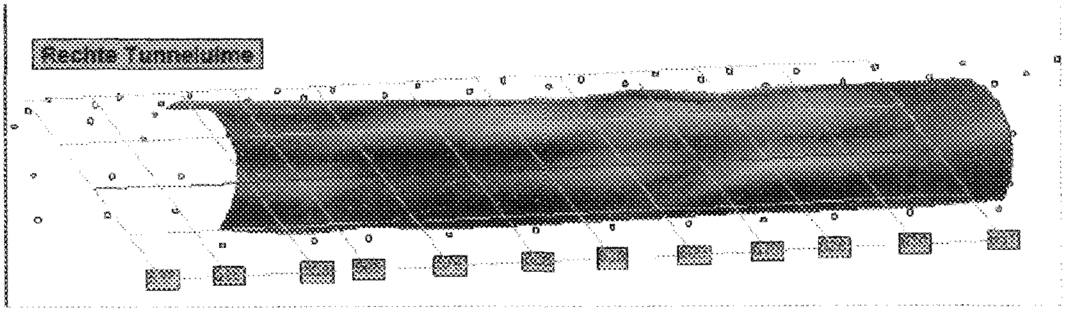

Fig. 7. Evaluated stress level in shotcrete lining with the software Tunnel:Monitor

The evaluation of stresses in the lining not only can be done after monitoring results are available (Figure 7), but can be predicted on the basis of the predicted development of the displacements.

In tunnels with high overburden and weak ground, the deformability of the lining in many cases is not sufficient to cope with the displacements without failure. Checking the expected lining stresses on the basis of the expected development of the displacements can help in making the right decisions for excavation and support. While in some cases, where the expected stresses in the lining only slightly exceed the strength, a reduction in progress rate might give the shotcrete enough time to develop strength, in other cases ductile linings could be required (Moritz, 1999, Button et al. 2003).

Figure 8 shows an example of the evaluation of a shotcrete lining utilization for an expected development of displacements. Due to the low strength of the shotcrete at an early age, the stresses would exceed the strength after one day (upper diagram). Thus either a reduction in excavation rate, or a change to a ductile lining are required. As a reduction in excavation rate will be acceptable only in cases of short sections of weaker ground, the change to ductile supports will be the reasonable measure for longer sections. With the incorporation of ductile support elements into the lining, the stress intensity is re-evaluated. The lower diagram in figure 8 shows the development of the utilization for the ductile lining. The maximum utilization rate is only 50% of the lining capacity.

The combination of comparing predicted to the measured displacements, and the check of the lining utilization considerably contributes to a reduction of "surprises" during tunneling.

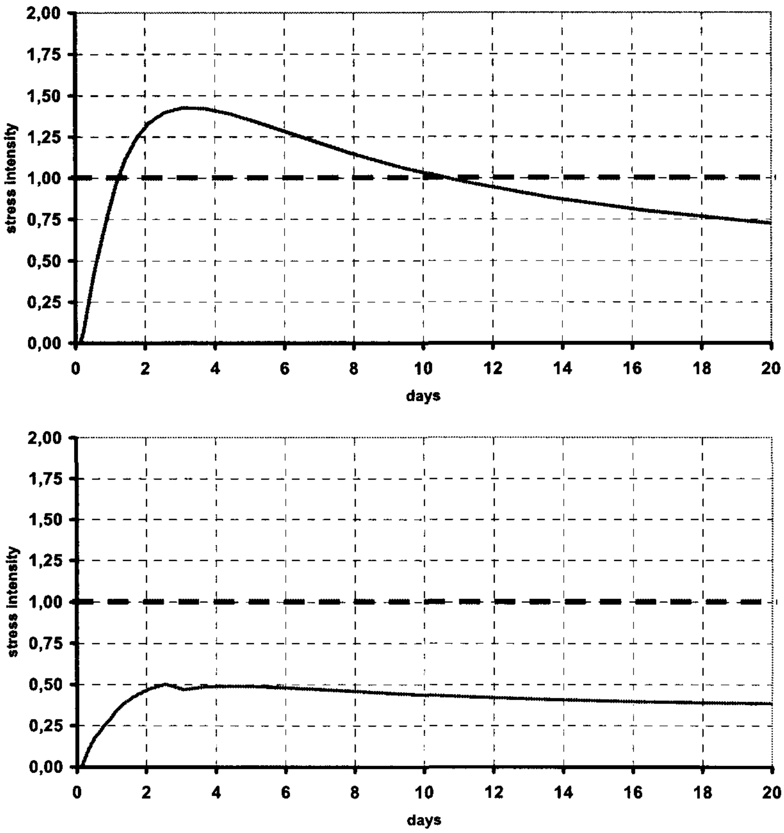

Fig. 8. Development of lining utilization factor for a stiff lining, using predicted displacement development (upper diagram), and lining utilization for a ductile lining (lower diagram)

4. The Role of Off-Site Experts

In most cases it is impossible to maintain appropriately experienced staff on site to cope with all expected and unexpected scenarios. For an optimal construction the involvement of experts is required. This involvement may be necessary only for short periods or over the whole construction time, depending on the complexity of the ground conditions, and the expertise available on site. The traditional way of acquiring external expertise is to call in an expert to the site, brief him on the situation, and expect a sound advice within hours. This procedure is not only time consuming and expensive, but also inefficient, as even nowadays nobody carries all the supporting hard- and software around the world. In addition the appropriate expert might be unable to allocate the time to go to a site being far away from his office.

The solution to this problem is to allocate appropriate experts for the problems expected which support the on-site staff from their offices in case needed. Data exchange and information is done via Internet.

4.1. *Requirements*

It is recommendable that already in the pre-construction phase the respective expert(s) are contacted and made familiar with the project. This will generate some costs, but is a good investment, as a familiarization during the construction inevitably leads to delays and/or lower quality advice.

Depending on the problems expected during construction, the type of data, their quality and quantity need to be determined. In general this will be geological data from face mapping and on-site modeling, data on excavation and support, and in particular displacement monitoring results. Photos can supplement the information. As the experts working off site cannot acquire a personal impression and thus lack a bit a "feeling" for the ground conditions, the data collected and transmitted must be objective, complete, and of high quality.

The format of the data has to be agreed upon to allow a processing in the office of the expert(s). Preferably the data are uploaded to a server, to which the expert(s) have access at any time.

4.2. *Co-ordination with site staff*

For a successful co-operation between the expert(s) and the on-site geological - geotechnical staff a start up meeting is very useful. In this meeting the site conditions should be discussed, as well as other boundary conditions clarified, like site organization, reporting scheme, etc. As in many cases the external expert(s) will not be involved on a daily basis, but more intensive in times of more critical geotechnical conditions, rules have to be established for the alert of the expert(s). This may be done by fixing warning and alarm criteria as appropriate.

5. Conclusions

The uncertainties associated with underground construction call for continuing design during construction. A continuous adjustment of the excavation and support methods to the actual rock mass conditions contributes to safe and economical tunneling.

A prerequisite for successful application of such an observational approach is an appropriate basic design, which should incorporate means and tools to cope with difficult conditions. Another must is the implementation of an adequate monitoring system, allowing the acquisition of accurate data in due time. The huge amount of data obtained during excavation needs to be processed, evaluated and interpreted. For an efficient decision process the results have to be available practically in "real time", which requires equipping the site with advanced software for data management and evaluation. Quite some progress was made in this respect over the last decade.

Interpretation of geological, geotechnical, and monitoring data due to the complexity of the ground and the interaction between ground and construction still relies a lot on education and experience. Responsible owners account for this by hiring qualified geotechnical personnel for the site assistance. Not only can qualified staff contribute to reduce accidents and damages, but can also identify opportunities to make the construction smooth and economical by optimally adjusting construction methods to the encountered ground. Hard- and software for the collection, processing and evaluation of monitoring data have enormously improved over the last decade. Last but not least, the contractual setup has to allow the continuous optimization of the construction.

Internet has made it possible to involve also off-site experts at comparatively low cost in real time. All data can be made available on a server, allowing to follow up the construction from any part of the world.

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ROCK MECHANICS AND EXCAVATION BY TUNNEL BORING MACHINE - ISSUES AND CHALLENGES

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Rapid development of tunnel boring machine (TBM) technology accelerates the extensive application of TBM in rocks and various geological conditions. TBM rock tunnelling is becoming a competitive approach, to the conventional drill and blast method. Over the years, many researches have been conducted to understand the rock fragmentation mechanism by TBM cutters, and to improve and predict the TBM performance in various rock masses. However, due to the complexity of natural grounds, TBM tunnelling still faces many challenges. This keynote paper examines rock fragmentation process, influence of joints and mixed face conditions on fragmentation and cutter force differentiation. Some of the studies show a prospect for improving TBM cutter and cutterhead design. The encountered problems in complex ground during TBM tunnelling are highlighted, and elaborated with examples. Finally, the needs for future research and development are proposed.

Keywords: Tunnel boring machine; Rock mass; Excavation; Fragmentation mechanism.

1. Introduction

The use of tunnel boring machine (TBM) can date back to 19th century. In 1851, American engineer Charles Wilson invented a tunnelling machine, which is generally considered as the first successful continuous borer for rock (Askilsrud, 1998). However, successful TBM tunnelling in hard rock ground only started in early 1950's when James S. Robbins redesigned and manufactured a TBM of 7.8 m diameter and 149 kW. Since then, the development of TBM has made a great progress. Rapid development in technology greatly improves the capacities of thrust and torque of TBM. The application of large diameter cutters with constant profile also permits TBM to overcome hard abrasive rocks. Table 1 shows the improvement of cutter technology including cutter diameter, cutter load, gauge cutter velocity and RPM. At present, the cutter size can be up to 500 mm and the thrust per cutter up to 320 kN. Different TBM types, such as gripper, open face, earth pressure balance (EPB), slurry, single and double shield, mixed shield and convertible shield are designed to suit for the different ground conditions. TBMs have technically reached a stage of development where a tunnel can be bored in any rock and ground. The reliability of rock TBMs is increasingly improved and its performance including penetration rate, advance rate and cutter wear has increased.

Nowadays, TBM is extensively utilized in rock tunnel excavation. For example, the Lötschberg base tunnel in Switzerland with an overburden depth of up to 2000 m (Vuilleumier and Seingre, 2005), was partially excavated by a gripper rock TBM with a diameter of 9.4 m. It had achieved 40.5 m advancement in 20 hours in hard rock of 160-280 MPa. Tunnelling requiring TBMs with larger diameter than ever before challenges TBM technology constantly. A TBM with diameter of 11.74 m was used for the Pinglin tunnel in Taiwan (Tseng *et al.*, 1998) and a TBM with diameter of 12.84 m for the Hida tunnel in Japan (Miura *et al.*, 2001). At the Kuala Lumpur SMART tunnel in Malaysia, the TBM has a diameter of 13.20 m. The largest rock TBM at moment is 14.4 m in diameter to be used for the excavation of the Niagara tunnel in Canada in limestone, sandstone and shale with strength up to 180 MPa. Tunnelling also encounters more and more challenging grounds, due to increasing underground development and environmental constraints. For examples, TBM are used for tunnelling in mixed-face and other complex grounds in both mountain and urban areas. Tunnels of the Deep Tunnel Sewerage System in Singapore had to frequently pass through rock-soil interface (Zhao *et al.*, 2006). The Oporto Metro in Portugal also encountered the mixed-face ground (Centis and Giacomin, 2004). Specially designed TBMs are also been sought as a possible solution for complex geology after other methods had failed. It was reported that the Hallandsas tunnel in Sweden, started in 1992 and stopped in 1997, will be restarted with a mix-shield TBM of diameter 10.5 m to overcome poor gneiss rock mass with extensive fissures and clay bands and a groundwater pressure of 13 bars.

Table 1. Cutting technology development.

Year	Cutter diameters (mm)	Average cutter spacing (mm)	Applicable cutter force (kN/cutter)	Design gauge cutter velocity, m/s	Design cutterhead RPM, rev/min
1956	280	Generally from 60 mm to 90 mm, increasing with the increase of cutter diameter	55	-	-
1970s	305		100	-	-
1980s	356		160	1.7	$32/D_{TBM}$
	394		190	2.0	$38/D_{TBM}$
	432		250	2.3	$44/D_{TBM}$
	483		280	2.6	$50/D_{TBM}$
1990s	500		320	-	-

Variation of ground at tunnel sites leads to different problems, in terms of boreability and tool wear, support and instability, groundwater pressure and inflow, and overburden pressure and in situ rock stress. Those problems remain challenges to TBM tunnelling. Generally, rock TBM excavation encounters two major problems that concern rock mechanics. One is the rock mass fragmentation by TBM cutters. It is relevant to the rock material and rock mass properties, rock mass composites at tunnel face, and interaction between rock and machine (TBM cutter spacing, cutterhead structure, thrust, torque and revolution velocity). The other is the tunnel stability and support that is the same as the drill and blast method. Due to the inflexibility and mobility of TBM, the rock pressure or squeezing ground may leads TBM stuck. This problem should be specially considered in TBM tunnel excavation.

This paper discusses the issues related to rock fragmentation process induced by TBM cutters, the influence of joint properties on fragmentation excavation, the challenges of complex ground conditions to TBM, and finally the needs of research and development on TBM technology.

2. Rock Material Fragmentation by TBM Cutters

The rock breakage process under TBM rolling cutters may be divided into two continuous stages. The first is indentation, that the rolling cutters are pressed into rock surface and then crack the rock immediately beneath the cutter. The second is chipping, that cracks between two adjacent cutters propagate across and join to form chips that can be removed.

The process of indentation on brittle rocks is well understood that when brittle materials are loaded by an indenter, three different zones can be distinguished as shown in Figure 1 (Kou *et al.*, 1998; Chiaia, 2001). Immediately below the indenter, a hydrostatic core develops, due to the high triaxial compressive stress induced beneath the indenter. Thus, this zone remains relatively intact. It may collapse only by crushing. Outside this zone, a surrounding zone of large strains develops due to the push action of the core. In this zone, the micro-cracks are pervasive. Tensile cracks may be initiated from pre-existing flaws in the material. There are basically three types of cracks, namely median cracks, radial cracks and side cracks. Outside the crack zone, the material behaves elastically. The loaded process of rock indentation test was divided into four stages including building up of a stress field, formation of a crushed zone, formation of subsurface and surface chipping and formation of crater (Cook *et al.*, 1984).

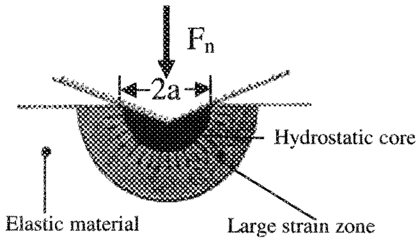
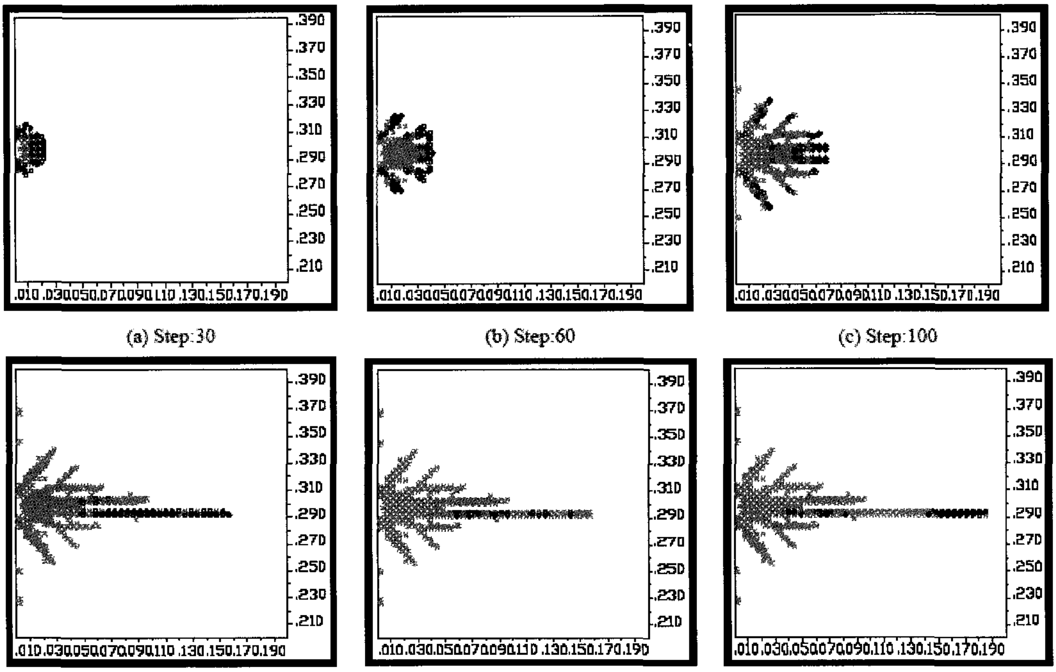


Figure 1. Indentation of brittle materials: formation of the hydrostatic core and of the large-strain zone (after Chiaia, 2001).

Gong *et al.* (2006) simulated the indentation process by single TBM cutter in granite using a discrete element modelling code. When the cutter firstly loads to the rock, a fan-shaped rock failure zone is formed as shown in Figure 2a. The frontier of the failure zone is governed by tensile failure. Immediately under the two corners of the cutter, the lateral cracks are initiated as shown in Figures 2a and 2b. As can be seen, the zone immediately beneath the cutter remains relatively intact. It is due to the high confining pressure, which leads to the phenomenon of hydrostatic compression state. As the penetration increases, the crushed zone that is directly located under the cutter is formed, as shown in Figure 2b. This zone is mainly composed of compressive failure elements including numerous micro-cracks. With the increase of penetration, the minor cracks zone is formed beneath the crushed zone, as shown in Figure 2c. The minor cracks including median and radial cracks mainly initiate from the tensile failure zone and propagate along the tensile failure elements. The crushed zone is symmetric, as shown in Figures 2b and 2c. As the penetration continuously increases, the crushed zone and minor cracks zone keep the same as shown in Figures 2d, 2e and 2f. The main cracks extend along certain directions, for example the median direction and lateral direction. The crack propagation is induced by the element tensile failure at the tip of the crack. Based on the indention process, the crack initiation and propagation process can be divided into three stages, namely the formation of a crushed zone, formation of minor crack zone and major cracks propagation.

For tunnel excavation, when TBM cutters roll across a tunnel face, they continuously expand the crushed zone beneath themselves, and cracks are initiated and propagated. One or more cracks under the action of the rolling cutter may reach the free surface or propagate to meet the cracks of the neighboring cuts. In these two cases, chipping occurs. The first case is similar to the chip formation of a single indentation process. The latter is the interaction between two adjacent cuts. It is directly relevant to the design of TBM cutterhead and the efficiency of TBM excavation.

The interaction process between neighbouring cutters is affected by the factors such as the acting loads, line spacing of cutters and rock material properties. When the cutter spacing is too large, cracks develop toward cutting face, reach free surface and then form small triangular chips. The material between two cutters is left intact. When the spacing is too small or the load is too high, longer but ineffective cracks can develop inward and meet in a steep angle. A trough between the two cutters is formed. For an optimum spacing, cracks are ideally propagated towards the neighboring cuts through a relative straight line which would be the shortest distance for crack propagation and is approximately equal to half the cutter spacing (Rostami and Ozdemir, 1993).



(a) Step:30 (b) Step:60 (c) Step:100

(d) Step:300 (e) Step:500 (f) Step:1000

Figure 2. Indentation process by single TBM cutter (circle denotes tensile failure, cross denotes compressive failure).

The rock chipping process in granite by two TBM cutters was also simulated by Gong *et al.* (2006). Figure 3 shows the crack initiation and propagation process in the rock with cutter spacing of 70 mm. At the beginning, each cutter acts on the rock independently. The stress field built up by each cutter is also independent. Two fan-shaped failure zones are formed beneath each cutter, similar to that for a single cutter, as shown in Figure 3a. With the increase of the penetration, the crushed zones are formed immediately beneath the two cutters as shown in Figure 3b. As the penetration increases continuously, the minor cracks including median and radial cracks continuously propagate. After that, the interaction between two cutters is observed. The side cracks between two cutters change their extension direction and propagate toward each other. The chip is not formed until the two cracks coalesce in the middle of the two cutters, as shown in Figure 3c. As the penetration increases, the minor cracks zone remains the same. The main cracks including median and side cracks develop continuously downwards and sideward due to the element tensile failure, as shown in Figures 3d and 3e. With the built-up of the stress field, the development of crack pattern is shown in Figures 3f, 3g and 3h.

Chip formation is greatly dependent on cutter spacing, critical cutter load and rock properties. The critical cutter load required for chipping increases with increasing cutter spacing. As long as the critical cutter load can be reached, the penetration rate will achieve the maximum value in that granite at about cutter spacing of 100 mm based on the simulation results. This penetration rate is more than twice of that for the cutter spacing of 60 mm.

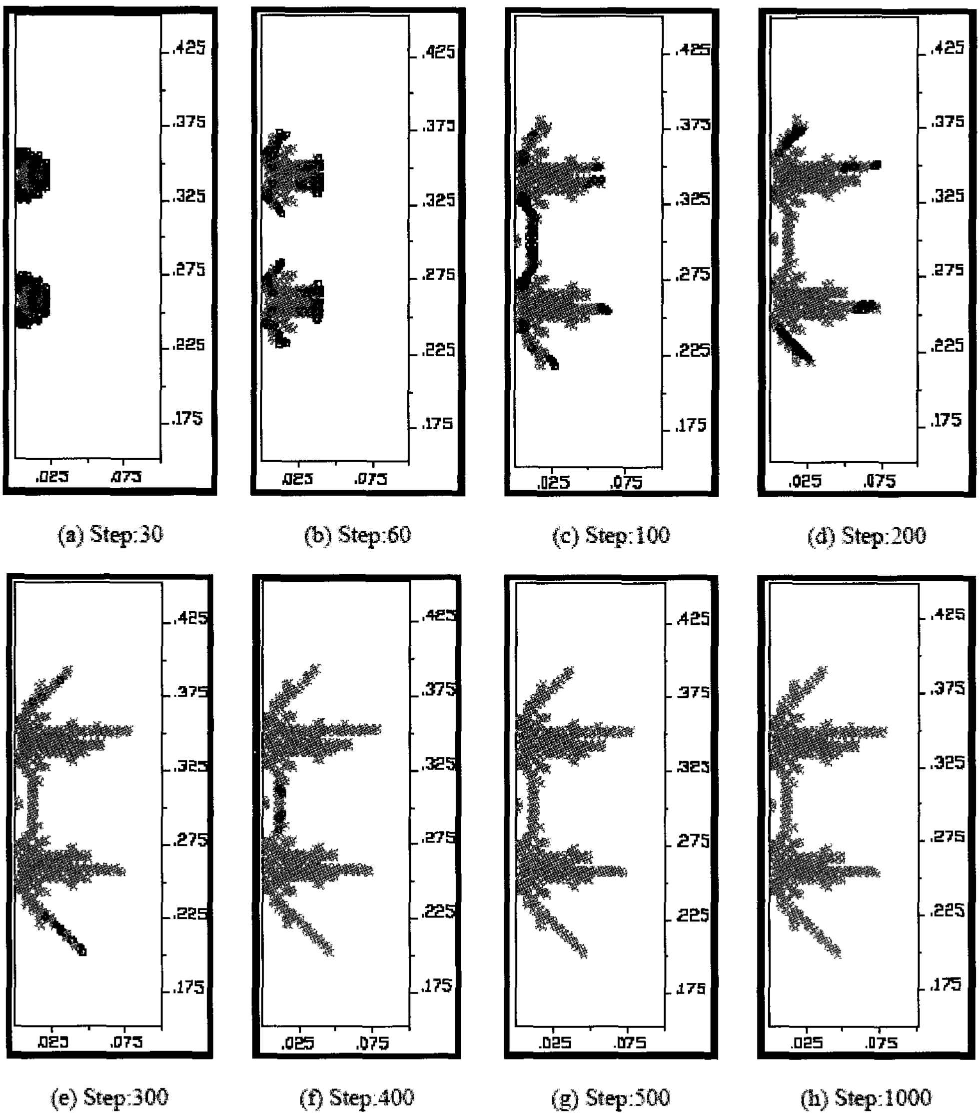


Figure 3. Two cutters interaction process (circle denotes tensile failure, cross denotes compressive failure).

3. TBM Cutting in Fractured Rock Mass

The previous section only shows the fragmentation process in brittle rock materials by TBM cutters. In practice, the rock mass under excavation contains discontinuities. It has been well recognized that existing joints and fractures in a rock mass facilitate the breakage, as they speed up the formation of rock chips or rock blocks during cutting. Figure 4 illustrates the possible influence of joint spacing and orientation on rock breakage process. There are two angles that may affect the rock breakage process, the acute angle α between tunnel axis and joint plane, and the attack angle β between cutter rolling direction and joint outcrop in the tunnel face, as shown in Figure 4. However, when TBM advances, and cutter rotates in concentric circles, the angle β constantly changes from 0 to 360 degrees. The effect of β therefore is evened out. As a result, only the effect of the angle α is important and needs to be considered.

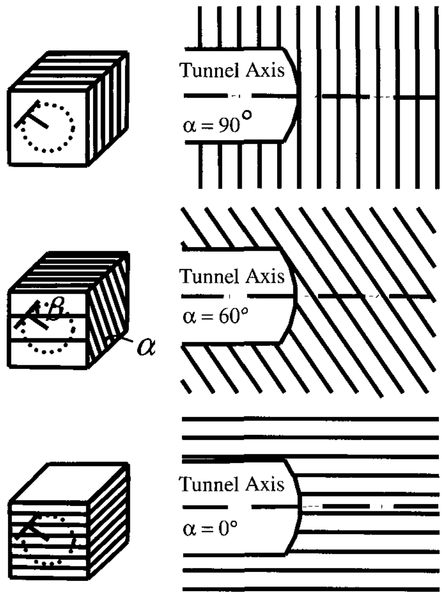


Figure 4. Influence of joint spacing and orientation on rock fragmentation process.

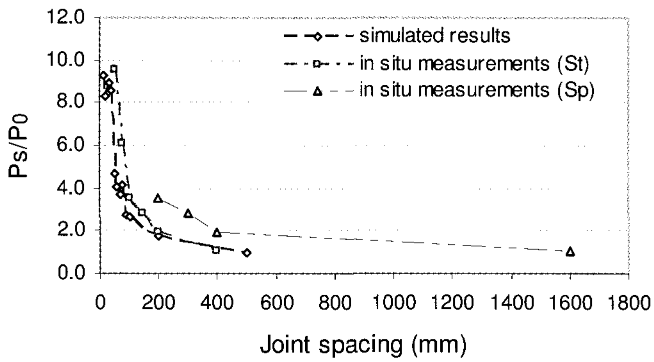


Figure 5. Effect of the joint spacing on the TBM penetration rate (St denotes fissures and Sp denotes joints).

Bruland (1998) divided discontinuities into three types, namely, fissure, joint and single marked joint. Fissures and joints were then classified into five classes. Based on a large number of case histories, a fracturing factor was obtained according to the type and class of discontinuities. With the decrease of the joint spacing, the TBM penetration increases distinctly. This phenomenon was observed at many TBM tunnelling sites.

Gong *et al.* (2006) simulated the influence of joint spacing on rock fragmentation process, and then the penetration rate by TBM cutters using discrete element modelling. Due to the existence of the joint plane beneath the cutter, the principal stress field is affected. It leads to the variation of the rock chipping angle and chipping pattern. The growth of cracks terminates at the joint interface which intersects the crack propagation. The simulation results of the effect of joint spacing on TBM excavation are shown in Figure 5. In this figure, P_s denotes the penetration rate at joint spacing S, and P_0 denotes the penetration rate without joints. With the increase of the joint spacing, penetration rate decreases. The results are compared with the field measurements by Bruland (1998) also shown in Figure 5. The curve shape of the simulated results shows good agreement with that of the field measurements.

In situ measurements in a homogeneous zone of schistose phyllite by Aeberli and Wanner (1978) and in phyllite and phyllite-carbonate-schist inter-stratification by Thuro and Plinninger (2003) showed that the advance rate of TBM increases with the increase of the angle between TBM axis and the planes of schistosity. Bruland (1998) concluded that when the angle is equal to 60 degrees, the penetration rate reaches the largest value and with the increase of joint spacing, the effect of joint orientation on TBM penetration decreases.

The effect of joint orientation on rock breakage process with joint spacing set at 200 mm was simulated by Gong *et al.* (2005). The effect of joint orientation on TBM excavation mainly concentrates on the changes of the rock chipping pattern in different angles. The simulation results are shown in Figure 6. In the figure, $P\alpha$ denotes the penetration rate at an angle between tunnel axis and joint plane is equal to α, and P_0 denotes the penetration rate at $\alpha = 0°$. As the angle α increases, the penetration increases until α reaches 60°, then the penetration rate decreases with increasing α. The results are in good agreement with the in situ observations by Bruland (1998).

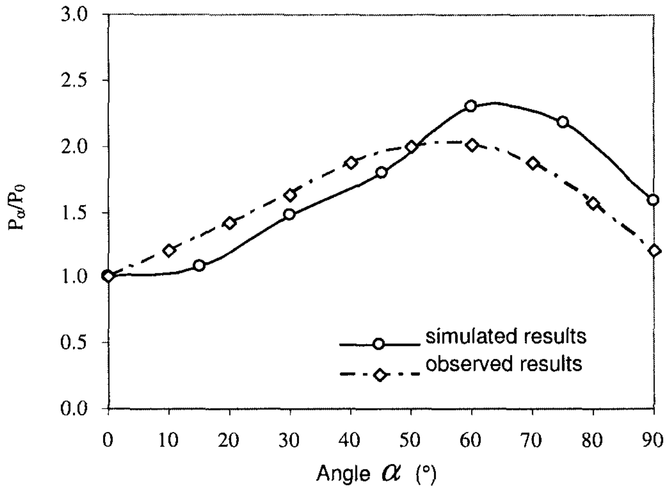


Figure 6. Effect of joint orientation on the TBM penetration rate.

4. TBM Cutting in Complex Geology and Mixed Ground

When tunnelling in grounds with mixed geomaterials (mixed-face), the most paramount effect on cutting process is the dynamic impacts on the cutters and cutterhead, which lead to tool damages and cutterhead vibrations. Cutter wear, cutter bearing failure and abnormal cutter ring wear such as pitted, chipped, flatted, multi-flatted wear or damage are usually high in such ground conditions. Face instability, large groundwater ingress and ground settlement frequently took place in mixed-face ground. In some cases, difficulties in cutterhead steering occur. These phenomena were observed and reported by a number of researchers (Wallis 2000, Della Valle 2001, Blindheim *et al.*, 2002, Steingrimsson *et al.*, 2002, and Zhao *et al.*, 2006). In order to minimize cutterhead vibration and dynamic impacts on cutters, it is of top priority to optimize TBM operation thrust and rotational speed. In practice, there is no definite approach to guide the TBM operation in the grounds with mixed geomaterials.

The cutter force differentiation in the mixed face conditions using a finite element numerical model was analyzed by Dong *et al.* (2006). Two cutters are applied with velocity loadings on the tunnel face of two rock materials with different strengths. The force differentiation on two cutters increases with increasing strength difference between the two materials. Based on the simulated results, the correlation between a cutter force differentiation ratio and the rock strength ratio can be obtained, as an example shown in Figure 7. With known area ratio of the hard to the soft portions in the mixed face, the total thrust that the cutters can bear may be calculated. In order to prevent large differential deformation of the cutterhead in the mixed face, cutter force differentiation can be estimated for cutterhead stiffness design.

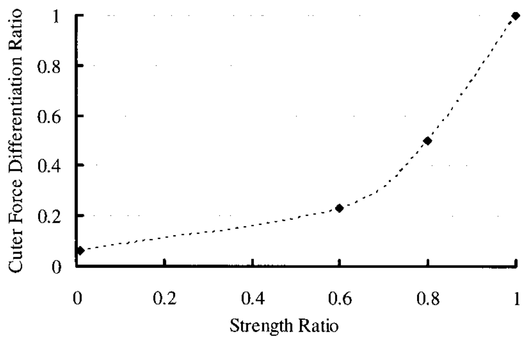


Figure 7. Correlation between cutter force differentiation and the rock strength ratio.

5. Challenges in TBM Excavation

TBM performance relies on the machine design, geological conditions, and tunnellers experiences. Site geological conditions are the fundamental factors. Machine optimal design and project planning are essentially based on site geological conditions. Thus, every successful tunnel project requires high level site investigation. Due to the natural variation of geological conditions along tunnel alignment, it is often difficult to predict exact rock mass conditions along the tunnel on the basis of site investigation. With varying geological conditions, TBM design is difficult to deal with all the possible scenarios in an optimum mode, and it has to compromise to suit the dominating geological conditions to strive an overall performance.

Barla and Pelizza (2000) defined limiting geological conditions for TBM's application as "the geological conditions are such that the TBM cannot work in the execution mode in which it was designed and manufactured. Thus the advance of the TBM is significantly slowed down or even obstructed". Over the years different types of TBMs have been developed to adapt different geological conditions. Especially mix-shield and EPB-slurry convertible machines are developed to cope with some complex geological conditions encountered in one TBM drive. It reduces the risks induced by varying geological conditions on TBM tunnelling. However, exceptionally difficult geological conditions, in spite of expected or not expected, do exist and are challenges to TBM tunneling. Such difficult conditions have influences on TBM performance, and affect the overall advance of the project and the cost.

Some typical difficult geological conditions include very high strength hard rock with high abrasivity, sticky soft rock, squeezing rock mass, rock spalling and bursting, fault zones, mixed face and rock-soil interface, excessive inflow and high groundwater table. These conditions may cause problems as follows:

(i) Ineffective rock cutting and excavation;
(ii) Instability of excavation zone;
(iii) TBM stuck;
(iv) High cutter wear; and;
(v) Need of ground treatment.

Case A: Hard Rock with High Abrasivity

Qinling railway tunnel lies in the middle of the northern Qinling Mountains, west China. It is composed of two parallel single-track tunnels with a separation of 30 m. One tunnel is 18,452 m long, excavated by two gripper TBMs. The other tunnel is 18,456 m long, excavated by conventional drill-and-blast method. The maximum overburden depth of the tunnels is about 1,600 m (Liu and Liang, 2000). The geological conditions along the tunnel alignment are mainly hard granite and gneiss formations (Figure 8) with uniaxial compressive strength averaging between 150 and 250 MPa and the maximum value of 325 MPa. The two gripper TBMs were 8.8 m in diameter. The designed thrust was 21,000 kN, torque 5,800 kNm, and rotation speed 2.7 rpm for the soft rock and 5.4 rpm for hard rock. The designed penetration rate was about 3.5 m/h for the rock with uniaxial compressive strength from 100 to 180 MPa, and more than 1 m/h for the rock with uniaxial compressive strength up to 325 MPa.

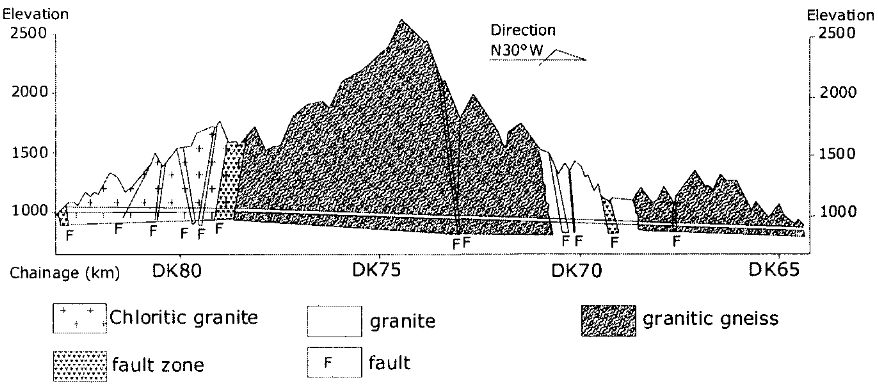


Figure 8. Geological profile of Qinling Tunnel.

The first TBM started excavation in December 1997 in the north drive. The encountered rock masses were massive granitic gneiss with high uniaxial compressive strength (average strength about 120 MPa). Based on statistical analysis of the first six months (Li and Yan, 1999), the advance rate was 236 m per month. The average penetration rate is less than 1 m/h, and the lowest penetration rate was 0.36 m/h. The cutter wear was very high, with average cutter life of 37.3 m^3/disc, which was far less than desired 100 m^3/disc. The cutter replacement led to long downtime. The utilization was less than 20%. TBM performance index was less than that predicted. These resulted in the delay and high cost of tunnelling.

Gehring (1994) reported the same problem which took place in the Clermont tunnel short after commencing operation in South African. The rock masses are composed of massive red sandstone with an unaxial compressive strength of 50 to 230 MPa and high quartz content of 55% to 70%.

During the excavation of the starting 493 m, the average cutter life was down to around 30 m^3/dics. Sometime, the cutter life located at gauge area was lower than 10 m^3/disc.

In these two cases, massive rock masses with high abrasivity were not fragmented effectively by TBM cutters, which also led to high cutter wear. Apart from rock properties, machine factors influencing rock fragmentation process should be considered in relation to rock-cutter interaction, including cutter spacing, cutter width and the exerted critical cutter thrust.

Case B: Rock Spalling and Bursting

Long tunnels at great depth are being constructed in many locations worldwide. With great overburden, stress induced problems can be anticipated. Application of TBM in those tunnels needs to handle the stress induced problems. In the Lötschberg Base Tunnel, the depth of the Aar massif granite and granodiorite rock cover along the route reached up to 2000 m. The rock spalling and bursting was encountered (Vuilleumier and Seingre, 2005). During the excavation, the following phenomena occurred (Figure 9):

(a)Rock blocks were formed in front of the TBM head. The blocks had no particular shape. They damaged mucking buckets and band conveyors, and caused disc cutter damage and high cutterhead wear. The blocks reduced the TBM utilization time and penetration rate.

(b)Due to the block formation, the tunnel face was blocky (Figure 9a). The blocky tunnel face caused cutterhead vibration and high impact load on cutters, and aggravated abnormal cutter wear.

(c)Behind the tunnel face, scales of low thickness peel off the walls in the excavation. Under high overburden, deep notches up to about 1 m appeared, typically in a symmetrical pattern (Figure 9b). The installation of rock support greatly slowed down the tunnelling progress.

(d)In some cases, the notches developed to such an extent that the griper pads cannot be seated on the side wall (Figure 9c), which led to more downtime.

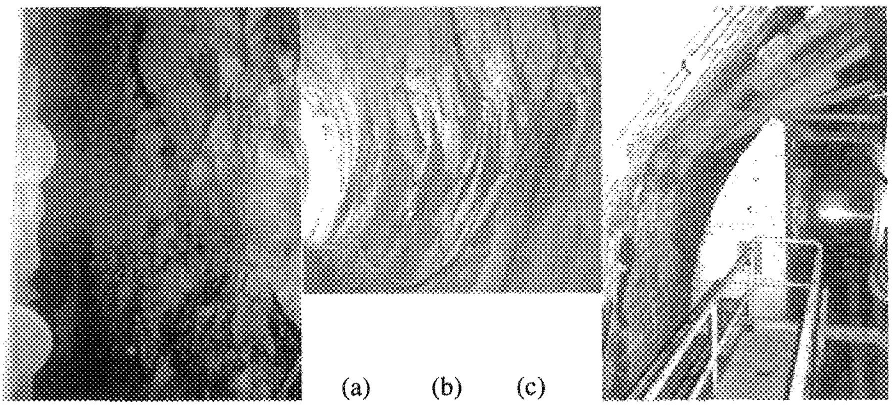


Figure 9. Spalling and blocky rock masses at the Lötschberg Base Tunnel.

The rock stress problems were also encountered in other TBM excavated tunnels (e.g., Myrvang *et al.*, 1998). However, favourable in situ stress increases TBM penetration rate in some tunnelling projects (e.g., Myrvang *et al.* 1998; Boniface, 2000). Only when the in situ stress is too high and rock is overstressed, rock mass slabbing and spalling, ravelling, face overbreaking and ground squeezing may occur. The present assumptions of TBM cutting are on rock fragmentation of a flat front surface. The effects of spalled blocky surface on fragmentation mechanism are not well examined. In addition, fragmentation mechanism and requirements for cutters (e.g., spacing and load) may be different under high in situ stress.

Case C: Mixed Face Ground

Zhao *et al.* (2006) analyzed the problems incurred by a typical rock-soil interface in a sewerage tunnel in Singapore. The tunnel passed through granite with different weathering grades from fresh rock to residual soil. The ground at tunnel level frequently changed from fresh granite to residual soil or vice versa. TBMs encountered rock-soil interface frequently.

The rock-soil interface often consisted of a thin layer of completely weathered granite between massive rock and residual soil. This thin layer was generally silty fine to coarse sand with gravels, and highly permeable. The high groundwater table resulted in unstable face conditions and the creation of an excessively wet, non-plastic, almost fluid soil in the cutterhead chamber. Excavation by TBM created a loss of groundwater head at the face. Water flew through the interface layers into the chamber leading to ground erosion and annular gap, as seen in Figure 10. With high permeability of the completely weathered material, water flow in this material consequently led to material erosion and loss of shear strength. The region affected by water flow and erosion extended, resulting in further increase of the water inflow. This process may induce large surface settlements and eventually collapse of the ground, as shown in Figure 11. High groundwater inflow also led to erosion of the grout in the annular gap.

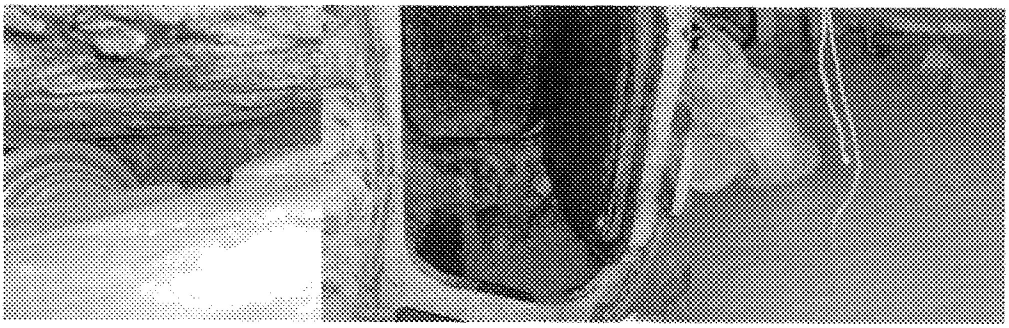


Figure 10. Water inflow from the tunnel face.

Figure 11. Tunnel face instability and ground collapse.

Due to unstable ground conditions and water inflow, the shield had to work in the closed or transition mode. The rock chips fragmented by disc cutters were difficult to be collected by buckets to be transferred into the cutterhead chamber. The spoil trapped between the rock and the cutterhead increased the rate of abrasive wear of face plate, bucket lips and drag bits and mounts, as an example shown in Figure 12. The cutter life-time was much lower than 100 m^3/cutter.

Due to rapid variation of hard rock portion (size and position) in the mixed face, control of admissible axial cutter forces becomes extremely difficult in the closed mode. The transversal shock load also varies greatly during the cutters run from hard rock portion to soft soil portion. Hence, the control of overloads on the disc cutters is very difficult. These axial overloads are resulted from the soil-rock interface and blocky ground in the tunnel face. The cutter overload contributes automatically to an unusual increase of the tool wear.

High groundwater pressure associated with high permeability of the ground has also resulted in the need to use relatively high compressed air pressures during maintenance interventions. In some cases, the loss of compressed air through fissures leads to local ground collapse and water ingress which has made conditions extremely difficult and dangerous.

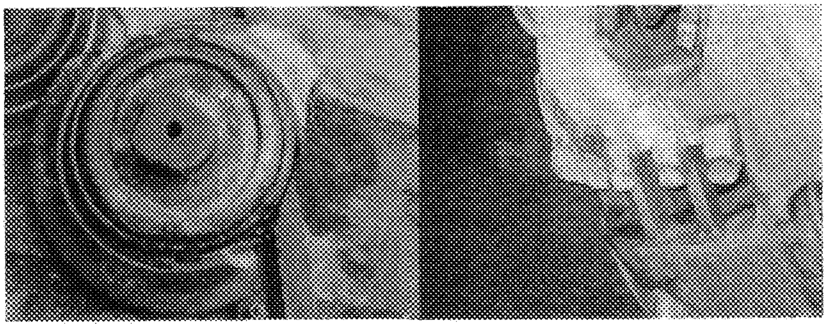


Figure 12. Cutter flat wear and abnormal cutterhead wear.

6. Needs of R&D in TBM Rock Tunnelling

TBM rock tunnelling process is a combination of excavation, face and excavated zone stability, system availability and construction management. In term of excavation, understanding of rock fragmentation mechanism through the interaction process between cutter and rock mass facilitates optimization of TBM cutter and cutterhead design, and of TBM components configuration, to achieve best penetration rate. However, due to the complexity of geological conditions, penetration or advancing process during TBM tunnelling faces many challenges. Technological solutions are needed to improve TBM performance, in, but not limited to the areas below.

(a) Understanding rock fragmentation mechanism by TBM cutters, specifically, (i) Fragmentation mechanism of low strength and non brittle rocks; (ii) Fragmentation mechanism of ultra hard crystalline rocks; (iii) Influence of cutter rolling speed on rock chipping process, i.e., loading rate effect; (iv) Influence of time gap between cutters on rock chipping process; (v) Relation between cutter spacing and rock material and rock mass properties; (vi) Interaction of existing fractures and new cracks during cutting process; (vii) Influence of in situ stress on rock cracking and chipping mechanism; (viii) Influence of uneven surface on rock cracking and chipping process; and (ix) Impact and impact induced damage of cutting tools in mixed ground.

(b) Improvement of cutter and optimization design of TBM cutterhead, including, (i) New material, cutter size and tip width to improve cutter hardness and toughness, and cutter lifetime; (ii) TBM cutterhead design considering cutter spacing, thrust and rock properties; (iii) Size and location of opening for EPB cutterhead design by addressing material movement; and (iv) Cutterhead stiffness for TBM in blocky and mixed face ground.

(c) Development of a rock mass classification to guide TBM selection and operation, for examples, (i) A rock mass classification scheme, incorporating appropriate rock mass properties for TBM excavation; (ii) An improved TBM performance prediction model for difficult rock masses and complex geological conditions; and (iii) A criterion or a guide for TBM operation in complex geological conditions.

Tunnelling, both conventional and mechanised, is an engineering process. It is and always should be a process of applying scientific principles to the planning, design and construction. It is foreseen that rock TBMs will be increasingly used for long tunnels. The challenges to TBM in rock tunnelling have to be met by technology innovations through research and development.

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ROCHA MEDAL AWARD PAPER

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STRATEGY FOR IN-SITU ROCK STRESS MEASUREMENTS

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In this paper, we propose a protocol for evaluating the complete stress field and its variation with depth using an integrated stress determination approach. Although the strategy is based on sampling the stress field with hydraulic methods, it may be applied to most stress measuring techniques and integrated studies. It is shown that the procedure will not be hampered by difficulties reported with the individual stress measuring techniques. Our strategy is based on the directions of the ISRM suggested methods for rock stress estimation by hydraulic fracturing and hydraulic tests on pre-existing fractures (Haimson & Cornet, 2003). In addition, we pay particular attention to the sampling strategy and considerations of the continuity hypothesis.

Keywords: Rock stress evaluation; Integration; Hydraulic fracturing (HF); Hydraulic tests on pre-existing fractures (HTPF).

1. Introduction

Knowledge of the prevailing stress field is important for rock mechanical studies, because it provides means to analyze the mechanical behavior of rock and serves as boundary conditions in rock engineering problems. Most in-situ stress measuring methods provide point-wise estimates of local stress tensors of a considerably smaller rock volume than that of the rock mechanical problem in question. The regional stress tensor can be determined from a number of rock stress estimates of local stress tensors. Because the local stress tensors often are influenced by discontinuities within the rock volume, the regional stress tensor can generally not be obtained simply by averaging the local stress tensors (Amadei & Stephansson, 1997). Rather, the extrapolating exercise from local to regional stress tensor requires great care.

Cornet (1993a) obtains the regional stress tensor for large rock volumes that takes stress variations into account. He defines the regional stress tensor with six functions of spatial coordinates from a number of local stress tensors for small rock volumes that each has six independent components.

The primary objective of this paper is to propose a strategy for evaluating the complete stress field and it's variation with depth. The strategy is based on collection of hydraulic stress data in thrust regimes but it may be generalized to various stress measurement techniques as well as for integrated studies. The secondary objective is to present a method for stress field determination, the Integrated Stress Determination Method (ISDM; Ask, 2004). In the ISDM, the full stress tensor and it's variation with depth can be determined for a variety of stress measuring techniques. A great advantage of data integration is that complementary data can be incorporated to obtain individually the best constraint on only some of the different model parameters. We emphasize that the aim is to combine all available data to achieve as complete a characterization of the mechanical model as possible, and not to identify a solution that fits only loosely the maximum amount of data. The third objective is to show how the proposed strategy for collection of hydraulic stress measurements will not be hampered by difficulties reported with the classical hydraulic fracturing technique.

2. Strategy for Hydraulic Rock Stress Measurements

2.1. *Definition of the domain(s) where the stress field is to be determined*

The concept of stress is a concept of continuum mechanics. It applies only to bodies that are regular enough to be approximated by a continuum. Because the stress at a point involves six components, the determination of the regional stress field includes determination of six functions for the domain under consideration. This requires integrating measurements conducted at points that sample properly the continuum volume of interest.

Thus, prior to discussing how to ascertain the validity of a specific stress measurement at a certain point, it is necessary to identify volumes where the continuity hypothesis is verified. It is completely pointless to compare a measurement at a given depth with that at another depth if the two points of measurements do not belong to the same continuum. This is one of the major difficulties in evaluating stress measurements, and only preliminary geological and geophysical reconnaissance can help answer this question.

Therefore, a crucial point for a successful choice of test locations is a proper evaluation of the continuity hypothesis. It is strongly recommended that careful analysis of cores and geophysical logs be conducted before selecting points of measurements and before selecting the set of data that will be used for the regional stress evaluation. Because decoupling zones may exist, identifying these and determining their effect on the stress field is an important objective (e.g. Ask *et al.*, 2006).

2.2. *Methodology for hydraulic tests in vertical and inclined boreholes*

The most common approach for stress field interpretation in boreholes is profiling, i.e. data are collected along the entire or a long section of the borehole. Results are used thereafter for determination of the stress field and its variation with depth or along the borehole length. However, in the case of non-linearity, the linear interpolation method may not provide satisfactory results. In such cases, an alternative method is the cluster approach. The methodology involves three steps (Ask *et al.*, 2006):

(i) Identification of domains where the continuity hypothesis is validated;

(ii) Combination of measurements in a clustered procedure so that each cluster corresponds to a small enough volume to permit complete stress determination without considering stress gradients; and

(iii) Integration of results from all clusters so as to establish the validity of the continuity hypothesis and determine the complete stress field within the domain of interest with proper attention to decoupling zones.

We describe the validation of this strategy in more detail in section four, with emphasis on collecting data with hydraulic methods. Preceding this, we present highlights of the ISDM (e.g. Cornet, 1993b; Ask, 2006) in section three.

3. The Integrated Stress Determination Method (ISDM) and Recent Developments

The ISDM (Cornet, 1993b) involves several steps that must be considered for each case study (Fig. 1):

(i) The number and type of available data defines the parameterization of the stress field within the rock volume of interest. An increasing number of data can solve an increasing number of unknown model parameters, provided that the stress data sample more than one stress vector. For example, because the induced fracture during hydraulic fracturing measurements generally is parallel with the borehole axis, such data can only be used to determine the magnitude and orientation of minimum horizontal stress versus depth;
(ii) The rock volume, which is defined by the distribution of the available stress data, should be considered with respect to the homogenization criterion. Existing discontinuities may lead to subdivisions of the rock volume and data sets;
(iii) Selection of a proper mathematical algorithm. The early applications of the ISDM were using a non-linear least squares method (also applied by Ask (2004) and referred to as the Gradient method; e.g. Cornet (1993b) but recent work has also been based on Genetic Algorithms (e.g. Cornet *et al.* (1997));
(iv) *A priori* values for the Gradient method are determined from available stress data, or from a global Monte Carlo search for model parameters; and finally
(v) The solution is verified.

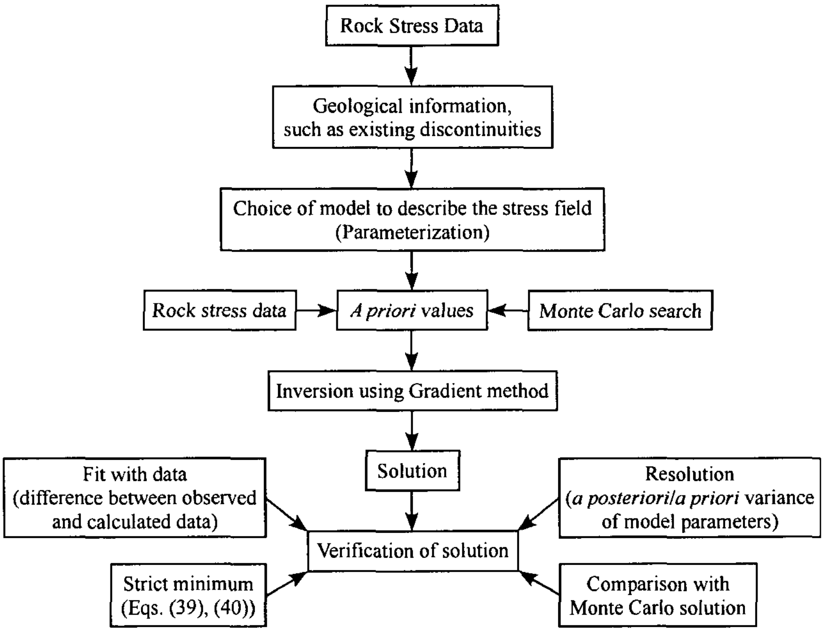

Fig. 1. Approach for stress determination using the ISDM based on the Gradient method. The rock stress data and the geological information control the parameterization of the stress field in the rock mass. *A priori* values for the Gradient method are derived from available stress data or from Monte Carlo simulations (in this study). When a solution has been found, it is verified using four methods (After Ask (2006)).

When different types of stress measurements are integrated, care must be taken regarding e.g. the number of each data set, the nature of the different data sets, and volume involved during measurement. These differences can be overcome by definition and inclusion of misfit functions. This is further discussed in e.g. Ask (2006).

3.1. *Parameterization*

In the ISDM, the measured rock volume is discretized into sub-volumes in which the stress field is approximated by it's first order linear expansion (Cornet, 1993b). The stress at a point X^m of the m^{th} measurement is given by:

$$\sigma(X^m) = \sigma(X) + (x^m - x)\alpha^{[x]} + (y^m - y)\alpha^{[y]} + (z^m - z)\alpha^{[z]} \tag{1}$$

where $\sigma(X^m)$ and $\sigma(X)$ are the stress tensor in points X^m and X, respectively, and $\alpha^{[x]}$, $\alpha^{[y]}$, and $\alpha^{[z]}$ are second-order symmetrical tensors characterizing the stress gradient in the x-, y- and z-directions. Equation 1 satisfies the following equilibrium constraints (Cornet, 1993b):

$$div(\sigma(X)) - \rho(X)b = 0 \tag{2}$$

where $\rho(X)$ is the density of the rock mass in the point X, and b is the gravitational acceleration ($b_i = g\delta_{i3}$; $\delta_{i3} = 0$ for $i \neq 3$; $\delta_{i3} = 1$ for $i = 3$). The first order approximation of the stress field requires determination of 22 parameters. If the data set is too small to determine all 22 parameters, the number of unknowns can be reduced using the following assumptions: (1) the lateral stress variations are neglected (which is exactly valid if only data along one single rectiliniear borehole direction are available); (2) one principal stress is vertical throughout the volume; (3) if 2 applies, the rock mass density is obtained from direct measurements on cores (neglecting effect of fractures on density); (4) there is no rotation of principal stresses (in small rock volumes); (5) the stress field is continuous up to ground surface.

In regions with negligible topographical effects and a homogeneous rock mass, implying that lateral stress gradients can be neglected, Eq. 1 is reduced to:

$$\sigma(X^m) = \sigma(X) + (z^m - z)\alpha^{[z]} \tag{3}$$

In the chosen parameterization, $\sigma(X)$ and $\alpha^{[z]}$ are expressed with three Euler angles and three principal values. For $\sigma(X)$, the eigenvalues are S_1 to S_3 and the three Euler angles are E_1 to E_3, which are expressed in the geographical frame of reference. Corresponding eigenvalues for $\alpha^{[z]}$ are α_1 to α_3 and the three Euler angles E_4 to E_6, which are expressed in the $\sigma(X)$ frame of reference. Thus, the gradients α_1-α_3 correspond to the vertical gradient of S_1-S_3 only if E_1- E_3 are equal to E_4-E_6 (Ask, 2006).

3.2. *The inverse problem and it's solution*

The inversion is performed using a method developed by Cornet and Valette (1984), based on the least squares criterion (Tarantola & Valette, 1982). In this method, *a priori* knowledge of the unknown model parameters is assumed to exist, which can be formulated in terms of expected value, variance and covariances. In practice, large error bars are placed on assumed central values for the unknown parameters. The hydraulic fracturing and HTPF data involve 4 components and the general expression for the fracture normal stress, σ_n, from hydraulic fracturing and hydraulic tests on pre-existing fractures is:

$$[\sigma(X^m)\vec{n}^m]\vec{n}^m = \sigma^m_{normal} \tag{4}$$

where n^m is the normal of the m^{th} fracture plane and includes the dip direction, ϕ^m, and the dip, φ^m, of the normal to the m^{th} fracture plane with respect to the vertical direction. Equations 3 and 4 can be used to define a vector function $f(\pi)$ for which the m^{th} component is given by:

$$f^m(\pi) = \sigma_n^m - \left(\left[SB \cdot \left(S^o + \left(z^m - z\right) \cdot AB \cdot A^o \cdot AB^T\right) \cdot SB^T\right] n^m\right) n^m \tag{5}$$

where matrices S^o and A^o represent the stress and gradient tensors, AB includes Euler angles E_4 to E_6, which describe A^o in the S^o frame of reference, SB includes the Euler angles E_1 to E_3, which describe S^o and A^o in the geographical frame of reference, z^m is the depth of the m^{th} fracture, and z is the chosen calculation depth (normally the average depth of the data set). Note that for overcoring, there are four different expressions for $f^m(\pi)$; for axial, tangential and 45°- and 135°-inclined gauges. Continuing with the hydraulic fracturing data case, the solution of the inverse problem is defined by the minimum of:

$$\left(\pi - \pi_o\right)^T C_o^{-1} \left(\pi - \pi_o\right) \tag{6}$$

The problem is a conditional least square, i.e. the minimum of the least squares criterion (Eq. 6) is sought that satisfies the condition $f(\pi)=0$. Tarantola and Valette (1982) showed that this could be solved using the iterative algorithm based on the fixed-point method:

$$\pi_{n+1} = \pi_o + C_o F_n^T \left(F_n C_o F_n^T\right)^{-1} \left[F_n \left(\pi_n - \pi_o\right) - f\left(\pi_n\right)\right] \tag{7}$$

where F is a matrix of partial derivatives of $f(\pi)$ valued at point π.

4. Validation of Strategy for the Case of Hydraulic Stress Data in Thrust Regimes

The validation of the suggested strategy consists of two parts, that associated during collection of hydraulic data (i.e. the cluster approach) and that associated with results. These are described below.

4.1. *Validation of cluster approach*

HTPF measurements are generally used to constrain the magnitude of primarily σ_H but also σ_v, once σ_h has been determined using hydraulic fracturing measurements. Thrust regimes in which the minimum principal stress is vertical is a special case because conventional hydraulic fracturing tests will not yield satisfactory results: hydraulic fracturing tests will induce a fracture that will rotate from the axial to the horizontal direction (in vertical boreholes). Hence, only the vertical stress component will be measured, which can be regarded as a lower limit of σ_v (e.g. Evans *et al.* 1989). We show below that the HTPF technique is not affected by the deficiencies of the conventional hydraulic fracturing technique.

4.1.1. *Single clusters*

In the HTPF technique, isolated fractures with a large variety of dip and dip directions are sought. The intension is to re-open pre-existing fracture planes with different orientation and to determine the pressures that exactly balances the normal stress acting on the respective fracture plane. In addition to select suitable pre-existing fractures with different orientations, several other factors must be addressed for achieving successful results (Ask, 2006):

(i) The fractures should be distributed with a sufficient spacing to prevent opening of additional fractures during injection testing;

(ii) If a limited number of fracture orientations exist, fractures with similar orientation should be spaced at least 50 m apart so that a stress gradient can be picked up;

(iii) The chosen fractures should be at least partially opened or coated with weak fracture minerals, which, using a low flow rate test, enhances the possibility for re-opening as the fluid has time to penetrate the fracture plane and add an additional stress component;

(iv) Individual tests should be separated by ca. 2 m to avoid the local stress change caused by the neighboring test as a result of the mechanical opening of a fracture; and

(v) Fracture selection should preferably be conducted by using electrical imaging logs (e.g. HTPF tool; Mosnier & Cornet, 1989), for electric conductivity is often a useful analog of hydraulic conductivity.

Because thrust regimes involve large horizontal stress magnitudes, it is possible that pre-existing fractures perpendicular to σ_H may not be opened, because of limitations in the testing equipment, or because fractures with other orientations are opened at lower pressures. As a result, fractures somewhat inclined to σ_H may have to be chosen (which effectively lowers the normal stress). The objective is to take advantage of optimally oriented pre-existing fractures, even if they are in a very limited quantity (Ask, 2006).

In the following, we consider the ultimate case in which only one single direction is observed within a given restricted depth range. If more directions are available, the resolution of proposed methodology will be improved.

After identification of fractures of interest, both hydraulic fracturing tests and HTPF will be conducted. From the hydraulic fracturing tests the geometry of hydraulically induced fractures, run within 20 m of the chosen pre-existing fracture(s), will confirm that, for vertical boreholes, the vertical direction is indeed principal. This will leave three unknowns to be determined, namely the principal stress components. Moreover, the hydraulic fracturing tests will provide both a breakdown pressure and an evaluation of the minimum (vertical) principal stress. This leaves two unknowns, the maximum and minimum horizontal principal stress components. These can be solved with two types of data: breakdown pressures and normal stress measurements on the selected pre-existing fracture(s). When possible, test zones will be selected so that at least two pre-existing fractures can be tested. Hence, with four tests (minimum 3), the two unknowns will be resolved satisfactorily (Ask *et al.*, 2006).

Tests run in inclined boreholes can be used for two purposes; complementary HTPF tests may be used for evaluating lateral extension of continuous domains and hydraulic fracturing tests for trying to generate en echelon fractures. The orientation of these en echelon fractures provides excellent constraint on the maximum horizontal stress magnitude when all other five stress components have been accurately determined (e.g. Hayashi et al., 1989, Peska and Zobak, 1995, Wileveau et al., 2006).

4.1.2. *Multiple clusters*

The primary objective is to determine the complete stress field in domains of interest, and to identify decoupling zones to other stress domains. If additional data from other methods are available, a secondary objective is to take advantage as much as possible of already existing results.

Potential decoupling zones can be identified using various methods, e.g. using the HTPF tool, Formation Micro Scanner and Borehole Televiewer data. Once completed, a set of depth intervals for both hydraulic fracturing tests and HTPF can be selected. Preferably, they include as much as possible volumes where other stress measuring techniques have been conducted to allow a subsequent integrated stress determination approach (Ask *et al.*, 2006).

The number of clusters is of coarse dependent upon the domain(s) of interest, but if a stress profile for a 1000 m borehole is to be determined, it is recommended to choose at least three, possibly four, depth intervals per borehole for clustered measurements so that at least three, and possibly four, complete stress determinations are conducted without any hypothesis on stress field continuity. Optimally, they should not be extended over a depth range larger than 50 m, so as to limit the number of model parameters for each individual cluster. Hydraulic fracturing tests and HTPF may initially be independently analyzed. Thereafter, they may be integrated with complementary data, if available (Ask *et al.*, 2006).

Finally, all data may be integrated progressively into larger volumes so as to provide the required complete stress field determination over the desired domain (Ask *et al.*, 2006).

4.2. *Validation of results*

For stress determinations, two efficient ways are usually considered for validating the results (Ask *et al.*, 2006):

(i) Consistency of the results for large data sets, and
(ii) Similarity in the results obtained from different techniques.

At most sites, regional principal stress directions can be produced fairly reliably through hydraulically induced fracture orientations and borehole breakout orientations, if they are observed. The real question usually concerns the possible role of large fracture zones as source of decoupling so that rocks on both sides of the fracture zone, away from the zone, may not support the same stress field. Let us note that because equilibrium conditions are satisfied, the normal stress component is always continuous.

It is essential to obtain a sufficient number of data within the domain of interest in order to confine uncertainties to reasonable limits. Normally, directions are produced with uncertainties smaller than 15°, and can be further reduced to 10°, if rock heterogeneity remains minimal (Ask *et al.*, 2006).

For thrust regimes, the most important question concerns the magnitude of both the horizontal principal stresses. Only through complementary techniques will it be possible to ascertain the domain of confidence of the results. Indeed, whilst inverse problem theory provides means to take into account objective uncertainties on the various measurements, it does not evaluate the influence of simplifying hypotheses inherent to the stress measuring method under consideration; hence, the necessity to reach the results through different independent means. With this respect, the proposed combined hydraulic fracturing – HTPF methodology is self-consistent for determining the complete stress field and the solution for principal stress directions can be compared with those obtained by other means. The same can be said for stress magnitudes. At a later stage, if additional data are available at the site, integrated inversions may outline those data that are not consistent with the solution. In this manner it is possible to detect whether only specific tests cause problems, or whether many results obtained with a given technique constitutes the source of difficulty. A careful statistical analysis of the results provides an efficient evaluation of the validity of the final solution; hence, the necessity to produce a large volume of reliable data.

The next question is whether the proposed methodology will produce enough data for the statistical analysis to be reliable. For example, the diversity of orientations of open or partially open pre-existing fractures, within restricted depth intervals, may be limited. However, it was shown previously that horizontal stresses may be solved even for the ultimate case in which only one single direction is observed within a given restricted depth range (Ask *et al.*, 2006).

5. Conclusions

We present a strategy for determining the stress field and it's variation with depth. This strategy is not hampered by limitations in the classical hydraulic fracturing technique, even in thrust regime settings where the number of pre-existing fractures with different orientations is limited.

Although we focus on sampling the stress field with hydraulic methods, our strategy can be generalized to most stress measuring techniques as well as for integrated studies. A strong benefit of this integrated approach is that data from different sources may be compared and validated to an existing solution. Another benefit of data integration is that the final solution involves a considerably larger data set, which implies that the final solution becomes more reliable.

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EXTENDED ABSTRACTS

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1. TUNNELLING

1.1. General

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DISCONTINUUM ANALYSIS OF A HIGHWAY TUNNEL

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Tunnels are conveyance systems, used in the transportation sector for conveyance of highway and railway traffic. They also serve as a means of conveyance of water in irrigation and hydroelectric power projects. During construction of tunnels variation of strata is often encountered. This poses various problems in tunnelling. Historically tunnelling projects have suffered significant cost and time overrun on account of variation in geological formation. Variation in strata may lead to various problems like tunnel caving in, water seepage and many more. Also, the decision regarding the support system required for various strata is critical to ensure trouble free construction.

The proposed tunnel alignment encounters mainly fresh and hard Quartz-Mica Schist with a few prominent sheared rock zones. The geological set up of the area indicates that folds are not present in the rock bed and the general dip ranges from 15° to 30°. Generally the alignment runs more or less favourable to the strike of the foliation of the rock. The proposed tunnel runs through high overburden depths upto 1900 m covered with steep slopes, dense forest and perennial snow. Among the sections tried, the horse shoe section is found to be the most suitable section for such conditions of high in-situ stress and space requirement.

As the depth of overburden is varying along the tunnel alignment from 200 m to 1900 m, the entire tunnel length is divided in to five segments with vertical in-situ stress varying from 16.2 MPa to 52.3 MPa. The present study discusses about the elasto-plastic modelling of one of the five sections analysed. The vertical in-situ stresses likely to be encountered in this section are 32.4 MPa. The in-situ stress ratio in this region is 1.0. The computed Q value of the rock mass at the north and the south portals is 5.01 and 5.0 respectively which indicate that the rock mass for the proposed tunnel can be classified as "fair". Similarly the calculated value of RMR at the north and the south portals is 48 and 59 respectively, which also indicate "fair" rock condition in the region. However, the rock mass classification along the tunnel reach is likely to vary from good to very poor. This study is focussing on the fair rock mass condition. The tunnel is analysed through a rock mass having three prominent joint sets. One joint set each is horizontal, vertical and dipping at 15° inclination.

The stress-deformation response of the tunnel without support clearly indicates there is a release of stresses around the tunnel which indicates that the rock mass around the tunnel is fully deformed. The displacement vectors around the tunnel periphery show a deformation of 0.675 m at the crown level. The principal stress distribution and the displacement vectors for the tunnel with support system shows that the installed supports have effectively controlled the deformation around the tunnel. Due to the installation of the support the deformations around the tunnel have been reduced by almost 66%; the maximum deformation being 0.229 m. However in such structures which are subjected to high in-situ stresses, large deformations are occurring at the invert level. In such cases provision of support in the form of rock bolts along with shotcrete along the floor of the tunnel will reduce the deformation of the floor. The above analysis indicates that the proposed support system is effective in controlling the deformations around the tunnel.

Keywords: Tunnel; stress-strain response; support.

GROUND REACTION TO DEEP DRAINING TUNNELS

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Knowledge of the elastic induced stresses and deformations is the first step in computing the rock elasto-plastic reaction to deep tunnel excavation. The following steps are well defined but it is the first step that, in permeable rocks, is unlikely to be satisfied. The reason is that the closed elastic part of the solution suffers logarithmic and higher order divergences. Unlike tunnels deep caverns do not present any diverging phenomena and the paper will focus on the tunnel problem. A renormalization of seepage forces is necessary to eliminate diverging effects. Depending on the rock yield criterion and potential flow, stresses and displacements may be obtained in closed forms all over the rock mass. Anyway, first order Taylor expansions of the yield criterion and the plastic flow potential allow a close insight into local drainage effect. Drainage leads to an early failure, reduces cohesion and produces a greater radius of the elasto-plastic interface. A procedure that is compatible with analytical or numerical treatments is used to compute ground reaction which is useful for tunnel design and lining dimensioning. Tunnel deformation or ground reaction is defined by two variables that are the support pressure and the water inflow.

Keywords: Seepage forces; Water inflow; Lining; Tunnel; Cavern; Plasticity.

Table 1. Asymptotic radial behavior of the main variables in an infinite permeable rock mass.

	Cavern	Tunnel
Mechanical stress	r^{-3}	r^{-2}
Mechanical displacement	r^{-2}	r^{-1}
Pressure gradient	r^{-2}	r^{-1}
Water Pressure	r^{-1}	Log r
Water induced displacement	r^{0}	r Log r

BEHAVIOUR OF TUNNELS IN JOINTED ROCK MASS

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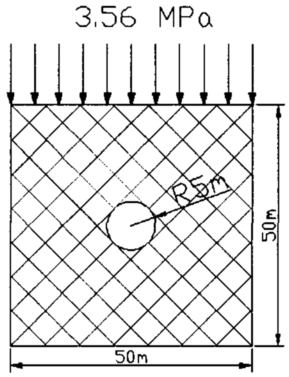
The stability of an underground opening is more or less dependent upon the strength of the rock formation and the state of stress around the opening. The strength of the rock formation is governed by the strength of its discontinuities, joints or bedding planes and should never be overlooked while analysing the underground opening. Numerical analysis on the stress – deformation response of the underground opening in intact rock has been carried out world over extensively using the continuum method. But, the behaviour of the complex rock mass with joints and bedding planes needs to be modelled using the discontinuum analysis. Further, the in-situ stresses play a major role in defining the behaviour of the underground opening. The paper focuses on the discontinuum analysis of an underground circular tunnel in a jointed rock mass idealized by an orthogonal joint set.

The parametric study on the 10m diameter circular tunnel consists of two variables: a) The variation of inclination of an orthogonal joint set of 5 m spacing viz. 15°, 30°, 45°, 60° and 75° from the horizontal b) Variation of In situ stress ratio viz. 0.5, 1 and 2. The effect of joint orientation and the effect of in-situ stress ratio on the deformation around the tunnel, at the crown, invert and at the spring lines are studied. In order to have an in-depth study of the above parameters, a rock mass of 50 m × 50 m around the tunnel has been chosen for finer zoning of the tunnel region. An overburden of 125 m of rock mass has been simulated by applying a uniform vertical stress of 3.56 MPa on the upper boundary of the problem geometry.



The effect of joint orientation and the in-situ stress ratio on the stress-deformation response of the tunnel has been studied. The study indicates that the displacements are maximum at the crown irrespective of the joint orientation and in-situ stress ratios. As the in-situ stress ratio increases, the displacement at the crown, invert and the spring lines increases irrespective of the joint inclinations except for the tunnel with 45° joint set. The analysis indicates that, the tunnel with the joint sets of 15° and 30° are symmetrical with the joint sets of 75° and 60°.

Keywords: Tunnel; rock mass; rock joint.

ON THE STABILITY OF A TWIN-TUBE TUNNEL UNDER COMPLEX GEOLOGY

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Designed as a twin-tube and a four-line tunnel, the under constructed Longtan Tunnel is a major part of the highway linking Shanghai and Chongqing city. With a total length of 8.695 km, it ranks the second longest in China. Averagely, it is 9.75m in width and 5.0m in height, with a spacing of 50 m between the two tubes. The maximum depth reaches up to 500m. The tunnel is located in limestone, the karst cavities, the fractures, the underground rivers and water bearing formations are expected in the surrounding rock. Moreover, an in-situ horizontal stress higher than 8M has been measured.

In this study, considering the underground water, the high in-situ stress, the rock joint sets and the combined supports, the stability of Longtan Tunnel is numerically investigated by using the commercial code UDEC. The simulated results agree well with the measured data. The results show that the designed support types can absolutely meet the need of deformation control and safety requirement. However, the rock bolt installation and the water-proof layer construction must be paid special attention. Note should be taken that the karst cavities, which usually exist in limestone but can not be included in the simulation, may possibly threat the tunnel construction safety. Figure 1 and Figure 2 illustrated the Contours of major principal stress and the pore water pressure in the surrounding rock of the tunnel.

Keywords: Highway tunnel; Stability; UDEC; In-situ stress; Support.

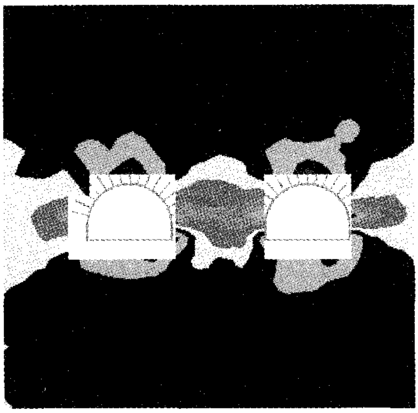

Fig. 1. Contours of major principal stress.

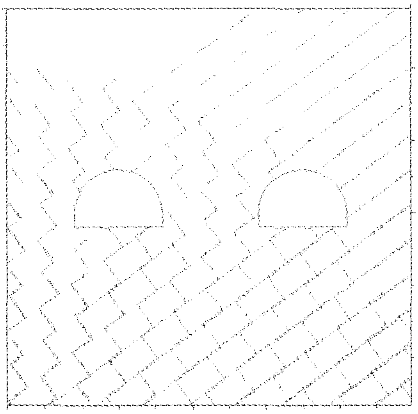

Fig. 2. Pore water pressure.

NEW APPROACH OF TUNNEL OBSERVATION USING DIGITAL PHOTOGRAMMETRY

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An innovative approach of digital vision system to get the absolute displacement of rock mass in tunnelling is presented. Computerized digital vision system can be seen as connecting one or more cameras to a computer and processing the acquired images on it, in order to get measurements of the recorded objects. A special imaging device based on a digital panoramic camera is used. It can take images at resolving fine object details within inside view of tunnel. At least two different images of a rock face are taken and then processed with proprietary software components, that manage the large data amount, handle the special geometry of the images and allow several kinds of image analyses, such as spatial inspection and geometric measurements. Using this system 3D displacement of rock mass behavior can be acquired with combination of total station system. The problem of using laser targets to get the measurement in digital vision system can be solved by combining points cloud projector with this system. As the data acquisition and analysis processes take place without physical contact, the measurements are taken indirectly. Measurements are made at an arbitrary number, without danger and pressure of time. The digital vision system maximizes support efficiency and improves excavation or outcrop planning, both reducing costs and increasing safety.

The new 3D absolute monitoring technology in combination with newly developed methods of data evaluation has improved the possibilities for the prediction ahead and outside of tunnel face. In particularly, in heterogeneous/anisotropic rock masses, the prediction of the vector orientation and function parameters plays an important role with respect to safety and economical aspects of a tunnel project.

Displacements are strongly influenced by the weakness bands, specially, more close to more weaker band, more stiffly larger the total amount of displacements, and the trajectory of the function parameters shows some relationship between the stiffness ratio and the distance from weakness band. Also, the thickness, strike and dip angle of the weakness band are dominantly influencing the displacement magnitude and trend, the convergence parameters and the stress redistribution around tunnel. Specially, in order to consider the anisotropic nature of rock masses, transversely isotropic or orthotropic rock mass models are studied on the effects of changing the tunnel convergence function parameters and displacement trajectories dependent on the tunnel excavation histories and support patterns.

Keywords: Tunnel; displacement; photogrammetry.

OPTIMIZATION OF THE ROUND LENGTH IN DESIGN STAGE FOR TUNNEL EXCAVATION IN WEAK ROCK

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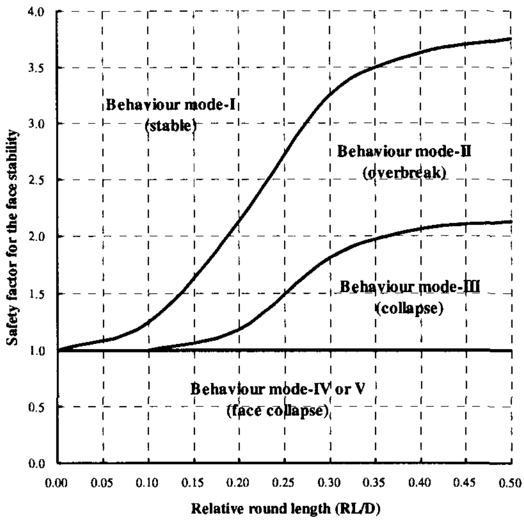
The behaviour mode of the face and unsupported span was investigated in this study for weak rock tunnels in shallow to medium overburden depth. Total five types of behaviour modes are suggested for the planning of excavation and support. Based on the results from the PFC3D analyses, the equivalent models were analyzed by a FDM code, using elastic material behaviour. Using the relative shear stress (RSS) concept, a correlation between the maximum relative shear stress (MRSS) and the different behaviour modes was found.

The safety factor for the face stability is defined by the concept of the 'critical cohesion' and it is formulated by fitting the results of FDM analyses. This safety factor can consider the influence of overburden, which Vermeer's equation disregards. The safety factor for the face stability is adopted as an indicator for the behaviour mode. The results are illustrated in the 'Conditional chart for excavation plan in weak rock tunneling' which shows the relation of the safety factor and relevant behaviour mode as round length varies.

With detailed construction information such as cycle time, unit price of materials etc., the optimization of excavation can be carried out in the design stage. According to the conditional chart, the behaviour mode can be determined with the applicable range of round length. Depending on the site conditions, round lengths causing a limited volume of overbreak can be considered in the excavation plan. Although the proposed method in this study has some restrictions, this method can provide useful information for the optimization of the excavation planning.

Keywords: Weak rock tunneling, Round length, Behaviour mode, Safety factor, Face stability, Optimization.



Conditions:
Overburden (H) < 150m
Tunnel diameter (D) = 5-15m
1< H/D < 15
Ko = 0.5 – 1.0
Friction angle (φ) = 20° - 40°
Cohesion (C) > 10 kPa
Round length < 5m
Top heading excavation (tunnel height is half of diameter).

Fig. 1. The conditional chart for excavation plan in weak rock tunneling.

THE IMPACT OF ROCK TUNNELLING ON GROUNDWATER IN EPI-FISSURE-KARST ZONE AND ECOLOGICAL CONDITIONS

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The water cycle in a karst mountainous area can be divided into two parts: shallow water cycle and conduit flow in the deep karst zone. Usually, in the tectonically upheaval zone (e.g., Laoshan mountains), the depth of conduit flow is very large. For Laoshan tunnel, the groundwater table of conduit flow is far below the tunnel elevation, thereby the tunnelling construction does not affect the deep water cycle and the regional water supply in this area. However, such engineering practice has great effect on the shallow water cycle, which is very important for the existence and growth of natural vegetation.

Although the karstification in Laoshan mountains is not intensive like that in the southern areas, there is a permeable and usually water-bearing layer in the shallow surface of carbonatite in Laoshan mountains. The permeable layer is a near-surface network of fissures, pores and karst-system, which is called Epi-Fissure-Karst zone (EFKZ) in the present study. Differing from Epikarst Zone, karstification is not the uppermost mechanism of the EFKZ. The EFKZ is an intensively fractured, weathered, root penetrated and karstified zone.

Monitoring results show that the groundwater in the EFKZ does not have uniform level and is closely related to the rainfall. When the tunnel approaches the monitoring boreholes, the drawdown of groundwater in the monitoring boreholes begins to increase significantly. The significant increase of water drawdown is because the tunnel excavation and rock blasting cause the damage of cements in the joints and thereby result in the increase of hydraulic conductivity of these joints.

Based on monitoring results and comprehensive analysis, this paper studies the influencing factors of tunnelling construction on the groundwater in EFKZ as a first-step research. Four factors including T (Tunnel depth and tunneling method), E (Epi-fissure-karst development and the soil cover), I (Infiltration condition and mechanical properties) and F (watery preferred Fracture) are presented and discussed. In the future, a prediction model including these factors will be proposed to assess the environmental impact of rock tunnelling and give corresponding methods to control the environmental consequence at a safe level.

Keywords: Epi-Fissure-Karst Zone (EFKZ); rock tunnelling; groundwater; environmental impact.

STUDY ON PREDICTION OF TUNNEL DEFORMATION CONSIDERING DEGRADATION OF ROCK MASS

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The authors have been studying on the quantitative criteria for determining remedial measures in consideration of deformation progress over time. In this paper, a rock mass strength degradation model to represent long-term deformations is developed to simulate long-term measurement data. Then, the impacts of long-term deformations on the lining are evaluated.

The purpose of this study, that is to design the deformation remedial measure in consideration of the time degradation of rock strength, the measurement and analytical results are combined to introduce the notion of time to the prediction of deformation progress. Figure1 is a schematic drawing of the deformation prediction method "**PAM**" (The Prediction Method of Tunnel deformation using Analysis and Measurement) that combines measurements and analysis. As shown in the figure, the relationship between the shear strength c and convergence u is obtained in the analysis. Here, therefore, the relationship between the measured convergence u and time t is fitted to establish the changes in shear strength c over time. Assuming the measurement period is from t_1 to t_2, the post- t_2 process represents the predicted values for future.

In order to quantitatively verify the displacement controlling effects obtained in the analysis, a comparative study was performed on the analytical and measurement results. The displacement controlling effects of remedial measures calculated by the rock mass degradation model are shown in Figure2. As shown in the figure, the convergence speed is significantly controlled by Remedial Measurement, and its displacement controlling effects are well reproduced in the analysis. Given the above, it is proved possible to quantitatively evaluate the remedial effects through analysis by the rock mass degradation model.

Keywords: Deformed Tunnel; Analysis; Measurement; Strength Degradation.

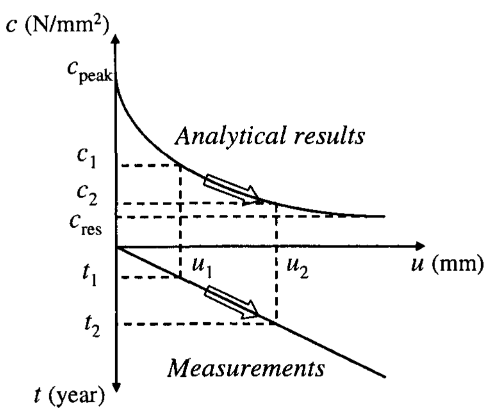

Fig. 1. Fitting the analytical results and measurements.

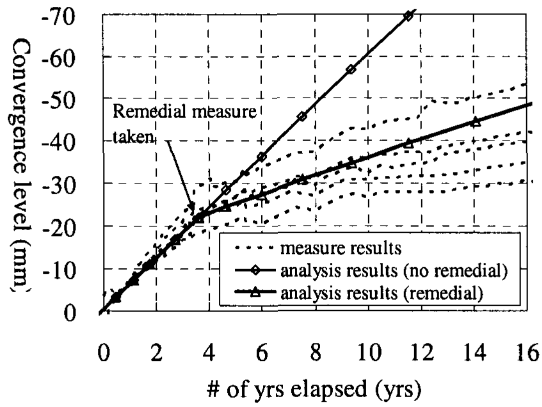

Fig. 2. Prediction results.

OBSERVATIONAL METHOD FOR TUNNEL CONSTRUCTION CONSIDERING ENVIROMENTAL IMPACT TO GROUNDWATER USING THE SWING METHOD

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Groundwater poses problems for the construction of mountain tunnels in that measures to prevent or control face collapse and tunnel drainage are needed due to the large quantities of water inflow that result from the discontinuous nature of the ground. There have been many cases in which such problems have inevitably led to enormous cost overruns and missed deadlines. Numerical analysis has been a primary method to evaluate such groundwater problems, but it requires the time and effort of preparing ground models of a certain scale and processing the data. For this reason, in almost all cases, the evaluation of groundwater problems has relied on preliminary evaluation throughout, and reverse or verification analysis rarely been performed during the construction. Moreover, even with current analysis tools it is difficult to conduct impact evaluations that reflect the preceding excavation data.

This study proposes a new observational system for mountain tunnel construction, the Simplified Seepage Analysis System ("SWING"), which enables streamlined construction with risk evaluation for cost fluctuations while at the same time considering measures to deal with groundwater problems. The SWING system is not a numerical analysis method using a ground model, as represented by seepage flow analysis or the modified tank model. Based on the water inflow incurred by actual tunnel excavation, the model is conducted with a unit slice volume of 50 meters in the excavation advance direction, and a hydraulic equation is applied within this slice volume to determine the coefficient of permeability and the effective porosity.

The SWING system was used to make predictions based on these initial settings. As the construction progressed, the excavated sections were reviewed based on the preceding construction data, and continued estimates of the as-yet-excavated sections were made. In the case study here, identification of the excavated sections and predictive evaluation for the as-yet-excavated section was done in about 2-3 hours, including correction of the input data. In some cases, the results of drain boring at the face were retrieved rapidly and determination of the quantity of water inflow during the main shaft excavation and predictions of the effect of the accompanying drought were done instantaneously. Therefore, this system can be considered to be one of sufficient practical value. The difficulties in determining the stability and permeability of the ground prior to tunnel construction often result in inevitable groundwater-related troubles. In addressing these problems, the SWING system is expected to play an important role in future tunnel construction, in that it allows the observation and measurement of the surface water and groundwater produced both inside and outside the tunnel shaft in the course of construction, as well as the immediate feedback of the results in design and construction.

Keywords: Tunnel, Groundwater, Observational Method.

NET PENETRATION RATE AND CUTTER CONSUMPTION IN JOOK-RYUNG TBM TUNNEL

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Full face mechanical excavation using hard rock TBM was applied to Jook-Ryung tunnel before it was enlarged by drill-blast. Two pilot tunnels with a total length of 7.3 km were excavated separately by two sets of TBMs with the diameters of 4.5 m and 5.0 m respectively in about one year. The tunnels passed through Precambrian gneiss complex, which would cause the low level of net penetration rate and the high level of cutter consumption. The small faults crossing the tunnel would also cause the degree of operation in low level.

This report analyzes the data of TBM performance such as net penetration rate, degree of operation and cutter life. A relationship between the net penetration rate and the elastic modulus of the rock mass which was obtained by TSP measurement, is also described as shown in Fig. 1. The results in this study are as follows:

- Lower peaks of the weekly advancing length can explain that the geological condition is adverse to the TBM performance. However, its upper peaks cannot exactly explain a better geological condition.
- The net penetration rate is found to go down as the rock mass class estimated by RMR and TSP is harder, which coincides with the previous studies. Thus TSP measurement can also be a qualitative tool to predict the tunnelling condition and the net penetration rate.
- The cutter life is estimated as 230 m^3/c and 420 m/set with 30% of error range, which can be applicable to wide range of the rock conditions in tunnelling design. The higher the net penetration rate is, the lower the cutter life in time and in the length of rotating trace shows. The life of the center cutter and the gauge cutter is very low, about 40~60% of that of the outer cutter.

This kind of assessment is expected to help designing the future projects and performing the TBM operation more effectively.

Keywords: TBM; Jook-Ryung tunnel; net penetration rate; degree of operation; cutter life.

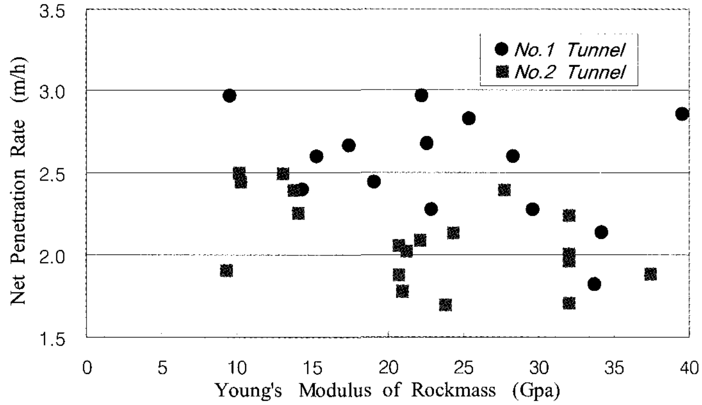

Fig. 1. Relationship between net penetration rate and Young's modulus.

STABILITY ANALYSIS OF SURROUNDING ROCK OF DEEP-LYING LONG TUNNELS

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Underground tunnel projects in alp and canyon area are always characterized by large overburden, long tunnel axis, and complex geological structure and so on. Accordingly, geological difficulties such as high earth pressure, high external water pressure, and water outburst will be met during the construction, which will bring much inconvenience. This paper takes the diversion tunnels and auxiliary tunnels of Jinping cascade II hydropower station for example, based on the data from the field measurement of auxiliary tunnels, analyzes the geological problems which may be met in the zone by numerical simulation, and through regression analysis, we find that the maximum principal stress along the tunnel axis comes up to 63MPa, and belongs to the super high earth pressure area. If routine reinforced concrete liner is used, the maximum external water pressure on the liner is about 11MPa, and it is very high in constructed engineering. The predicted steady quantity of outgush of tunnels is 5-6m^3/s. Rock burst and water outburst are prominent, rock burst occurs primarily in grade II surrounding rocks where the earth pressure is high, the rock is hard and rock integrity and quality are good, and in other locations occasionally. According to the characteristics and requirement of the project, and level of construction technology nowadays, we put forward the "based upon blocking up and combined with drainage" water-control countermeasure, and the design concept that surrounding rocks and supporting structure carry loads together. Based on this concept, preliminary supporting structure is proposed, the impacts of high external water pressure and high earth pressure on the stability of surrounding rocks are studied, and three-dimensional numerical analysis of the surrounding rocks and supporting structure is implemented, the calculated results show that the minimum external water pressure on tunnel liner reduces to 0.27MPa, and the maximum quantity of outgush reduces to 1.7m^3/s. Through optimization of the supporting structure design and construction sequence, development of plastic zone is limited, rock burst is controlled, and surrounding rocks are stable. Thus, when constructing underground buildings in deep-lying and water-rich regions, "based upon blocking up and combined with drainage" design concept can be used to reduce the external water pressure and the quantity of outgush, and the surrounding rock mass in grouting zone and the supporting structure can be combined to bear loads together, which ensures surrounding rocks and supporting structure are stable and safe. These measures can be used for references in the construction of future projects with similar nature.

Keywords: High earth pressure; High external water pressure; support design; stability analysis.

A TOOL FOR ROCK TUNNEL DESIGN BY CONVERGENCE-CONFINEMENT METHOD

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This study deals with the development of an easy-to-use tool to conduct the design of a circular rock tunnel based on the convergence-confinement method. The developed software consists of analyses of the longitudinal deformation profile, ground response curve, and support characteristic curve. A simple numerical stepwise procedure, modified from Brown et al. by considering the effects of elastic strain increments and variable dilatancy within the plastic region, is used for the evaluation of the ground response curve excavated in elastic-perfectly plastic, elastic-brittle-plastic or elastic-strain softening rock mass compatible with a Mohr-Coulomb yield criterion. The support systems by shotcrete/concrete rings, blocked steel sets, and ungrouted bolts and cables are considered for the support characteristic curves. An application of support design for a circular tunnel is given to explain the capacity of the developed tool. The advantages of the software are its user-friendliness, requirement of easily available input data, and faster evaluation time. Although the applicability of the developed tool is limited in scope, it appears to be useful for the preliminary design of a circular rock tunnel to obtain the reasonable support system.

Keywords: Tunnel design; Convergence-Confinement method; Ground response curve; Elastic-plastic rock.

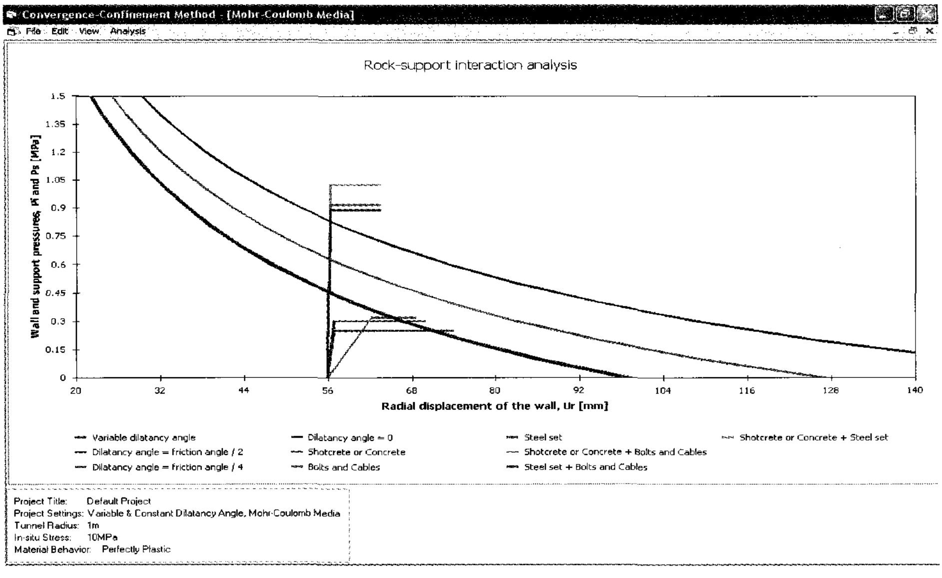

Fig. 1. Rock-support interaction analysis.

CONTRIBUTION TO THE DESIGN OF TUNNELS WITH PIPE ROOF SUPPORT

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In the last decades shallow tunnels were increasingly constructed in weak ground. These tunnels are often situated in urban areas, where project requirements, such as limited settlement requirements, constitute the necessary support. The experience gained from former tunnel projects indicate that the pipe roof support system not only increases the stability of the working face but also decreases the subsidence induced by the excavation. Due to these experiences a number of tunnels were additionally supported by this system without clear design rules for the determination of the design parameters.

On this account the design of the pipe roof support system is usually based on experience. By performing additional horizontal inclinometer measurements to supplement the state-of-the-art geodetic survey the first step for understanding the system behavior was done. On site this additional data can be used to optimize the construction process of pipe roof supported tunnels as well as to determine changes in the ground quality ahead of the face. For this study the detailed data in combination with laboratory data was used as input and control parameters for numerical investigations. The back calculation clarified at first the geotechnical model for the pipe roof support system. Afterwards the number and dimension of pipes was changed and the influence on the subsidence evaluated.

Even though the grout was neglected due to the ground conditions of the investigated projects the calculations clearly showed a decrease of the pre-settlement amounts up to 30 % at the tunnel level when using a pipe roof pre-support system. The different, investigated cases displayed that the decrease of pre-settlements increases with increasing number of pipes as well as with bigger dimensions.

Keywords: Shallow tunnel; weak ground; pipe roof support system; design parameters; numerical simulation; pre-support system.

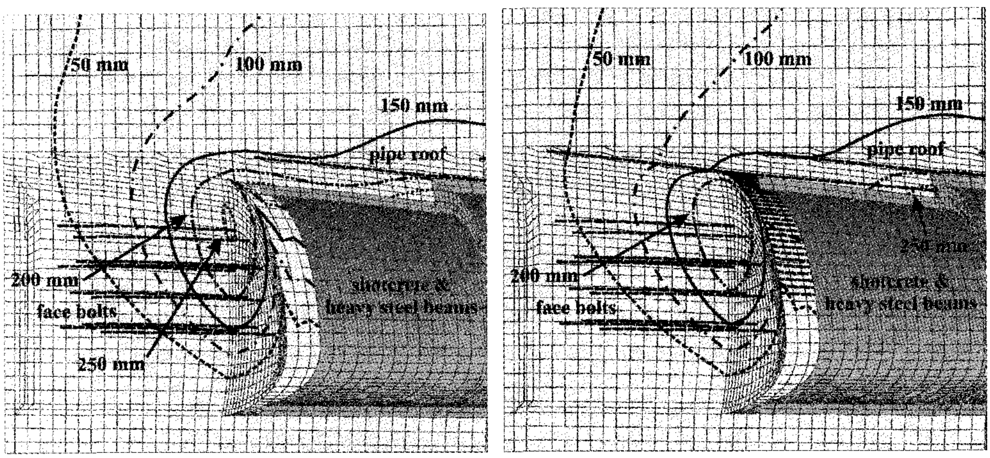

Fig. 1. Calculated settlement values without (left side) and with pre-support (30 pieces of 114.3 x 6.3) (right side).

ESTIMATION OF EXCAVATION DAMAGED ZONE OF LONG-SPAN AND SHALLOW OVERBURDEN TUNNEL

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Estimations of Excavation Damaged Zone (EDZ) are important for design and construction of tunnel, especially for long-span and shallow-overburden tunnel. The Hanjialing tunnel passes through the hilly areas around the Dalian City, in Liaoning province, China. The tunnel was a four-lane highway tunnel. A length of 760m of the relocation line was planed to Hanjialing tunnel that was the long-span and shallow-buried in China at that time. This tunnel is 21.242 m in width and 15.52m in height respectively. The most long distance from the top of tunnel to the ground is 189m. The main primary supports for this tunnel included shotcrete, wire mesh, H-beams and rockbolts. Due to the complexity of geological conditions and the efflorescent rock on the ground. A better understanding of the mechanics of influence, especially regarding the assessment of EDZ (Excavation Damaged Zone) is required.

In this paper, Firstly, geological characteristics of the long-span and shallowly-buried tunnel is introduced, based on the fieldwork of Hanjialing tunnel site, and the complexity of geological conditions and the efflorescent rock on the ground are analysised. Second, numerical experiments are conducted to investigate the effect of tunnel construction, the factors which affect the stability of large-span and shallowly-buried tunnel are studied with numerical method. Then, a better understanding of the mechanics of influence, especially regarding the risk assessment of faults is required. The main characteristics of stress induced brittle failure of the site are introduced. Forth, the regional stress of Excavation Damaged Zone (EDZ) in Hanjialing tunnel are estimated considering to different construction methods, As a result, the multi-layers excavating method (the drill and blasting method) of this tunnel and the pre-support method are suggested. In the end, some other meaningful suggestions about the construction of the large-span and shallowly-burden tunnel are suggested.

Keywords: Excavation Damaged Zone (EDZ); Long-span and shallow-overburden; Anisotropic rock; Construction method; Stability analysis; Design and construction.

DEFORMATION MONITORING ON THE DIVERGING TUNNEL AT BAZILING, P. R. CHINA

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In tunnelling engineering, deformation monitoring is a common means for selecting construction way, ensuring tunnel safety, and controlling tunnelling quality. The paper presents a site deformation study on a new structure tunnel at Baziling, which is one of the first tunnels using the diverging structure in P.R. China. The plane layouts of the tunnel and monitoring sections are shown in Fig. 1. The employment of the tunnel not only decreases the total investment, but also avoids potential slopes instability and environmental ruins around the tunnel export, which may be induced by other alternative ways. So the diverging tunnel is promising in communication construction in Central and West China or other mountainous areas.

Firstly, the geology and construction of the tunnel are introduced. Then, the monitoring plan is described detailedly. The special deformation characteristics of the tunnel are described in three aspects: longitudinal deformation distribution, dynamic deformation response on construction and spatial effects of working faces. Finally, the implications on construction technology are suggested based on the special deformation characteristics. From the study, the following conclusions can be drawn: the structure effect in the junction part between double-arched and clear-distance segments makes the part weaker part in the overall diverging tunnel, much consideration in the part should be taken into the construction of diverging tunnels; the introduction of part-face excavation presents the dynamic response of tunnel deformation on construction significantly, an optimal construction process could be devoted to improving the stability of the tunnel under construction; and spatial effects of working faces are shown remarkably in the double-arched and clear-distance segments, furthermore, the mutually spatial effects of adjacent working faces in the leading and following tubes are discussed in the study. The study provides indispensable evidence for the design and construction of the diverging tunnel.

Keywords: Diverging tunnel; deformation monitoring; analysis.

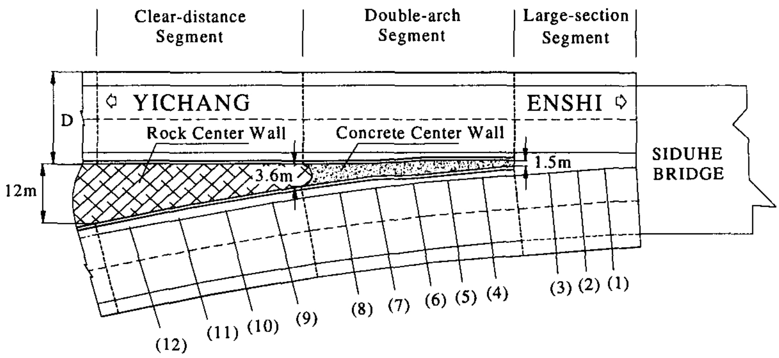

Fig. 1. Plane layouts of the diverging tunnel and monitoring sections at Baziling.

FUNDAMENTAL STUDY ON EXCAVATING CHARACTERISTIC OF ROCK TYPE SLURRY SHIELD IN SOFT-ROCK

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Rock type slurry shield method is used for pipe jacking in various rocks. In excavation soft rock, faceplates of this type are often blocked by sticky rock chips leading to the difficulties in excavation. This study is to research the cause of the blocking phenomenon and the method for preventing the blocking phenomenon in soft rock using rock semi shield. The experiment was using excavation model device. As a result of experiment, flow to the face space and reducing of the length of disk cutter were effective for preventing the blocking phenomenon.

We prepared a 1/5-scale model of the actual machine, with the same rock type slurry shield mechanism. While the cutter head rotates during excavation in the actual machine, the model machine advances without rotation for the sake of simplicity, and the mold (surrounding rock mass) is rotated instead. Similar to the actual machine, the model machine has a slurry chamber that allows slurry circulation through feed and drain pipes.

As a test piece, an artificial soft-rock was produced and used. In order to evaluate the predictions related to the blocking phenomenon of face plate when excavating, PI, sand containing percentage and unconfined compressive strength were adopted.

In order to prevent the face plate from blocking, there were conducted experiments both in case where the shape of cutter for the rock type slurry shield was changed and in case where clear water or muddy water is directly poured into the facing to study the effectiveness as a countermeasure against blocking of face plate.

The major findings can be summarized as follows:

(1) From the results of laboratory experiments using the model machine, the process of blocking during excavation can be described as follows: the flow rate of slurry in the face space is decreased, the amount of muck accumulated in the face space is increased, and the muck is compressed by thrust, is dewatered, and adhered to the cutter head.

(2) The results of model machine experiments indicate that supplying water directly to the face space and decreasing the cutter protrusion length are effective measures to prevent blocking. Further investigation is required to determine the optimum protrusion length of disc cutters.

(3) Torque index was used to set criteria for the judgement of blocking in the model machine experiments when water was direct supply to the face space in addition to the supply from the chamber. Under the standard conditions, torque index remains within the acceptable range and blocking does not occur if the flow rate of directly-supplied water is kept at 1.0 l/min or greater. There is no significant difference between the use of water and slurry regarding the prevention of blocking. However, muck is more effectively discharged when slurry is supplied.

Keywords: Rock type slurry shield, Excavation, Soft-rock, Blocking, Artificial soft-rock, Predictions.

1.2. Theoretical and Numerical Analyses

3-D AND QUASI-3-D ANALYSES OF UNDERGROUND EXCAVATIONS

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Constructions of underground excavations are of great interests to mining engineers. Geological features, geomechanical parameters of rock mass and stress conditions have essential role in designing of underground projects. In rock mechanics activities, empirical knowledge and engineering judgment play an important role. But with advancement of computers, numerical analyses of underground openings become crucial. Due to controlling influence that stress magnitude and its orientation have in the development of brittle fractures, rock strength degradation (i.e. damage) and rock mass instabilities, assessment (knowledge) of the induced stresses and deformations of the underground structure is important in designing the proper support system and it is necessary to incorporate the material properties, presence of discontinuities, nonhomogeniety and state of in situ stresses existing in the rock mass.

In this research, with use of FLAC3D code, Quasi-3-D (3-D model with thickness of unit) and 3-D continuum analyses of the extension phase of Masjed-E-Soleiman hydroelectric project, Khuzestan province, Iran, has been considered. It is assumed that the rock mass obeys Mohr-Coulomb criterion. Deformations and stress distribution in the periphery of the powerhouse cavern have been computed. Both the computed deformations and loads in bolts have been compared with in situ measurements. The study reveals that 3-D analysis has better agreement with in-situ measurements.

Keywords: Powerhouse cavern; 3-D model; Quasi-3-D model; numerical analysis; deformation.

ANALYSIS OF TIME-DEPENDENT TUNNEL BEHAVIOR USING A NEW ROCK-SUPPORT INTERACTION MODEL

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Excavation of tunnel releases the stress and then causes the displacement in rock mass. The support elements are installed in order to control the deformation behavior, and also this enables the tunnel to be safe. The property of shotcrete, which is one of the major supports in NATM, changes by hardening effect with time. Thus the time effect should be considered in the analysis of the rock-shotcrete interaction. On the long-term durability of shotcrete, the deterioration of shotcrete lining results from aging, deformation, fracturing and accumulation of defects. The deterioration of the shotcrete lining can be regarded as the time-dependent degradation of properties. The degradation of shotcrete properties causes the decrease of shotcrete durability and a serious safety problem.

In this study, a new rock-support interaction model, considered synthetically the time-dependent factors such as the rheological characteristic of rock mass and the hardening effect of shotcrete, is proposed. The proposed model can estimate the time-dependent tunnel convergence and support pressure for a circular tunnel excavated in the homogeneous isotropic rock mass subjected to hydrostatic stress state. A more realistic rock-support interaction is obtained by including the effects of the radial displacement at the tunnel face that cannot be measured directly in the field. Moreover, the factors affecting the rock-support interaction, such as rock condition, tunnel size and unsupported span, are investigated by using the proposed model.

Also, the effects of the shotcrete lining deterioration on the tunnel safety are studied. Exponential equations for shotcrete degradation are obtained from experimental data on the shotcrete properties varying with time in acidic fluids. The effects of the lining deterioration on the rock-support interaction are studied for two conditions: (1) time-independent deformation of rock and/or support, (2) time-dependent deformation of rock and/or support. The results of analysis for degradation using the theory of rock-support interaction show that the time-dependent characteristics of rock mass and shotcrete have a great influence on the tunnel behavior.

Keywords: Rock-Support Interaction, Time Dependency, Shotcrete Hardening, Shotcrete Deterioration.

MODELING COUPLED HYDRO-MECHANICAL RESPONSE OF HETEROGENEOUS FRACTURED ROCK DURING TUNNEL EXCAVATION

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The Jinping hydroelectric power generation plant with a maximum generating capacity of 4400MW is one of the major hydropower projects in China. The construction of the four water transmission tunnels is vital for this project. Coupled hydro-mechanical (HM) processes of heterogeneous fractured rock during the tunnel excavation under high initial stresses and high underground water pressures are analyzed in this paper.

A three-dimensional stochastic continuum model that accounts for the heterogeneity of the hydraulic properties of fractured rock has been developed to simulate the HM behavior. The heterogeneous fracture permeability field generation is performed with the sequential indicator simulation, which is a nonparametric geostatistical technique. The nonparametric technique allows the input of the permeability probability distribution of arbitrary shape and assignment of different isotropic or anisotropic correlation structures to different classes of the stochastic variable. The generated permeability fields are then mapped onto the computational grids for coupled hydro-mechanical analysis. Deformation-dependant permeability is used to simulate the evolution of hydraulic properties during tunnel excavation. The importance of material compressibility is also examined in this paper.

The results have shown that the radial displacements and plastic strain are smaller when considering the compressibility of fluid and solid phases. It is found that neglecting the material compressibility will result in an overestimation of rock failure. Comparison of the calculated inflow rates to the measured values indicates that the heterogeneous model gives a better prediction of inflow rates than the homogeneous model in the transient state. When the steady state is reached, the computed inflow rates agree well with the measured values, whether the permeability fields are considered heterogeneous or homogeneous. Due to highly permeable fractures in the heterogeneous continuum, an unsaturated zone is predicted, which is not found in homogeneous model results.

It can be concluded that the heterogeneity of fractured rock and compressibility of fluid and solid phases have considerable effects on the hydro-mechanical response during the excavation stage.

Keywords: Unsaturated flow; Heterogeneous fractured rock; Coupled HM; Stochastic continuum model.

THEORETICAL SOLUTIONS FOR NATM EXCAVATION IN SOFT ROCK UNDER NON-HYDROSTATIC IN-SITU STRESSES

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The Convergence Confinement Method (CCM) represents a classic and effective tool for the support system design in NATM, to compromises the complex nature of interaction between the rock mass and the supports with a general albeit simplified approach. However, the hydrostatic *in-situ* stress assumption limits its applicability. Under the angle-wise approximation assumption and supposing that rock mass satisfies the Mohr-Coulomb failure criterion and exhibits strain-softening behaviors, this paper proposes a set of theoretical solutions to account for NATM excavation in soft rock under non-hydrostatic in-situ stresses.

The proposed method is coded into program under the VB Development Environment, and an illustrative case is studied by the proposed method. The ground response curves at different elevation angles are depicted in Fig. 1. It is found that the non-hydrostatic in-situ stresses condition (supposing $P_v > P_h$) makes the rock mass at the horizontal elevation angle more disadvantageous due to the arch effect. Then combined with the support characteristic curve and using the philosophy of CCM, the proposed method can help the design of tunnel lining more effectively.

Keywords: Strain-softening behaviors; non-hydrostatic in-situ stresses; NATM; GRC; SCC.

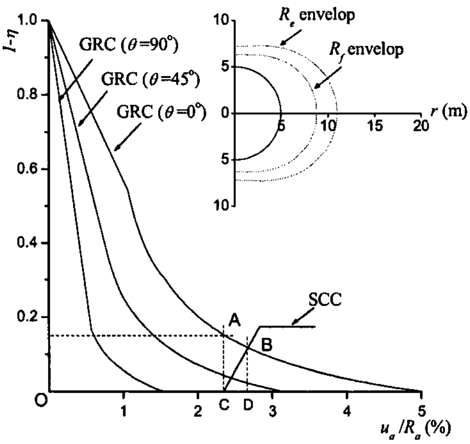

Fig. 1. The ground response curves and the schematically representation of the tunnel lining design.

3D NUMERICAL MODELING OF GATE SHAFTS AND SURROUNDING TUNNELS IN GOTVAND DAM PROJECT

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Gotvand dam is a rock fill type dam, under construction 25 km north of Shooshtar city, Khuzestan province in south of Iran. It incorporates 1000 Mega Watts hydropower plant. Water intake system consists of four tunnels, gate shafts and surge shafts. These underground structures will be under water pressure at time of operation, thus special care has been necessary in design and build stage. Diameter of the tunnels and shafts was 12.6 m and 13 m respectively. The shafts were closer than 2.5D to each other. This geometry could increase concerns over the stability of the structure.

Initial design of the tunnels and shafts has been done using experimental methods and rock classifications. Since excavation was done gradually in the top portion of the tunnels and pilots in the shafts, results of some monitoring and observation could be used to re-evaluate the initial design. Due to the geometry and intersection of the shafts and the tunnels, it was decided to use 3D modeling for the purpose of stability analysis. Flac3D was chosen as the suitable model for the area. Using the symmetry in the model, a 130 wide, 140 long, 90 high, model was constructed which included two tunnels and two shafts. The depth of the shafts was 70 m. Excavation of the tunnels and shafts was done gradually and changes in the rock displacement and stress was monitored. The models were run for different order of excavation in the tunnels and shafts. It was found that the order of excavation could, to some extent, affect the final stability. Some instability was observed around the intersections and optimum support was suggested to stabilize the area. Results of the modeling showed that support installation could prevent the possible major instabilities. Later observations were also in conforming to the theoretical results. This paper expresses details of the 3D modeling and reviews the difficulties encountered and also includes the results.

Keywords: Flac3d; shaft; tunnel; intersection; modeling; stability analysis.

ELASTIC-PLASTIC ANALYSIS OF CIRCULAR OPENINGS IN BROKEN SURROUNDING ROCKS

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In China, a lot of mines have been explored down to eight hundred meters, even more than one thousand meter. When reaching a certain degree mining depth, the surrounding rocks of an opening are often broken under high in-situ stress state. It is necessary to analyze the deformation behavior under the breaking condition of surrounding rock, in order to grasp the interaction mechanism of surrounding rock and support structure and make a more reasonable and reliable support design. In the case that the cross-section of mine openings is regarded as a long and circle, the elastic–brittle–plastic model is adopted in before studies.

In this paper, the mechanical behavior of surrounding rocks before and after failure is considered as an elastic-plastic model. That is, if the stress in the surrounding rocks reaches or exceeds its peak strength, the rock is considered to can be broken, its strength is reduced suddenly and the post-yielding softening behavior is followed; and then the peak strength is taken to simulate the mechanical state of the unbroken surrounding rocks in plastic state. Therefore, the surrounding rock mass are divided into three regions, broken region, plastic region and elastic region. The stress and strain relationship of the broken and plastic regions is determined by the Tresc criteria.

For a 2-D plane-strain circular opening of radius a in an infinite rock mass subjected to the action of a hydrostatic in-situ stress q, the distributions of stress, strain and displacement in the broken, plastic and elastic regions are obtained. And the relation of the radius Rc of the broken region and the radius Rp of plastic region is also obtained, this is

$$\sigma_{sp}\ln R_p-(\sigma_{sp}-\sigma_{sc})\ln R_c=q-\frac{1}{2}\sigma_{sp} \tag{19}$$

Finally, when $q=\sigma_{sp}$, three examples are given to express the distributions of radial and circumferential stresses. And using the results of this study, two working cases are discussed for the explanation of the stability of surrounding rock and the reinforcement (support) around the opening.

Keywords: Surrounding rock; broken region; Plastic region; Elastic region; Theoretical analytical method.

CONTINUUM METHODS FOR STRESS AND STABILITY ANALYSIS OF BOREHOLES AND TUNNELS

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Continuum methods for analysis of circular openings are reviewed including analytical, semi-analytical, elastic and elastoplastic solutions for linear and non-linear openings in isotropic and anisotropic *in situ* stress field. The major differences and similarities between tunnels and boreholes are summarized first. Then the definition of instability is introduced and the instability mechanics is discussed such as breakouts, plastic deformation, stress redistribution, weakening at the wall, radial fracture or bifurcation instability. It is indicated that borehole stability is a geomechanical issue: hydraulic and chemical factors are vital, but instability is a function of stress, strain and yield. First-order mechanical parameters are strength, stiffness (as a function of confining stress), permeability, and rock tendency to dilate. These are affected by extrinsic factors (temperature, ionic concentrations, and pore pressures), which – in turn - change in time. If intrinsic material properties change, analysis must employ a non-linear theory of some type (non-linear elasticity, elasto- or visco-plasticity, bifurcation analysis, etc.). It is indicated that quantitative borehole stability analysis can vary from statistical methods based on history and lithostratigraphy to fully coupled plasticity solutions implemented through the finite element method.

Finally, various stress analysis tools available for computing stresses and displacements around openings are reviewed, starting from the basic linear solutions through elastoplastic analyses to semi-analytical, non-linear elastic analysis where stiffness is related to confining stress, radius or strain, to bifurcation instability models, incremental plasticity models, and boundary element method-based non-linear solutions obtained for boreholes in anisotropic in situ stress field. It is pointed out that many forms of non-linearity can be introduced but not all are physically justified by the additional testing or computational effort required and the introduction of material non-linearity usually leads to differential equation formulations which can be solved by (simple) numerical methods, providing geometry and boundary conditions are simple.

It is highlighted in conclusions that useful design goals can more readily be achieved by careful goal definition and use of parametric analysis before numerical approaches are employed. Because of geological complexity, parametric analysis tools available are best viewed as aids to design, rather than fully predictive methods. Fully analytic solutions are limited to simplified constitutive laws and geometry, homogeneous rocks, and constant stress, pressure, and temperature boundary conditions. Continuum methods can be used with great success for boreholes, whereas better suited for tunnels are approaches which account for joints, blocks, the frictional resistance among them, and correct geometry which defines kinematic freedom. If varying boundary conditions, load path dependency of properties, geological heterogeneity including jointing, complex geometry, or sequential excavation are important, numerical methods are required.

Keywords: Stress/Stability Analysis, Circular Opening, Borehole, Tunnel, Non-linear Effects.

ELASTO-PLASTIC NUMERICAL SIMULATION OF DEEP CIRCULAR TUNNEL SUBJECTED TO NON-HYDROSTATIC LOADING

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In this paper, a circular tunnel in a rock mass subjected to non-hydrostatic loading is analyzed. The tunnel problem is idealized by an axi-symmetric assumption (homogeneous and isotropic etc.), a combination of a two-dimensional finite element analysis and a Fourier series. The rock mass is assumed to behave as an elasto-plastic material that is characterized by a cohesive frictional strength. The initial stress state in cylindrical co-ordinates is expressed as follows:

$$\begin{aligned}\sigma_r &= \frac{\sigma_x+\sigma_y}{2}+\frac{\sigma_x-\sigma_y}{2}\cos 2\theta \\ \sigma_\theta &= \frac{\sigma_x+\sigma_y}{2}-\frac{\sigma_x-\sigma_y}{2}\cos 2\theta \\ \tau_{r\theta} &= -\frac{\sigma_x-\sigma_y}{2}\sin 2\theta\end{aligned} \tag{1}$$

The analysis includes the cases related to the principal stress directions and the tunnel axis. Three cases for initial stress condition are assumed. Figure 5 shows variation of α, where α is a deconfining ratio which is obtained by comparing the tunnel wall displacements in this analysis with the plain strain convergence curves. It could be said that the relations for two longitudinal sections are almost the function of z/a. Figure 6 shows the relationship between $(1-\alpha)$ and u_r. From these two figures, we could learn again that the rapid decrease in $(1-\alpha)$ with the increase in u_r near the face. Needless to say, tunnel face stability during excavation is very important. In this paper, face extrusions are presented as well.

Keywords: Tunnel face, Non-hydrostatic stress state, Elasto-plastic, Fourier series.

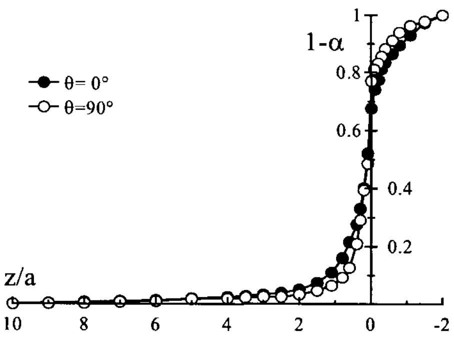

Fig. 5. Variation of α in two sections (Case 1).

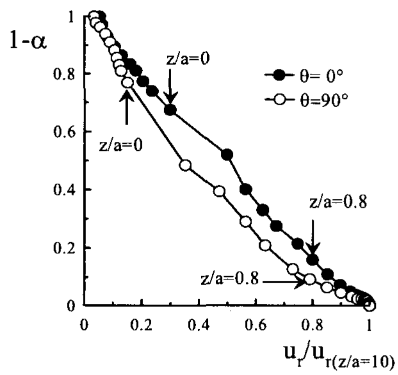

Fig. 6. Relation between α and $u_r/u_{r(z/a=10)}$.

NUMERICAL ANALYSIS OF THE CHANGE IN GROUNDWATER SYSTEM WITH TUNNEL EXCAVATION IN JOINTED ROCK MASS

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Proper prediction of groundwater table and groundwater inflows into a tunnel during and after tunneling is very important in the tunnel design stage. Groundwater inflows during tunneling prevent the progress of tunneling works and affect the stability of the tunnel. A drawdown of groundwater causes changes in the state of the ecosystem around the tunnel.

In this study, 2D finite-element model, SEEP/W, was adopted to investigate the amount of groundwater flowing into a tunnel and the groundwater table around wetland when the tunnel was excavated in rock mass. The analyses were carried out at four sites of the Wonhyo-tunnel in Cheonseong mountain (Gyeongnam, Korea) and the model domain of the tunnel included the wetland and fault zone. Anisotropic hydraulic conductivities of rock mass were calculated by using discrete fracture network (DFN) model and used as input parameters to the FEM modeling. To minimize uncertainties of hydraulic properties, analyses were performed for the parametric studies of the influencing factors such as hydraulic conductivities of the fault zone and grouting zone. The conductivity of fault zone was assumed to be 1.00×10^{-5}m /sec and 1.00×10^{-6}m/sec, respectively. The hydraulic conductivity of grouting zone was assumed to be 1/10, 1/50 and 1/100 of the conductivity of the rock mass. At the site where the ceiling of the tunnel was designed at a shallow depth, the increase of the hydraulic conductivity of the excavation damaged zone (EDZ) was considered. A total 6~8 cases of transient flow simulation was carried out at each site.

The hydraulic conductivities of fault zone showed a significant influence on the inflows into the tunnel when the fault zone and surface of tunnel were in contact. Groundwater table around wetlands maintained if the hydraulic conductivity of grouting zone was reduced to less than 1/50 of the conductivity of the rock mass at all analysis sites. When the conductivity of EDZ was 10 times higher than the conductivity of the undisturbed rock, the inflows into tunnel decreased by 40%, compared with those in case of 100 times higher.

Keywords: Groundwater; wetlands; continuum model.

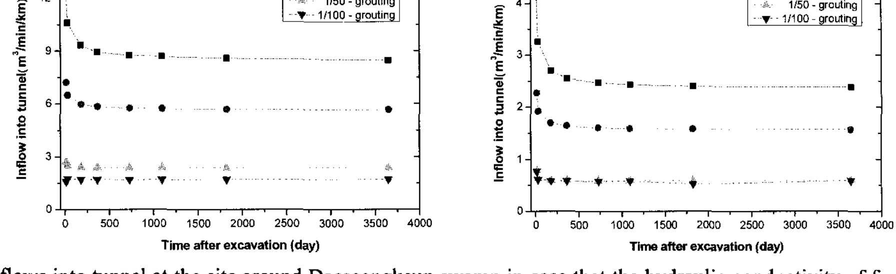

Fig. 1. Inflows into tunnel at the site around Daeseongkeun swamp in case that the hydraulic conductivity of fault zone is (a) 10^{-5}m/sec and (b) 10^{-6}m/sec.

A PARAMETER STUDY OF THE DAMAGED ROCK ZONE AROUND SHALLOW TUNNELS IN BRITTLE ROCK MASS

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As part of an ongoing study on the mechanical characteristics of the damaged rock zone (DRZ) around underground excavations and its influence on the overall performance of the excavation, a parameter study was carried out using the continuum method of numerical analysis. This assisted in identifying the sensitivity of the parameters tested, thereby laying the groundwork for further numerical investigation. The main parameters investigated include the extent of DRZ, the strength and stiffness of DRZ, and the depth of the excavation. The main quantity used to test the sensitivity of these parameters is the response of induced stresses around the excavation boundary resulting from the variation in these parameters. In the study presented in this paper, typical in-situ stress and rock mass conditions observed for shallow tunnelling projects in Sweden are used. The results clearly demonstrate the effect of DRZ on the excavation as well as the criticality of various parameter combinations. The strength of DRZ although sensitive, is the most complex to define and the tool used in this study to determine the empirical plastic strength components, namely cohesion and friction, is felt to be uncertain.

Keywords: Damaged rock zone, shallow tunnel, strength, stiffness, parameter study.

STUDY ON TUNNEL STABILITY IN SOFT ROCK CONSIDERING VOLUMETRIC STRAIN USING COUPLED ANALYSES

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The seepage of the groundwater depends on both permeability coefficient and storage coefficient around the underground caverns. In both soft rock and hard rock with discontinuities, dilation occurs in the mechanical process, and it is assumed that permeability does change. In particular, volumetric strain appears in soft rock and it is thought that there is correlation between the volumetric strain and permeability because soft rock is porous media.

In this study, we used several parameters of ground obtained through the theoretical resolution in one element using an elasto-viscoplastic constitution with strain-softening. Figure 1 shows the theoretical resolution using each parameter. In Case 1, volumetric strain shows dilation behavior. The result of triaxial CD test of soft rock under 0.1MPa confining stress is also shown in Fig. 1. It is considered that parameters in Case 1 are proper to describe the soft rock from comparison. In Case 3, different parameter is given, and volumetric strain shows compression behavior.

In this study, we performed a numerical analysis of an excavation using an elasto-viscoplastic 2-D finite element method which considers the seepage and the characteristics of strain softening and hardening. We changed the parameters which affect the behavior of the volumetric strain, that are obtained through one-element theoretical analysis. One shows the dilaion behavior on the volumetric strain, the other shows the compression. We used these parameters to change the permeability in the model in each case. Through this parametric study, we examined the effect on the stability of cavities by changing the material parameter of the ground.

Keywords: FEM; Coupled analysis; Strain-Softening; Volumetric strain; Permeability.

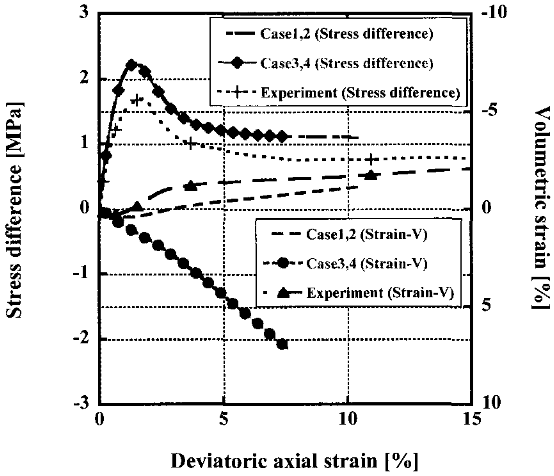

Fig. 1. Theoretical solution and experiment data.

BOUNDARY ELEMENT ANALYSIS OF TUNNELING THROUGH A WEAK ZONE

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Generally, a weak zone can be considered as a layered system composed of formations with different material properties. In order to understand the potential mechanism for material instability in relation to the material contrast, it is essential to consider the problem as a multi-layered system. For the analysis of multi-layered elastic media, a linear variation displacement discontinuity method was developed based on a scheme superposing two sets of bonded half-planes and subtracting one infinite plane. The model was verified before applied to the study of the fracture zone behaviour of tunneling through a weak zone. The results show that the influence of the material contrast on the stress distribution might be insignificant. However, the fracture zone behaviour, especially the fracture patterns, can be quite different for different material contrasts.

A special boundary element method has been developed for the simulation of the development of fracture zone in rock mass. This approach comprises a boundary element method and a tessellation mesh generator. The displacement discontinuity element, an indirect boundary element method, is good for fracture simulation. And, linear variation of stress or displacement quantities is embedded for better accuracy. By using the tessellation generator, a set of pre-defined potential fractures is defined with a Delanauy triangulation procedure. The fractures activate when the failure criterion is exceeded and are included into the displacement discontinuity elements. The final activated fractures near excavation can be determined by the sequential activation calculation process.

The results shows that the influence of material contrast on the stress distribution might not be very significant. However, the fracture zone development, especially the fracture patterns, can be strongly influenced by the interfaces between layers with material constrasts.

Keywords: Tunnel mechanics, numerical simulation, boundary element method, weak zone, rock mechanics.

PHYSICAL AND NUMERICAL MODELLING OF UNDERGROUND OPENING IN JOINTED ROCK MASS

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Two large sized physical model tests were performed to investigate closure of underground openings of inverted U and circular shapes subjected to different *in-situ* stress conditions. The tests were conducted under plane stress conditions. A model material (UCS = 7.0 MPa) was used to simulate intact rock. Blocks of the model material having size 25mm x 25mm x 75mm were arranged to form a jointed mass of size 750mm x 750mm x 150mm and having two orthogonal joint sets. A biaxial loading frame was used to apply *in-situ* stresses. After loading the mass to specific *in-situ* stress level, the openings were excavated at the centre of the mass and deformations occurring around the periphery of openings were recorded. If the deformations were found to be negligible, the size of the opening was increased. The following are the details of the tests.

Test-I: Dip of joints = 00°/90°; *In-situ* stress before excavation $\sigma_x = 2.45$ MPa, $\sigma_y = 2.48$ MPa (Hydrostatic); Shape of the opening = inverted 'U' shape. No significant closure was observed for an opening upto size 15cm under hydrostatic state. After changing *in-situ* stresses to $\sigma_x = 2.17$ MPa and $\sigma_y = 2.66$ MPa appreciable deformations were observed.

Test-II: Dip of the joints = 45°/45°; *In-situ* stress $\sigma_x = 1.38$ MPa, $\sigma_y = 2.36$ MPa (Biaxial); Shape of opening = circular; No sign of appreciable deformations were observed for 10 cm circular opening. Maximum deformation of about 2 mm was observed for the opening size of 18 cm. After altering the *in-situ* stress state to $\sigma_x = 0.086$ MPa and $\sigma_y = 2.95$ MPa (nearly uniaxial), appreciable deformations were observed at this stage.

UDEC models were developed to simulate the closure. The comparison of results from UDEC and physical model test-II are shown in Fig.1. It is concluded that the closure is anisotropic and depends on the configuration of joints, orientation of joints with respect to the *in-situ* stress field and principal stress difference. The relative size of the opening with respect to the size of the block is another important parameter governing the closure.

Keywords: Jointed rock mass; Underground opening; Closure.

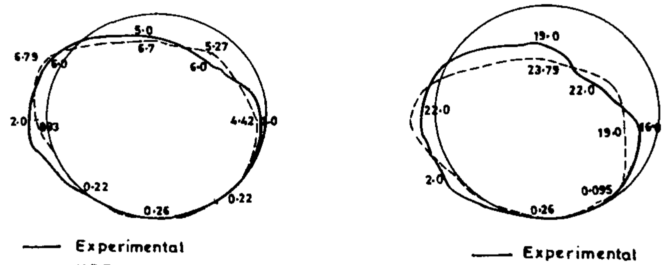

Fig. 1. Comparison of experimental and UDEC results (test-II).

THREE DIMENSIONAL MODELLING OF A TUNNEL CAVE-IN AND SPILING BOLT SUPPORT

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This article is a part of a PhD thesis, in which a tunnel in Vietnam with a serious cave-in is analysed. The tunnel is Buon Kuop headrace tunnel, which is located 60 m below the surface and in young sedimentary rocks. There is a 15-20 m wide weakness zone cutting across the tunnel. The weakness zone caused a cave-in in the tunnel and created a sinkhole on the surface. In the thesis, the cave-in and sinkhole are analysed thoroughly by several methods including numerical analysis. However, this article focuses on only two issues: rock mass parameter study and modelling for spiling bolts.

The equivalent Mohr-Coulomb material is used. The challenge is to obtain the most reasonable values for cohesion and friction angle from the verified Hoek-Brown material. Fitting technique that was used in Hoek et al (2002) does not provide a reasonable result concerning the deformation. Hence, the equivalent cohesion and friction angle of the rock mass are obtained by using a different way. With a comprehensive parameter study, the models are able to reproduce some practical results concerning cave-in, sinkhole, and spiling bolts performance.

It may be the first time – as far the authors know – that spiling bolts are modelled using pile elements in FLAC3D. With pile elements, the models are able to describe a correct and a more complete picture of the supporting action of spiling bolts. This goal is achieved by including the ability to carry out bending momentum for the bolts. The model also shows that some of the input parameters for the "pile elements" spiling bolt are difficult to quantify. These input parameters are only possible to obtain by in situ tests. References have to be made so that input parameters, to some extent, can be obtained. The FLAC3D models show a great improvement of the roof stability when the outer ends of the spiling bolts in the tunnel periphery are properly fixed. Forces acting on bolts and the bolts deformation are also studied. The models indicate that with spiling bolts in the roof, the instability problem will move from the roof to the floor ("umbrella" effect of the spiling bolts). Floor instability can be easier handle than the one on the roof.

Keywords: Weakness zone, cave-in, sinkhole, FLAC3D, pile element, spiling bolts.

CIRCULAR TUNNEL ELASTIC-PLASTIC ANALYSIS

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Presently, the Mohr-Coulomb criterion is often used in elastic-plastic analysis of stress field of tunnel surrounded with rock mass. However, it doesn't consider the middle principal stress. Circular tunnel with integrated rock mass is studied in this paper. The rock is considered as the 2nd kind of rock material in Miller, R. P's experiment. Based on the above assumption, a new elastic-plastic solution of circular tunnel is derived by using the unified strength theory. This new solution includes the stress of surrounding rock mass, plastic area radius Eq. (1), etc.

The expression of η and ξ (the introduced parameter) has been deduced in two strength indexes system α, f_c and C, φ. Then author has analyzed the relationship among different strength indexes by curves. The solutions based on modified Fenner formula and Kastner formula are the reduced forms of in new elastic-plastic solutions when b=0.

$$R = r_0 \left[\frac{(\xi + P\eta)(2-\eta)}{2(\xi + \sigma_0 \eta)} \right]^{\frac{1}{\eta}} \tag{1}$$

Finally, some new conclusions have obtained:

(i) This paper presents introduction parameters η and ξ under expressions of two different strength indexes systems α, f_c and C, φ. At the same time, it made relationship graphs of introduction parameters and different strength indexes in the range of 0<b<1.

(ii) A series of elastic-plastic new solutions in this paper is a group of solutions and has its own range. Different b correspond to solutions under different yield criterion. b=0(Mohr-Coulomb field criterion) and b=1(twin shear field criterion) determine upper limit and lower limit of the group of solutions; For instance, modified Fenner formula and Kastner formula are degraded forms under solution of b=0 in this paper.

(iii) Twin shear unified yield criterion is recommended through analysis in this paper. It's better to exert strength potential of elastic-plastic rock material and have objective cognizance of surrounding rock self-bearing capacity, so as to well supervise tunnel construction and economize investment.

Keywords: Unified strength theory; Surrounding rock mass; Stress field; Elastic-plastic solution.

NUMERICAL STUDY OF CAVITY UNLOADING IN BRITTLE-PLASTIC ROCK

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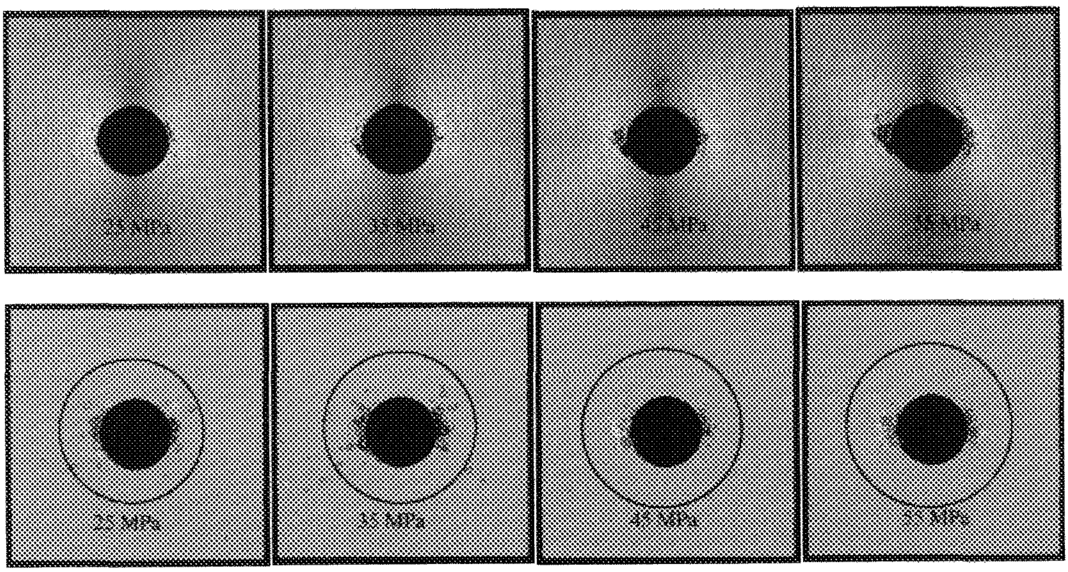
The prediction of the wall movements of tunnels (boreholes) excavated in rock at great depth is an important problem in rock mechanics and mining engineering. Based on the homogeneous and isotropic material of rock, it is well established that cavity contraction theory can be used to obtain reasonable solutions for this type of problems when circular tunnels are considered. However when considering the heterogeneity of rock and the failure process of rock, analytical method has been proved helpless to solve such problems. Therefore, a numerical code called RFPA (rock failure process analysis) was used to simulate the fractures evolution process due to cavity unloading. The numerical results can explained the evolution of borehole breakout shape and how the shape of the breakout related to the direction of the stresses in the rock.

Keywords: Rock; Cavity Unloading; Failure; Collapse; Numerical Simulation.



(a)

(b)

Fig. 5. Distribution of stress and AE events during the formation of notch around tunnel when the loading condition k=1/3.

NUMERICAL SIMULATION FOR SHALLOW TUNNEL UNDER UNSYMMETRICAL PRESSURE

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The Hurongxi highway that is being constructed is one of the important parts of the central artery of Hurong road. It includes many shallow tunnels under unsymmetrical pressure due to mountain chain of precipitous topography and complex geological condition. One instance out of many is Longtan tunnel. The burying depth of its outlet section is less than 10m. In tunnel engineering, the relative convergence of side walls, the ratio of convergence, the range of plastic zone and whether the vault collapses are considered as the criterions to estimate the stability of tunnels. However, in designing very shallow tunnels, the proximity of the ground surface and the tendency for the material ahead of and above the tunnel face to "cave to surface" have to be taken into account (Evert Hoek, 2004). Furthermore, Longtan tunnel is under unsymmetrical pressure. So it is not applicable to use the traditional analysis method such as confinement method to solve the problem.

In this paper, full three-dimensional FLAC^{3D} model is used to simulate the excavation and supporting process of the exit section of Longtan tunnel. The Mohr Coulomb criterion is used with the cohesive strength and angle of friction determined from tests. Firstly, different excavation sequences are performed and then the optional one is obtained by comparing the effects of excavation sequence on the response of the rock mass surrounding the tunnel. Secondly, two cases including with support and without support are analyzed on the basis of the optional excavation sequence. The stabilities of ground surface and face of the tunnel are also analyzed. Computed results and monitored results are compared. Finally some useful conclusions are given.

By means of Longtan tunnel, some of the problems of numerical modeling for shallow tunnels under unsymmetrical pressure are discussed and practical solutions to these problems are presented. Several useful conclusions can be used to conduct the later construction of Longtan tunnel and other tunnels under similar condition.

Keywords: Numerical Simulation; Shallow Tunnel; Unsymmetrical Pressure; Stability.

TECHNIQUE FOR DETERMINATION OF BOUNDARY STRESS CONDITIONS IN DEEP TUNNELING

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The natural stress state in rock masses is of fundamental importance for overall stability analysis of underground construction. Direct measurement of the far-field state of stress is difficult and costly due to the complexity of geological environment. Thus, determination of the initial stress state through back analysis utilizing measured displacements or local stresses at certain points is a very worthwhile undertaking which is complicated with high nonlinearity due to nonlinear behavior of rock masses. Different optimization theories have been developed to solve the optimizing problem.

In this paper, we developed a back-analysis technique based on Genetic Algorithms (GAs) to determine the boundary stress condition for stability analysis of rock masses with nonlinear behavior around a tunnel. The GAs are numerical search tools for finding the global optimum of a given real objective function of one or more real variables, possibly subject to various linear or non linear constraints. The search procedures provided by the GAs resemble certain principles of natural selection, genetics and evolution, during which no knowledge or gradient information about the objective function is needed.

The GAs optimized back analysis technique is then applied to a synthetic example of a deep tunnel in yielding rock. A nonlinear 2-D plane strain finite element model is used for the prediction of the behavior of the excavation system, in which the rock mass is numerically simulated as a non-tension elastic-plastic material. Displacements of the tunnel wall, as well as the local stresses are used to identify the boundary stress conditions. The magnitude and orientation parameters of the principle stresses are estimated by a real-coded GA whose objective function is the root mean square of the residuals between the pseudo-measuring data, obtained with the finite element model using true values of the parameters, and the prediction results computed with the same model using the estimated values of the parameters. The results indicate that the present method is capable of estimating the boundary stress parameters values with stable and good convergence. Numerical experiments are also carried out to check the influences of position and number of measurements to the reliability of the back-analysis results. Furthermore, the sensitivity analysis of the GAs optimization procedure is discussed in terms of identification of boundary stress parameters.

Keywords: Back-analysis; natural stress state; boundary stress conditions; genetic algorithms; finite element; tunneling.

1.3. Field and Laboratory Studies

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STUDY ON MINIMAL ROCK COVER AND ROUTE OPTIMAL SCHEME OF SUBSEA ROAD TUNNEL

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This paper investigates the subsea tunnels all over the world and concludes the three methods, such as Norway experience, the minimum leakage method and coal cutting experience under water. Norway experience is that Norway experts present the minimum rock cover under sea as function of the water-depth according to construction experience on Norway subsea tunnels. The minimum leakage method of Japan is that experts postulate the tunnel lies in a homogenous media with constant permeability in all directions so that they obtain the optimal rock covers of subsea tunnel. The leakage into a rock tunnel can be the least under the rock covers according to formulate (1):

$$Q = 2\pi kL \frac{H + h}{\ln\left(\frac{2h}{r}\right)} \tag{1}$$

The coal cutting experience under water is that experts calculate the minimum rock cover in order to avoid water filling into the tunnel according to experience on rational height of safety coal and rock pillars under soft water bearings.According to above-motioned methods, the engineering analogy is used to decide the minimum rock cover of a subsea tunnel at Xiang Shan harbor. The results of three methods are thought over overall. Meanwhile its geology is considered. Then the optimal rock cover is gained. To take into account the subsea landform and geology, one profile is chosen to be the reference profile. According to the design gradient of the subsea, the tunnel motherboards and the rock covers of profiles are obtained. Comparing those with the results of engineering analogy, the shortcut of tunnel is got under the safety condition by adjusting the gradient of the part of the tunnel motherboards, so the optimal scheme to construct the subsea road tunnel is decided.

Keywords: Subsea tunnel, minimum rock cover, maximal gradient, optimal scheme.

TBM BREAKDOWN CAUSES AND EFFECTS ON TUNNELING PERFORMANCE IN TARABYA SEWERAGE TUNNEL

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The performance of a full face tunnel boring machine (Herrenkecht TBM) was evaluated within the research program of this study. TBM having a diameter of 2.9 m and cutting power of 560 kW was used to excavate the tunnel of 13270 m in length, which was situated between Sariyer and Baltalimani. The main rock formations are paleozoik aged sedimentary rocks, limestone, sandstone-siltstone, andesite dykes and sediment. The project was commissioned to Tinsa – Öztas – Hazinedaroglu – Simelko consortium. The performance of TBM was recorded continuously for detailed shift analysis.

TBM assembly labors are recorded during site observations. TBM is ready to bore after 30 days of arrival to site. The cutterhead was overhauled in shaft S11. Therefore performance of TBM analyzed in two sections as shafts S13-S11 and S11-S10. Machine utilisation and advance rate in S13-S11 are 28% and 0.32 m/h. After overhauling the cutterhead TBM performance is increased as 35% machine utilisation and 0.51 m/h advance rate. Before the overhaul the average shift advance and average monthly advance are 2.43 m and 221,1 m. After overhaul they are increased 69% (4.11 m) and 40% (309.42 m) respectively.

TBM downtime causes are grouped for analysis such as cutterhead and shield, cutters, back-up, non TBM caused downtimes. Frequencies and durations of the downtime causes are compared for shafts S13-S11 and S11-S10.

Breakdown causes implies the important TBM design points. Cutterhead design, cutter mount design, segment crane and place of surveying system computer are important design parameters that effected TBM performance. Organization and planning are the other key points for the economics of a project that uses TBM.

Keywords: Tunnel Boring Machine; TBM performance; TBM breakdowns.

EXPERIENCE ON THE WORLD'S LONGEST RAILWAY TUNNEL ST. GOTTHARD

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With two 57 km long stretches, the St. Gotthard Base tunnel represents the world's longest railway tunnel. Not only because of its length rather because mechanized tunnel excavation enters new dimensions in terms of size, geological complexity and depth of underground constructions this project is of interest to the general public, contractors and planners.

The St. Gotthard tunnel project that is being carried out as part of the AlpTransit Railway Project in Switzerland requires an extremely high level of implementation accuracy with regard to the necessary performance rates and rock support of the bored tunnels. The tunnel excavations through generally extremely hard and abrasive rocks but also through challenging fault and disturbance zones put staff and technology to the test.

To meet the complex project demands, that have been unique in international tunnelling up to now, four Single Gripper Tunnel Boring Machines (TBMs) are deployed for crossing sections of the St. Gotthard massif. They currently bore full-face through the Gotthard Massif with the relevant back-up logistics. Southbound, two machines excavate the 'Amsteg' tunnel sections, each with a length of approx. 11.4 km; northbound, two machines bore the tunnel sections 'Bodio/Faido', each with approx. 25.5 km and 27 km respectively. Overall, the four tunnel boring machines will excavate and secure 75 km of tunnel, at overburdens of up to 2,300 m including some through demanding fault zones.

The concept for the implementation of this technically challenging task is based on the following principles: high safety standards, spaciousness in the front machine area for the staff, smooth allocation of processes between the machine and the staff and construction of a top-quality tunnel. In terms of design, the open Gripper TBMs are adapted to the requirements of the client. This article focuses on the experience, latest developments and trends of Gripper technology, based on the worldwide unique reference project, the St. Gotthard Railway Tunnel in Switzerland.

Keywords: Tunnel; case study; tunnel boring machine.

A MICROSEISMIC MONITORING TRIAL FOR THE STABILITY ASSESSMENT OF A SUPER TUNNEL AT JINPING DAM, CHINA

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The Jin Ping hydropower dam is one of the biggest hydropower dams currently under construction in China. In order to construct the dam, two super tunnels are being constructed for the purpose of water diversion. The dam site is under high stress. Faults and fracture zones were well developed. Several significant roof falls were encountered during the tunnel excavation, so the stability of the tunnels during and after the construction becomes a concern. In order to understand the mechanisms of fracturing and structure movement near the tunnels, a microseismic monitoring project was carried out at a supper tunnel site. The microseismic results have shown that rock fractures were developed at the intersection of the water, traffic and exploration tunnels where rock weakening between these tunnels might be developed (Figure 1). Fracturing was also detected where a significant roof fall occurred nearby. The microseismic results have provided good information for engineers to understand stress conditions in the rock mass and to assess the stability of the tunnel. This project has also identifies several challenging aspects in using microseismic monitoring techniques for the assessment of tunnel stability during its construction.

Keywords: Stability; tunnel; dam.

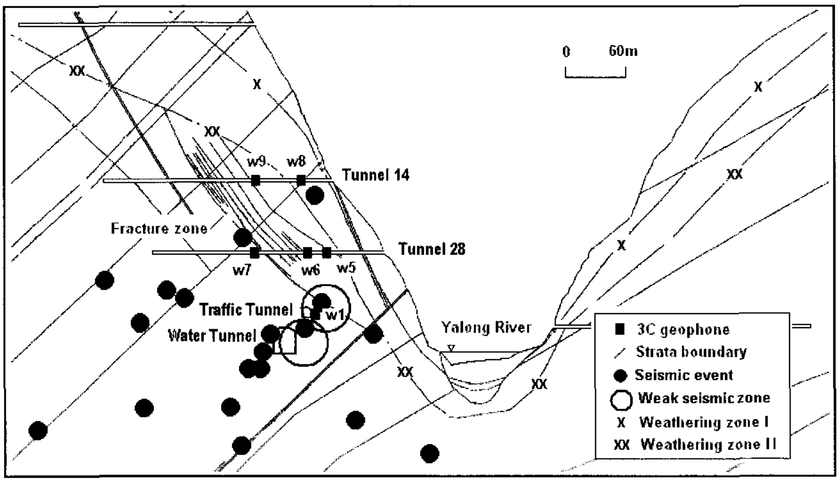

Fig. 1. The locations of weak microseismic zones and events in a vertical cross section near a water tunnel at the Jinping hydropower construction site.

STRESS-STRAIN ANALYSIS OF "HS KOZJAK" TUNNEL DUE TO MOVEMENTS IN TECTONIC FAULT

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The construction of pipeline's tunnel of pumping hydroelectric station Kozjak is foreseen. It is composed of three main parts: engine house, reservoir (accumulation lake), and the pipeline that connects engine house and reservoir. The engine house is located near river Drava, reservoir is on the 700 m higher plateau of Kozjak, and the pipeline is few ten meters under the eastern slope of Kozjak.

The geological characteristics were preliminary investigated in 1979 and 1980. The further activities of the project were stopped, until it was anew activated in year 2004. The project is in the outline scheme phase. The layer of gravelly sandy silt soils of thickness 0.5 to 2.0 m overlays rock base. Retrograded altered rocks lay in superficial stratum on the top of the considered slope. They are weathered in the upper 30 m thick layer, and deeper strongly cracked. At the toe of the slope appear above all two rocks, marble and calcite schist. Later forms cross to the marble as distinctive carbonated metamorphosed rock and common biotite muscovite micaschist and gneisses. Marbles and calcite schist alternate irregular. Represented rocks are classified into ten categories regarding to the degree of faults. First five categories represents weathered and intensively faulted rocks with $RQD < 30$ and $Q < 1$. In higher categories slightly to moderately faulted rocks are ranged. The laying out mainly lays in the layers between categories VII and IX, and it crosses eleven tectonic faults with system N-S and NW-SE. At these places the rock is crumbled, and due to the water flow weathered. According to the prognosis it is placed between categories I and V. The widths of tectonic faults are 25 to 80 m, and the distances between them are about 200 m (regions of compact rock).

The paper considers the problem of tunnel's deformations and stresses due to the movements in tectonic zones. The main questions that arise due to the movements in tectonic zone are:

- deformation of the tunnel's structure,
- distribution of normal and shear stresses in the tunnel's cross-section,
- bending moments and shear forces in the tunnel's cross-section,
- influence of the movement in tectonic fault, the tectonic fault's width, and the ratio of stiffness in compact rock and tectonic fault.

To find the stress-strain response due to the movement in tectonic zone, the mathematical model was made in the form of differential equations. The general solution of the problem is given which is not in the closed form. Introducing assumed shape of the deflection line that is in accordance with the resistance line of the rock in tectonic zone, we obtain the simple solution in a closed form.

The solutions of the equations are functions of deflection, slope, bending moment, shear force, and resistance intensity. They are functions of movement in tectonic zone, width of tectonic zone and compact rock, and stiffness of tectonic zone and compact rock.

Keywords: Tectonic Fault; Fault Movement; Tunnel.

INSTRUMENTATION AND MONITORING TECHNOLOGY FOR UNDERGROUND CONSTRUCTION IN CHINA – REVIEW AND FORECAST

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This paper reviews the history and current status, and forecasts the future direction of instrumentation and monitoring technology for tunneling and underground excavation in China. Underground work is constructed in the geologic media such as rock and soil, and underground structures are integrated with rock/soil materials. Because the geological conditions are substantially reply on instrumentation & monitoring works. If monitoring methods are not suitable and monitoring work is insufficient, engineering accidents will frequently happen, and the construction progress can be dramatically delayed, resulting in very high unexpected cost. In this papers, the recently developed technologies in automatic data collection, processing methods and information management system are discussed. Both successful engineering cases and unsuccessful cases with accidental collapses due to the lack of instrumentation and monitoring are given. In recent years, in China there are a great number of underground projects are on going, but the construction safety and quality are of severe problems and face a big challenge. A package of solution for solving the problems is proposed.

Keywords: Instrumentation and monitoring, Underground engineering, Tunneling, Safety, Risk management.

COMPARISON OF IDENTIFICATION AND QUANTIFICATION OF SQUEEZING CONDITION BY DIFFERENT APPROACHES

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The magnitude of tunnel convergence, the rate of deformation and the extent of the yielding zone around the tunnel depend on the geological and geotechnical conditions, the in-situ state of stress relative to rock mass strength, the ground water flow and pore pressure, and the rock mass properties. Squeezing is therefore synonymous with yielding and time-dependence. This research deals with not only an overview of the methods used for identification and quantification of squeezing along with the empirical and semi-empirical approaches presently available in order to anticipate the potential of squeezing tunnel problem, but also Nosoud tunnel as a practical case study has been applied to evaluate and comparison of the potential of squeezing in different methods. The results of analyses were shown that about 3 Km of second part of the longitudinal axis tunnel has high squeezing potential. These parts are located in shale, marlstone, marly limestone and marly dolomite that are affected in tectonic activities. These zones were often been in high stress region and categorized in GPN class in support point of view. The details of the studies performed are presented in this paper.

Keywords: Empirical approaches, semi-empirical approaches, squeezing, Nosoud tunnel.

INSTRUMENTATION AT HEAD RACE TUNNEL UNDER ADVERSE GEOLOGICAL CONDITIONS

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Tala Hydroelectric Project is a run-of-river scheme with a planned capacity of 1020MW on river Wangchu in South-West Bhutan in eastern Himalayas. Construction of 23km head race tunnel (HRT) was part of the many underground constructions being carried out at the project. HRT is planned to carry 142.5 cumecs design discharge.

Most adverse geology was encountered for a length of 337m where major shear zone was encountered. The excavation was completed with drainage, reinforcement, excavation, support solution (DRESS) technique of excavation and supporting by deploying a forepole machine.

Instrumentation at this stretch of HRT was carried out with a view to understand the rock mass behaviour and magnitude of displacements and loads acting on the support system. The load on the support system was measured using load cells and convergence of the ribs was measured using reflective targets and total station. Fifty load cells were installed in 25 sections and reflective targets were also installed in the same sections. Maximum load on the rib was 178.09 tons and a very high convergence of 463.8mm was measured in tunneling in very weak rock mass conditions.

Instrumentation data was regularly analysed to asses the health of support system and additional strengthening measures were adopted as and when required. MAI anchors were installed to strengthen the rock mass as well as to drain out the water. After taking remedial measures, instruments were further used to assess efficacy of these measures. Convergence rates at AGO reach during Jan-05 to Sep-05 is given in Table 1. This paper discusses the results of instrumentation and how it was effective in decision making during excavation of HRT in adverse geological condition.

Keywords: Adverse Geological Condition; Convergence; Load.

Table 1. Convergence rates along AGO reach during Jan-05 and Sep-05.

RD	Convergence, mm	Elapsed Time, days	Convergence Rate, mm/day							
			Jan-05	Feb-05	Mar-05	Apr-05	May -05	Jun-05	Jul-05	Sep-05
Kalikhola upstream										
1008	12.66	850	0.04	Nil	0.03	Nil	0.01	Nil	Nil	-
1039.65	23.95	693	0.01	0.003	Nil	0.04	0.001	Nil	0.001	0.00
1083.5	125.82	560	0.05	0.01	Nil	Nil	0.04	Nil	0.05	0.11
1101.20	148.81	461	0.03	0.06	0.03	0.01	0.05	Nil	0.2	-
1116.2	129.59	415	0.09	0.05	0.04	0.05	Nil	0.09	0.18	0.03
1124.15	143.15	427	0.12	0.15	0.03	0.05	0.05	0.05	0.29	-
1130.55	161.06	367	0.18	0.13	0.09	0.13	0.16	0.19	0.59	0.01
1139.05	463.89	399	0.31	0.30	0.23	0.15	0.27	0.42	0.75	0.01
1141	112.10	271	0.32	0.25	0.24	0.24	0.31	0.39	0.76	0.01
1150.15	211.05	356	0.33	0.27	0.22	0.23	0.32	0.31	1.01	0.01
1158.75	166.31	342	0.13	0.03	0.08	0.08	0.12	0.18	0.29	0.002
1167.85	151.55	278	0.08	0.12	Nil	0.03	0.08	0.03	0.08	0.005

MODIFICATION OF THE PROPUSED SYSTEM OF RATING FOR ROCK TUNNELLING MACHINE SELECTION USING THE AHP METHOD

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Tunnel construction projects often include complex series of events and technical systems. Were geomechanical properties of the rock mass that influence the tunnelling machine selection are very complex and associated with large uncertainties. So by using the AHP (Analytical Hierarchy Process) Method, a system of rating for selecting the tunnelling machine was developed. Like other rock rating systems, which are normally modified by achieving more experiences and expertizes, this rating system has been subjected to some modifications, as well. To do this, some tunnelling machine preference data have been extrapolated and three new rating parameters have been taken into account. These parameters are: Tunnel construction in water bearing media, safety of tunnel workers and tunnelling in mixed face conditions. The tunnelling machine performance data for this purpose have been collected from two tunnelling projects in Iran. These data are used as reliable experimental basis to establish a modified rating system for selecting proper tunnelling machine for tunnel construction in rocks.

Keywords: Analytical Hierarchy Process, AHP, Modification, Tunnelling Machine Selection, Iran.

INFLUENCE ON GROUNDWATER LEVEL CHANGE DUE TO WATER SEEPAGE IN CHIKUSHI SHINKANSEN TUNNEL, JAPAN

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The influence of tunnel excavation on the environment is a prime consideration in today's environmentally conscious society. Groundwater is usually a major consideration in most tunneling and underground construction operation. The water inflow in tunnel affects not only the construction schedule, stability and safety, but also the groundwater level. In this paper, the impact on groundwater level due to water seepage in Chikushi Shinkansen tunnel, Japan, is analyzed by coupling GIS (Geographic Information Systems) and Flac3D. Firstly, the area around Chikushi Shinkansen (high speed trains) tunnel was drawn into watersheds using the hydrologic-analysis of ArcGIS, and Tank model, which is a method for hydrologic balance analysis of precipitation, evapotranspiration, water flow and infiltration, is used for computing the rainfall recharge (Fig. 1). All the data, such as the geological data, terrain and rainfall, are processed using ArcGIS. Then, a three-dimensional groundwater flow numerical model was built to predicate the water inflow in tunnel and the groundwater level fallen down around the tunnel. Based on the Flac3D numerical model, considering the rainfall-infiltration recharge and tunnel excavation sequence, a spatio-temporal simulation of the water inflow and its influence on the groundwater level of Chikushi Shinkansen tunnel is presented (Fig. 8 and Fig. 9(b)). The results show that the three dimensional real terrain, the geological condition and the rainfall infiltration will affect the groundwater inflow and the change of groundwater level during and after tunnel excavation.

Keywords: Groundwater level; Water inflow; Geographic Information Systems (GIS); Tank model; Spatio-temporal simulation.

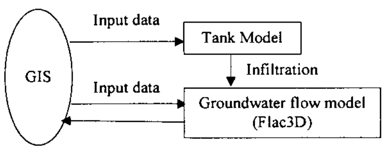

Fig. 1. Flow chart of the study.

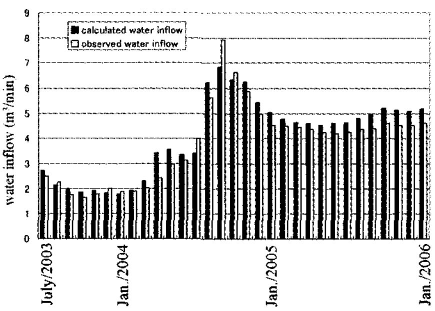

Fig. 8. Water inflow in Kawachi area from July, 2003 to January, 2006.

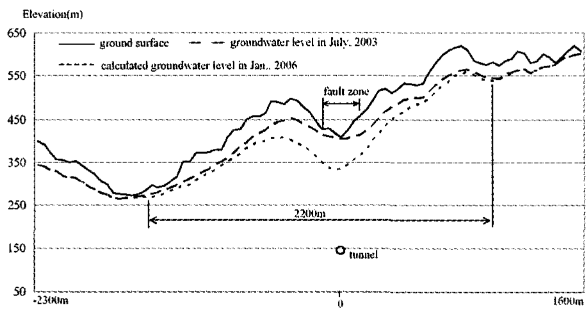

Fig. 9. (b) Groundwater level in BB cross section.

2. ROCK CAVERNS

2.1. General

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A THERMO-MECHANICAL ANALYSIS AROUND LINED LNG STORAGE CAVERN

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The concept of underground LNG storage in lined rock cavern has been developed and tested for several years. According to the pilot scale study from 2003, heat propagation in rock due to cryogenic storage could be well predicted without difficulty. General thermo-mechanical effect on rock near cavern also showed favorable behavior in terms of cavern stability. However, the full scale thermo-mechanical response of rock mass including fractures under cryogenic condition will be quite complex according to fracture characteristics. The movement and the opening of fractures can be expected to significantly affect rock movement and cavern stability.

In this study, a series of thermo-mechancal analysis were performed to understand the behavior of full scale LNG storage cavern, rock mass around the cavern and rock fractures to assure cavern stability during long-term storage of LNG in the cavern. Rigorous models were prepared with UDEC ver. 4.0 and the effect of explicit excavation and lining construction was simulated together with 30 years of storage period. Calculations were carried out with different condition in fracture distribution pattern and orientation etc.

General results show that rock displacements near cavern in most directions except the roof were directed toward not excavation faces but rock mass due to thermal contraction of rock mass itself during long-term cooling. As cooling started, compressive rock stresses fell down rapidly and tensile rock stresses were generated and increased until 5 years of cooling.

According to the results, maximum rock mass temperature fell down to about –50°C for 30 years of cooling, no excessive displacements and stresses were generated.

It is realized from the results that the behavior of rock fractures(opening and shear displacement) due to long-term cooling would be strongly dependent on fracture spacing, distribution pattern and direction. Opening and shear displacement were generally increased until 8~12 years.

The complex action of intersecting fractures can influence either favorable or adverse way, therefore the lay of the cavern should be determined considering orientation of fractures to assure cavern stability.

Keywords: Thermo-mechanical; cavern; rock fracture.

EVALUATION OF THE STABILITY FOR UNDERGROUND TOURIST CAVERN IN AN ABANDONED COAL MINE

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A series of geotechnical surveys and in-situ tests were carried out to evaluate the stability of underground mining gallery in an abandoned coal mine. After closure of the mine, the underground mine drifts have been utilized for a tourist route since 1999. The dimension of the main gallery is 5m width, 3m height and 230m length. The surrounding rock mass of the gallery is consist of black shale, coal and limestone. Also, the main gallery is intersected by two fault zone.

Detailed field investigation including Rock Mass Rating (RMR), Geological Strength Index (GSI) and Q classification were performed to evaluate the stability of the main gallery and to examine the necessity of reinforcement. Based on the results of rock mass classification and numerical analysis, suitable support design was recommended for the main gallery. RMR and Q values of the rock masses were classified in the range of fair to good. According to the support categories proposed by Grimstad & Barton (1993), these classes fall in the reinforcement category of the Type 3 to Type 1. A Type 3 reinforcement category signifies systematic bolting and no support is necessary for the Type I case.

Many discontinuities (beddings, joints, faults, etc) discovered in the mine exhibition gallery took very complicated aspects. In addition, mechanical behavior characteristics of target rock mass were definitely differentiated between discontinuities and fresh rock mass. Therefore the discontinuity analysis considering mechanical characteristics as well as the geometrical distribution characteristics of discontinuity was conducted. For this, the stability of the mine exhibition gallery was analyzed by using UDEC (Universal Distinct Element Code). UDEC is a commercial program based on the discrete element method (DEM), and the whole model is generated by a number of distinct blocks, which can be divided into deformable triangular finite difference zones in UDEC.

The mine exhibition gallery has been currently reinforced with steel ribs and rock bolts. Considering that steel ribs are not simulated in the two-dimensional analysis, it is regarded that the mine exhibition gallery are wholly stable but additional reinforcement for partial unstable blocks is required. Consequently, when unstable factors are detected, the following measures should be taken:

First, the section of rock bolts installed partially should be observed continuously. If unstable factors such as collapse are detected, ceiling reinforcement by wire meshes and systematic rock bolts should be taken to ensure the long-term stability of gallery when needed.

Second, an effluent in some sections may cause rise the humidity in the mine exhibition gallery, corrode steel ribs and rock bolts, or pit prop and timber in the mine exhibition gallery, and so deteriorate the long-term performance of support materials. Outworn materials and facilities in the mine exhibition gallery should be replaced by new things

Keywords: Underground mining gallery, Rock mass classification, Reinforcement, Numerical analysis.

HYDROGEOLOGIC ANALYSIS OF GROUNDWATER DRAINAGE SYSTEM FOR UNDERGROUND LNG STORAGE CAVERN

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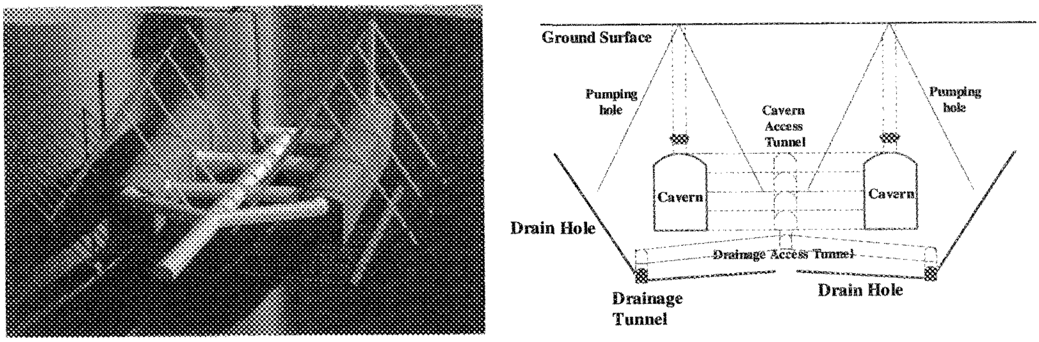
A new concept storing LNG in a hard rock lined cavern have been developed and tested for several years. In this concept, groundwater in rock mass around cavern has to be fully drained until early stage of storage operation to avoid possible adverse effect of water near cavern walls. For this reason, groundwater drainage system, which is mainly drained by gravitational force to inside of drain tunnels and holes underneath the cavern as shown by Fig. 1. And if this system is worried not to be enough or to take much time to drain, the vertical pumping hole is drilled on surface or shaft access tunnel.

In this paper, the hydrogeologic study related to groundwater drainage system of the LNG lined rock cavern that is proposed to be located in Pyeongtaek was performed. At first, groundwater levels observed in several monitoring holes, the recharge into the rock layer where the cavern was embedded and hydrogeologic boundaries such as hill area, river and sea were surveyed so that the distribution of groundwater was mapped around the supposed site. And by Modflow analysis in the viewpoint of aquifer and Seep/W analysis in the viewpoint of flow system, it was possible to evaluate if the drainage system was efficiently operated and if the position and space of drain holes was arranged adequately. Also, drainage rate into the drain holes and seepage rate into the caverns and tunnels was estimated though the evaluation of groundwater mass balance and the period of drainage could be calculated on design stage.

Keywords: Underground LNG storage, Drainage system, Drain holes, Groundwater modeling.



(a) 3D view

(b) Section view

Fig. 1. Schematic figure of the unlined underground LNG storage cavern.

DESIGN OF ROCK CAVERNS IN HIGH IN-SITU STRESS ROCK MASS

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Six caverns are designed in the world's longest double tube road tunnel, the Qinling Zhongnanshan tunnel, for special lighting and driving safety purposes. The tunnel is excavated through the Qinling Mountain Range of Shaanxi province, China, where the maximum rock overburden is about 1800 m. Having realized the importance of the in-situ rock stress, a field measurement program by overcoring was carried out in two boreholes close to the cavern sites. The overburden for the boreholes is 400 and 1600 m, respectively. The measurement at the low overburden hole was successful, whilst great difficulties were encountered in the high overburden hole. Severe core disking was observed. Finally, the 3D overcoring was replaced by doorstopper (2D overcoring). The measurement result indicates the highest major principal stress may be as high as 45 MPa. The rock type is mainly granitic gneiss of good quality.

Based on the requirements of lighting and driving safety the caverns are designed to have a length of 150 m and a maximum span of 22 m which is gradually reduced to the standard tunnel width of 12.9 m. However, restricted by the distance between the two existing tunnels the minimum width of the pillar between the caverns is only 8 m. In summary the main features of the caverns are:

- High in-situ rock stress
- Generally good rock
- Small pillar in comparison to the cavern size

The rock support design is based on empirical approach from Q rock mass classification and the design is then verified by the numerical analysis. FLAC3D is used in general analysis of cavern stability with focus on the 3D effects, i.e., the variation of stress and deformation in the direction of tunnel axis; while the 2D analysis with Phase2 is used in detailed study of entire construction sequence and functioning of each support element. Two different designs are proposed for low and high overburden conditions, respectively. The support means include rock bolts and steel fibre reinforced shotcrete. Special procedures for excavation, support installation and monitoring are also proposed.

Keywords: Road tunnel; rock support; in-situ stress measurement; numerical analysis; rock mass classification.

DEVELOPMENT OF THE GROUNDWATER RESOURCES IN BEDROCK USING ROCK CAVERNS

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Water shortage frequently occurs in the regions where there is no abundant river water or shallow groundwater. An efficient development of water resources in such regions is an important problem for sustainable activities of economy and social life. The authors have been interested in the groundwater in bedrock as new water resources for such regions, and we have shown the possibility to use the groundwater in bedrock through the studies on the tunnel water discharge in Japan. Based on these studies, we proposed "the groundwater-intake system using rock caverns" and the required technology to extract the groundwater from rock mass. Recently, "the simplified groundwater-intake system" was proposed as a developed type of the former system. The former system aims to develop the groundwater which amounts from thousands to ten thousand tons per day. On the other hand, the new system aims for obtaining the groundwater less than thousands tons per day. The two systems are properly selected according to the quantity of the required water, geological and geographical conditions of the site.

The water-intake principle of the systems is based on the mechanism which the groundwater flows into the tunnel according to the difference of the water head between the shaft and surrounding bedrocks. The performance of the systems is calculated by the 3-dimensional FEM analysis. The site selection and structural design of the systems are usually decided from the results of many investigations. In this paper, we show the computer-aided geographical features analysis for the presumption of the groundwater flow direction, and also show the quasi 3-D FEM groundwater analysis corresponding to the rain fall which supports the design of the systems.

In addition, a concept and a structural design of "a water-intake and reservoir system" which can store groundwater and river water in underground rock caverns are also introduced as another development method.

Keywords: Groundwater resources; bedrock; rock cavern; water-intake system.

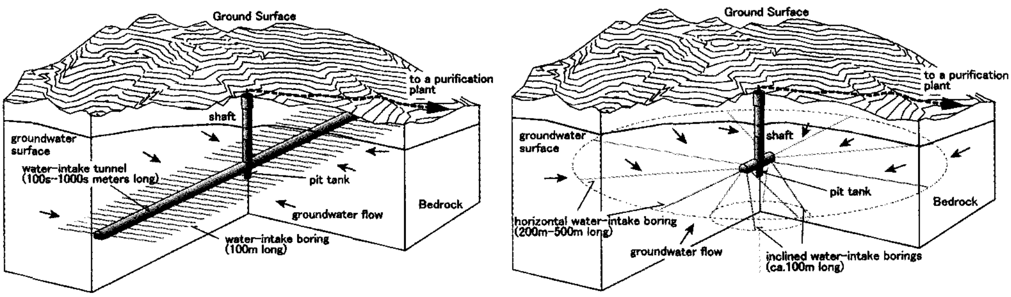

Fig. 1. Structural design of the groundwater-intake system in rock mass; left: the standard type, right: the simplified type.

CAVERN DESIGN CONSIDERATION WITH MODERN DRILLING EQUIPMENT

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Traditionally large caverns for underground storage or other purposes have been given an excavation sequence of a top heading of moderate height 7-8 m, a first horizontal bench 5-6 m and two vertical benches to achieve a final height of 30 m or more. Modern drilling equipment makes it possible to reduce the number of excavation stages from 4 to 3 by starting with a top heading having 10m height. This gives the opportunity to carry out the benching in two stages only and both by vertical benching. This a means an unelectable saving of the construction cost. The effects of this height increasing on the total stability of cavern are discussed for different geological and geometrical condition by analytical solutions and continuum numerical modeling. These different geological and geometrical conditions are quality of rock mass, depth of excavation, *in situ* stress and effects of other openings. Performed analyses indicate that in some cases, like excavating two or more deep caverns by applying a new excavation sequence will decrease the total displacement and the plastic zone. In case of excavating a cavern in hard brittle rock at shallow depth, using a new excavation sequence hasn't any effect on the total displacement of cavern. By applying the new excavation sequence on a deep cavern in hard rock as well as two or more cavern at shallow depth in poor rock will cause an increase of the plastic zone around the opening

Keywords: Drilling, Stability Analysis, Cavern, Design, Excavation Sequence.

APPLICATION OF COMPOSITE ELEMENT METHOD TO PUBUGOU UNDERGROUND ENGINEERING

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Discontinuity is the general term of faults, joints, beddings, etc. in the rock masses. The simulation of discontinuities is a crucial problem in the study of the deformation and stability of rock foundation, slope and underground cavern. Composite element method (CEM) is intended to solve the problem recently. The main advantages of the composite element method is that it can be incorporated into the conventional finite element analysis procedure, and the mesh generation of the large scale rock structures with considerable number of discontinuities requiring explicit treatment in the calculation will not be restricted by the number, position and orientation of the discontinuities.

As a new numerical method, lots of test should be carried out on the composite element method for discontinuity. Some trials, such as one elements containing one or more joints anti-press test, a dam with one or more faults in its base anti-press test and so on, have been successful. Underground engineering is more complex than the dam base, because the discontinuities in rock masses intersect with the caves. PuBugou large-scale underground cavern engineering provides a good chance to test the method farther.

The three dimensional model contains all six generator units in the engineering. Mesh simulates main cavern, main transformer chamber, bus bar gallery, tail gates chamber, diversion tunnel, tailrace tunnel, which contains 41580 nodes and 39690 elements, including 8517 composite elements. Excavation is divided into eight steps and reinforcement is applied after every excavation step. The operation between seven faults and mesh is carried out in pre-process, and its results are inputted into the computation program of CEM. The former practice shows that it is very difficult to simulate explicitly the tunnels and three faults intersected with them by FEM, but CEM is able to deal with this structure involving seven such faults in the dissertation. The results list the displacements, stresses and plastic regions of every generator unit section, stresses and safeties of every fault. The computation results accord with impersonality law, which shows the effect of the method in large-scale engineering.

Keywords: Underground; discontinuity; composite element method.

GEOTECHNICAL, ENVIRONMENTAL AND STRUCTURAL ASPECTS OF UNDERGROUND STORAGE OF HAZARDOUS SUBSTANCES

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Despite growing significance of use of renewable resources the increasing Industrial and agricultural production, energy consumption and traffic still requires continuous provision of hydrocarbons and other chemical substances. For strategic and economical reasons these materials have to be available even in times of supply shortages. Therefore large quantities of such substances have to be kept in store. Since most of such substances are inflamable, toxic and hazardous towards ground water apropriate storage facilities have to be designed and operated in compliance with high safety standards.

Decreasing availability of storage area above ground and security requirements often warrant construction and operation of such facilities in the underground.

Based on experience from design and construction various concepts of underground storage of liquid hydrocarbons are presented. Evaluation of these concepts with regard to safety and environmental aspects as well as product quality assurance shows that storage in unlined underground openings is no longer in line with recent authoritative regulations and legal requirements of many countries.

Considering the requirement for a multi-barrier confinement system a solution comprising a continuously lined underground opening with a double-walled liner was found to be best suited to fulfill the environmental and safety requirements.

Keywords: Underground storage; hazardous substance; multi-barrier confinement.

INFLUENCE OF CONCEALED KARST CAVERNS ON TUNNEL STABILITY

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Karst disaster is often occurs in the tunnel construction in mountain district. According to a survey on the tunnel Karst disaster in southwest of china, tunnel destabilization resulting from concealed Karst caverns fall into two types and the corresponding mechanics model is set up. When jumbo-sized concealed Karst caverns are concerned, the destabilization mechanism of rock between Karst caverns and tunnels is analyzed and simplified into collapse destabilization of beams and plates. And the theoretical formula for the minimal safe thickness of the tunnel roof and bedplate is deduced according to the elastic theory.

When most middle and small sized caverns are concerned, based on the formation of Karst caverns and the theological deformation characteristics of surrounding rock, the mechanics model of Karst caverns can be divided into open-field mechanics model and excavation mechanics model. The Karst tunnel problems caused by the middle and small sized caverns are mainly studied numerically with the consideration of the influence of different positions and scopes of the concealed Karst caverns on stress field, displacement field and the internal force of support structure for tunnel with the popular shape type. The influence of surrounding rock conditions and the initial stresses to the displacement, stress and internal force of supporting structure of the tunnels are tested numerically. On the basis of numerical experimental analysis, the present paper analyzes the internal mechanism of tunnel displacement variation rule resulting from different scopes and positions caverns and makes the partition graph of cavern influencing range.

Finally an in-situ measurement for a tunnel construction in the Karst region is conducted in Wu-Dou project. The variation of tunnel displacement with and without caverns influential conditions are observed and compared with the numerical results. And the rationality and creditability of numerical conclusions are proven.

Keywords: Tunnel; Karst caverns; Stability of tunnel; Numerical experiment.

FUNDAMENTAL STUDY ON LONG-TERM STABILITY OF THE UNDERGROUND CAVERN

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In this study, we focused on the long-term rock strength. The time dependency of rock mass strength was evaluated with BEM-CM based on measurements that had been carried out at an existing large underground cavern.

The results of analysis indicate that increments of rock displacements after the end of excavation can be explained with decrement of the peak strength of rock as in Fig. 1 and that the decrement of the peak strength may have a liner relation with time on log-log axis as in Fig. 2 in accordance with the relation that is evaluated by homogenization method for crack propagation in rock and that under a circumstance where the residual strength of rock is maintained, the mechanical behavior of cavern will converge in long time as in Fig. 3.

Keywords: Long-term stability, Time dependency of rock strength, Large underground cavern, BEM-CM.

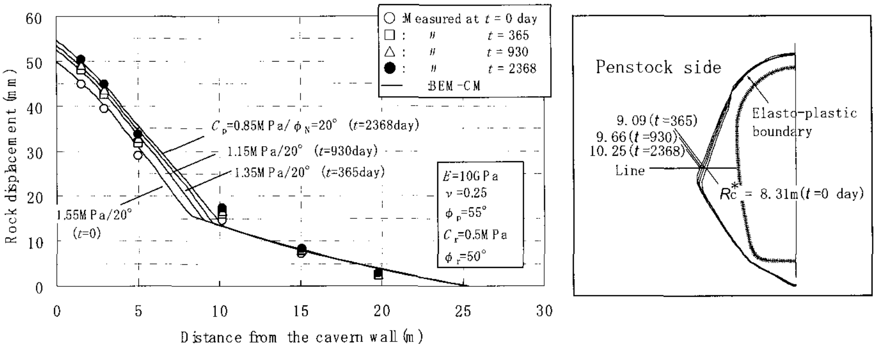


Fig. 1. Comparison of displacements between measurements and calculations.

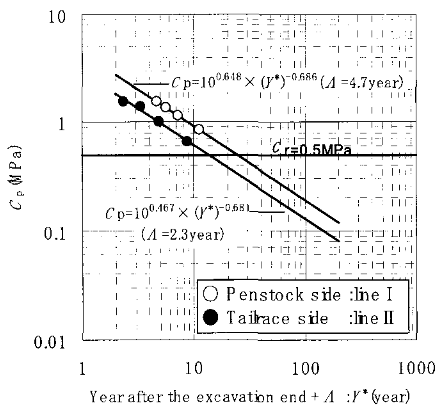
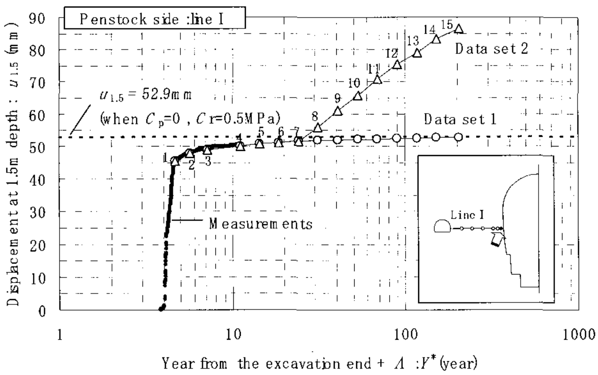


Fig. 2. Time dependency of C_p.



Data set 1; C_r (=0.5MPa) is constant through evaluation period.

Data set 2; C_r decreases similarly with C_p after C_p has decreased up to same value of initial C_r.

Fig. 3. Results of long-term evaluation.

DESIGN METHODOLOGY FOR HYDROCARBON CAVERNS, INFLUENCE OF IN-SITU STRESSES ON LARGE SECTIONS

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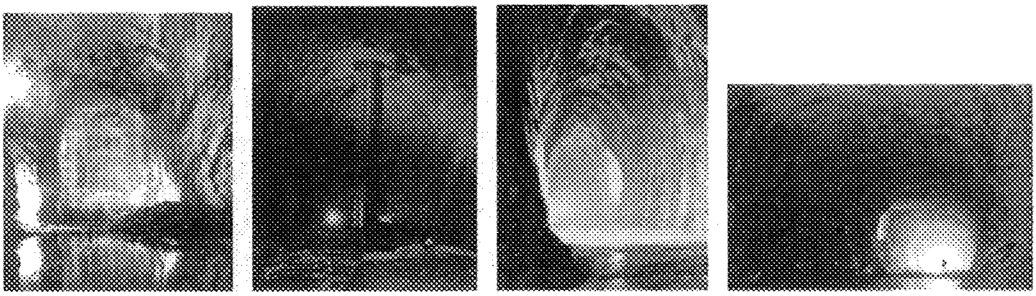
During the early days, mined rock caverns were based on intrinsic imperviousness of rock masses and the designers used mainly their mining experience. Then, the idea of pinched roofs facilitating roof joint closure appeared, ovoid and elliptical shapes were designed based on consideration of induced stress distribution and limitation of water flows. More recently, while in-situ stress measurement techniques improved, new projects were designed and constructed in rock masses characterized by significant anisotropic conditions. The authors present a panorama of forty years of underground storage section design and focus on a case-study still under construction, Visakhapatnam in Andhra Pradesh, India, for which the section had to be amended in order to better cope with the measured stresses.



Photos: Donges, Ulsan, Inchon and Sydney

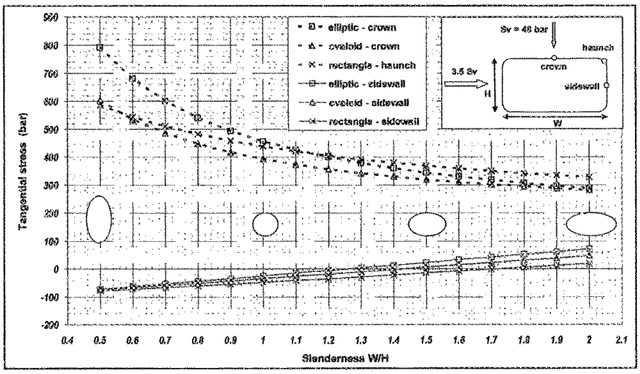


Fig. 7. Shape effect on the tangential stress for K=3.5 at storage depth: variation of the minimum and maximum stress as a function of slenderness for elliptic, ovaloid and rectangular cavern.

The new era of gas storage will also be challenging for design and construction engineers with growing consideration of coupled mechanisms, thermal loads and non-linear changes in rock parameters. It has been shown that new tools such as 3D, discontinuous and fully coupled codes are already needed and used by engineers. An interactive design process of storage under construction is presented showing the interest to obtain key parameters before critical construction path keeping some flexibility to accommodate new information.

Keywords: Design; Cavern; Storage; Section; Stresses; Case-study.

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2.2. Theoretical and Numerical Analyses

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FINITE ELEMENT ANALYSIS OF UNDERGROUND NUCLEAR REPOSITORIES WITH TEMPERATURE DEPENDENT ROCK PROPERTIES

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The design of a nuclear waste repository involves modeling a system of physical mechanisms operating over a long period of time. In case of a nuclear repository structure, temperature of rock increases with time and ultimately it reaches steady state condition. With increasing temperature, rock properties change considerably. With increasing temperature, modulus of elasticity (E) of rock first increases upto about 100^0C and then decreases by a large amount. All other rock properties like density (ρ), cohesion (c), angle of internal friction (ϕ), thermal conductivity (k) and Poisson's ratio (ν) decrease with increasing temperature of rock while coefficient of thermal expansion (α) of rock increases with increasing temperature. Therefore, considering temperature dependent rock properties in analysis is of major importance.

In this paper the elastoplastic thermomechanical analysis of a KBS3 type nuclear repository structure located at great depth in tuff rock has been carried out using temperature dependent rock properties for steady state temperature distribution. Results have been compared with those using temperature independent rock properties to study the effect of temperature dependence of rock properties on stresses, deformations and plastic strains. Mohr- Coulomb yield criterion has been used to simulate rock behaviour. Finite element code ABAQUS has been used for analysis. An overall increase of stress, displacement and equivalent plastic strain has been observed in temperature dependent case as compared to temperature independent case. The temperature independent case with a canister temperature of 200^0C has been considered as reference case.

Comparing the results of temperature dependent case with the reference case, a maximum stress increase of 20% has been observed. A maximum stress reduction of 6% took place in the plastic zone. The equivalent plastic strain was 0.2205% in case of temperature independent case which increased by almost 82% to 0.4089 % in temperature dependent case.

Keywords: Nuclear repository, temperature dependent; elastoplasticity; thermomechanical.

BEHAVIOUR STUDY OF LARGE-SCALE UNDERGROUND OPENING IN DISCONTINUOUS ROCK MASSES BY USING DISTINCT ELEMENT METHOD

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The deformational behaviour and stability of large-scale underground openings are ruled by the mechanical behaviour of discontinuities such as joints and faults in the discontinuous rock masses. It is recognized that the majority deformation and degradation of the host rock masses around underground openings often originates from the shear failure and dilation along the discontinuity surfaces. Therefore, in the analysis of the deformational behaviour of an underground structure by using numerical simulation, how to model the discontinuities becomes a key problem. Continuum analysis is a main method in the current design and construction practices, which assumes the discontinuities to be weak joints and decreases the mechanical properties of rock mass to build an equivalent continuum. However, continuum analysis has difficulty in evaluating the local and detailed behviour of underground openings, some of which are necessary information for design, construction and management. In this study, Distinct Element Method (DEM) was used for simulating the deformational behaviour of the underground opening of a pumped storage power plant based on in-situ geological data. Its validity has been evaluated by comparing the analytical results to the in-situ data. The differences of deformational behaviour due to different gradients of initial stresses and different shapes of openings (i.e. warhead shape, egg shape) were evaluated. The results show that egg shape is advantage for restraining the deformation of openings in a large width of the gradients of initial stresses. When the gradients of main discontinuities are close to the principal stress, care should be taken on the blocks which trend to "flow" into the opening.

Keywords: Underground opening; Distinct Element Method; Shape of cross section; Inclination of initial stress.

HYDRO-THERMAL COUPLED ANALYSIS OF ICE RING FORMATION IN UNDERGROUND PILOT LNG CAVERN

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Design and construction of underground LNG storage in lined rock cavern requires various theories and techniques based on rock mechanics, heat transfer and hydrogeology. Underground pilot LNG cavern located in Daejeon had been constructed and operated for the verification of brand new technologies. In this paper, ice ring formation is investigated through hydro-thermal coupled analysis using *FLAC*. Ice ring plays a secondary barrier in case of leakage of cryogenic liquid and a primary barrier against groundwater intrusion into LNG cavern. Therefore, the thickness and location of ice ring are very important for the safe operation of LNG storage and integrity of primary barrier composed of concrete, PU foam and membrane. *FLAC* provides easy method for coupled hydro-thermal analysis but does not provide phase change of water and corresponding permeability change. Those features are implemented using *FISH* language for accurate simulation of ice ring formation. Through the numerical analyses, the position and thickness of ice ring are estimated. Due to the difficulties of in-situ identification of ice ring, only the temperature and groundwater level are compared with monitored values. The numerical results show good agreement with measured values when temperature dependent properties and phase change are taken into account.

From the coupled analyses, we can make following conclusions:

1) The temperature dependency of the input properties must be considered for appropriate modeling of the cryogenic environment. Particularly when the temperature range of simulation is wide, the variation of properties has to be taken into account. 2) The capability of hydro-thermal modeling of ice ring formation has been verified by a comparison of numerical results with in-situ measurement data. Therefore, a similar approach can be extended to the simulation of ice ring formation in a full-scale LNG storage cavern. 3) The location of the ice ring is an important design factor in this new concept of LNG storage and has been found to be dependent upon the 0℃ isotherm spread as well as the time of the drainage stop, which can be quantitatively determined by the numerical method introduced in this paper.

Keywords: Hydro-thermal coupling, pilot LNG cavern, ice ring, phase change.

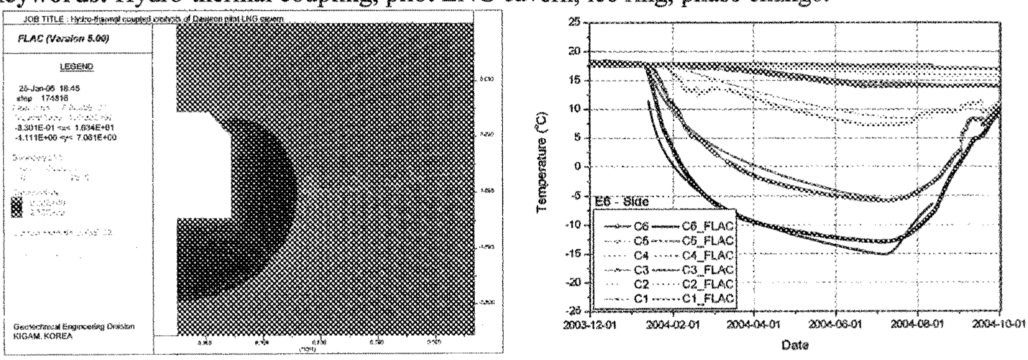

Fig. 1. Hydro-thermal simulation of Ice ring formation and results for Daejeon pilot LNG cavern.

HEAT TRANSFER AND BOIL-OFF GAS ANALYSIS AROUND UNDERGROUND LNG STORAGE CAVERN

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One of the important issues in cryogenic storage is to reduce the amount of boil-off gas due to heat loss. Though thick Polyurethane foam is used for containment insulation, certain percentage of boil-off gases is unavoidable because of extreme temperature changes between walls (Figure 1). Underground LNG storage in lined rock cavern has many advantages, especially in terms of cryogenic heat loss and storing efficiency, compared with existing aboveground or inground storages. Cooling front propagates progressively into rock mass, therefore refrigerated rock decreases the amount of boil-off gas because progressive decrease of rock temperature reduces heat loss. Design should be based on accurate heat transfer analysis, therefore it is very important to predict the heat propagation from storage into rock mass properly in terms of robust design.

In this paper, the results of heat transfer analysis for both pilot-scale and full scale storage caverns are presented. Monitored temperature profiles from pilot-plan are compared with calculated data, and they show good agreement with proper rock properties. For the prototype design of full-scale storage, a series of heat transfer analysis were performed to determine insulation thickness, pillar width between caverns and boil-off gas ratio in case of typical storage conditions. Overall rock temperature did not fall down to -50°C after 30 years of storage. Though zero-degree isotherm has propagated about 20-30m from cavern walls, there was no significant effect on ground surface in terms of environmental aspects. Boil-off gas ratio (BOR) was calculated based on heat flux output along insulation panels. Calculated boil-off gas ratio for underground storage cavern was quite low compared with aboveground storage. When the effect of groundwater was considered with coupled model, BOR could be much lower compared with heat transfer only case. It can be concluded that the heat transfer in rock mass around LNG storage can be well predicted with current numerical approaches, therefore efficient design of the storage cavern is possible with reasonable prediction of heat propagation in rocks.

Keywords: Heat transfer; gas; cavern.

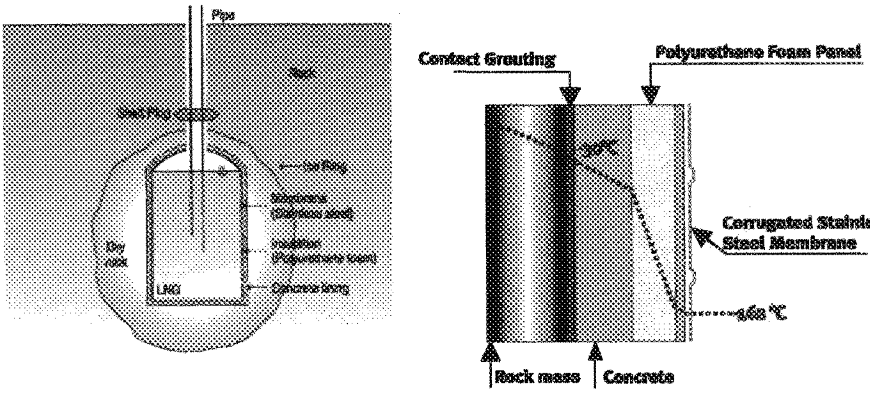

Fig. 1. Concept and insulation structure of underground LNG storage.

NUMERICAL SIMULATION AND DISPLACEMENT FIELD MEASUREMENT OF POWERHOUSE CAVERN EXCAVATION

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A nonlinear, three-dimensional finite element model was applied to simulate the conventional procedure of underground workhouse excavation. The Drucker-Prager elastic-perfectly plastic material model was used for the simulation of rock mass, faults and the supporting structures. The material non-linearity was dealt with using an incremental technique. To monitor the stability of the powerhouse cavern, displacements of the surrounding rock were measured during excavation. The field measurement results show that the surrounding rock of cavern is stable while the rock bolt and shotcrete were designed as the supporting structures.

The Baishan hydropower station was constructed on the second Songhua River in the Jilin Province, northeast China, between 1975 and 1983. The concrete gravity arch dam is 149.5m high, and 676.5 m long. In-situ monitoring during the excavation and at longer intervals after the underground cavern 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 rock mass to tunnelling and the effectiveness of the structural support. The main objectives of in-situ monitoring include: (1)To control the deformations of the cavern. 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 measure the development of stresses in the structural members, indicating sufficient strength or the possibility of strength failure. (4) To indicate progressive deformations, which require immediate action for surrounding rock mass and support strengthening.

In order to investigate stability of surrounding rock of underground powerhouse cavern, finite element model and sequential excavation procedures were proposed. The behaviour of an elastic perfectly-plastic material is governed by a yield criterion and an associated flow rule. The Drucker-Plager criterion is a simple modification of the Von Misses criterion involving the influence of the hydrostatic pressure on failure. The excavation process is simulated step by step with finite element method. The numerical solution is presented for evaluating the stability of surrounding rock around large-scale underground cavern. The field measurement of displacements indicates that cavern wall deformations can be used in assessing the condition of the rock mass around the cavern and the evolution of the loads on the temporary support. Safety during cavern excavation relies heavily on rock mass deformation monitoring, as early warning of incipient failures is often based on the rapid collection and evaluation of surrounding rock deformation measurements. As such failures usually evolve in a relatively short period automation in the collection and processing of monitoring data is thus necessary.

Keywords: Numerical simulation; Displacement field measurement; Underground powerhouse; Excavation process; Finite element method.

ONE SYSTEM ANALYSIS METHOD IN UNDERGROUND CHAMBERS EXCAVATION AND APPLICATION

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As more and more large-scale underground hydropower station have gone into design or construction stage in succession in recent years in China, the stability study of large underground opening group is placed on the new order. Based on the underground structure scheme of LONGTAN hydro-electrical project, this present paper outlined a prediction system for underground caverns deformation, especially for caverns in soft rock masses by the method of systems analysis. In analysing, 3D-FLAC program was employed and the specific factors, such as modulus of deformation, layout depth of underground cavern, height of main factory premises, coefficient of lateral compressive stress etc, were mainly concerned.

With a great number of elastic-plastic numerical simulation, the formulations with correlated curves and tables in terms of dimensionlessness were obtained by mathematical statistics method. The selected dimensionless parameters are $X = 1000\lambda\gamma H / E$ and $Y = 1000u / h$. When E is smaller relatively, the pure space B between power house and transformer house have big influence on the displacement of surrounding rock. The forecast displacement is amended by space B. Therefore, the forecast formulations could describe the effects of rock quality, overburden depth, horizontal initial stress, power house height, pacing interval of underground houses on the surrounding rock's stability and were used to predict the displacement of key points of the high side wall of underground caverns. Then the forecast formulations were used to predict the side wall displacements of five underground complex opening works of large scale in China. It was shown that there was a good agreement between theoretic results and monitoring result in situ, which verified the effectiveness of the proposed method. The method is guiding or referential significance to actual projects for design, construction and monitoring.

Keyword: Stability of underground cavern; Numerical simulation; Regression; Displacement forecast.

NUMERICAL MODELLING FOR FEASIBILITY ANALYSIS OF POWERHOUSE CHAMBERS IN WEAK FORMATION

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North-eastern states of India face a severe power shortage. Hydro-electric power is thought as a viable and eco-friendly option to overcome this impending crisis. Subansiri Lower Hydro-electric Project of National Hydro-Electric Power Corporation (NHPC), situated in the north-eastern states of Assam and Arunachal Pradesh, is one of such attempts to partially cater the rising power demand in these states.

It is therefore proposed to construct a water retaining dam across the Subansiri river, a tributary of the Brahmaputra river, and to excavate large power house caverns to produce 1000MW electricity. However, the feasibility of the project very much depends on the stability of these power house caverns, which are to be located in a weak and massive sandstone formation.

The stability of the proposed underground chambers for the power house, transformer hall and the surge chamber is analysed with the help of three-dimensional finite difference modelling software FLAC3D.

The safety factor contours depict an infeasible scenario even with dry rock properties and therefore, no further model runs are performed.

Following the infeasibility of the above underground chambers, the option of open chambers is sought after. A scheme for two open-pits at the downhill slope is prepared by NHPC. The feasibility of the above scheme largely depends on the stability of the open surge chamber, which will essentially be a vertical pit.

A three-dimensional numerical model is constructed and elasto-plastic analysis is performed in several stages to simulate the excavation. More realistic strength parameters, those pertinent to the saturated rock mass are used for this analysis. The open excavation scheme shows a much better stability as compared to the earlier underground chamber concept.

Though it is a great challenge to construct in such a weak formation, in the light of the above three-dimensional modelling studies, the open-chamber option is now considered as a feasible one from the stability point of view. The negative aspects are the large volume of earth to be removed for constructing the open chambers and requirement of heavy supporting, apart from the environmental factors.

This paper gives a brief description of the geological investigations, details of numerical modelling studies for the underground and the open chambers and analysis of stability using local safety factors and mohr-coulomb plasticity states.

Keywords: Numerical modelling; FLAC3D; large caverns; stability analysis; weak rock; power house cavern.

3D ANALYSIS OF POWER CAVERN IN ROCK MASS USING JOINT FACTOR AND NONLINEAR HYPERBOLIC MODEL

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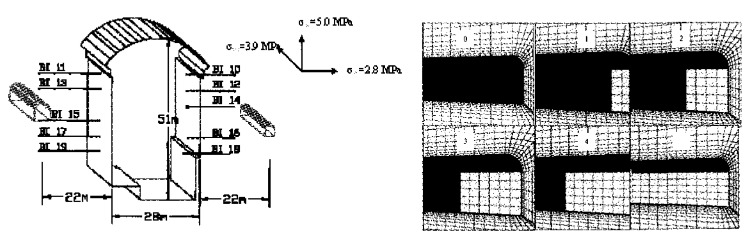
This paper focuses on 3-dimensional (3D) analysis of Shiobara power station cavern in jointed rock mass using *FLAC3D*. The rock surrounding the cavern is highly jointed with the frequency and inclination of joints varying around the cavern. The analysis is carried out using joint factor for describing the joints in the rock mass and their non-linearities together with hyperbolic model describing the constitutive behavior of the rock mass, referred as "Practical equivalent continuum model". These have been programmed in a new *FISH* function in *FLAC3D*. Sequential excavation is simulated in the analysis by representing the excavated rock mass using null model. Fig.1(a) shows the outline of the Shiobara power cavern and the different measurement locations. Fig.1(b) shows the excavation scheme in which the cavern has been excavated. The deformations reported from the field studies were compared with the predicted observations from the numerical modeling. The values predicted from the present analysis are consistent and compare well with the measured deformations (see Table 1). Further, the results obtained from the practical equivalent continuum model in 3D have also been compared with six different methods of analyses for jointed rock mass such as micromechanics based continuum model, equivalent rock model, strain softening analyses considering joint failure, multiple yield model, crack tensor model and damage tensor model as reported by Horii et al.(1999). It is found that the results from the present work are comparable with the results from these different models for the case of Shiobara power cavern (see Table 1).

Keywords: Equivalent continuum; joint factor; hyperbolic model; *FLAC3D*.



(a) (b)

Fig. 1. a) Outline of the cavern and locations of MPBX for the cavern at Shiobara power station.
b) The various stages of excavations using null model available in *FLAC3D*.

Table 1. Displacement values (in mm) along measurement lines.

Distance from the cavern wall (m)	*Along BI 10*		*Along BI 11*	
	Measured value Horii et al.(1999)	Estimated using ECM-Present analysis	Measured value Horii et al.(1999)	Estimated using ECM-Present analysis
0	11.4	21.1	33.9	25.4
1.5	11.1	16.3	29.2	20.6
3	6.5	13.1	19.1	16.0
5	3.8	11.0	6.3	12.2
10	1.2	5.6	1.3	7.7
15	0.2	0	1.7	4.8
20	--	--	0.26	0

COMPARISON OF 2D&3D ANALYSES RESULTS OF MASJED SOLIMAN POWERHOUSE CAVERNS BY ANSYS SOFTWARE

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Hydroelectric power plant of Masjed soliman is constructed in two separate phases, first phase and extension phase. The powerhouse cavern has 256m length, 30m width and 18 to 50 meter height, and the transformer cavern has 220m length, 13.6m width and 21m height.

This Hydroelectric power plant is located in sedimentary rocks of Bakhtiary geological formation. Overburden of these caverns varies from 270 up to 320 meters. This paper presents a Comparison between 2D&3D analyses results of these caverns, done by Ansys software. The analysis results show that 2D & 3D results could be different, i.e. displacement measured from 2D analysis is more than that of 3D analysis. Therefore, with emphasis on the results and displacement records, the new support system is designed and prepared.

At the end with trial and error method, a relation between 2D & 3D is suggested.

Keywords: Back analyses; 2D and 3D analyses; powerhouse cavern; Masjed Soliman power plant.

NUMERICAL SIMULATION OF GAS STORAGE CAVERNS IN QOM REGION

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The rock mechanical design of gas storage cavern in salt requires the analysis of the stability and the usability of the cavern over the planned operating time period. The design includes the build up of a rock mass model and a numerical model taking into account the geological situation, load condition, geometrical condition, and material parameters. In this paper multiple caverns in salt formation with geological and geomechanical property in Qom (central part of Iran) was investigated a using creep model. Minimum safe center to center distances (CTCD) of multiple horizontal caverns were also studied. CTCD of caverns interact at less than two times of cavern diameter. As result with increasing the CTCD to 2.5 times cavern diameters, diminish most interaction.

Keywords: Salt; Cavern; Creep; Qom; Numerical Simulation; Stability.

STABILITY ANALYSIS OF A LARGE UNDERGROUND POWERHOUSE CAVERN BY 3D DISCRETE ELEMENT METHOD

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The logic has been applied to analyze the stability of China Yantan underground powerhouse in this paper. The appropriate calculation method should be selected based on the dominant mechanism of rock failure. The failure of rock mass in this project is mainly controlled by the discontinuity distribution because of the low level in situ stress, so the discontinuous numerical simulation method was adopted to solve this type of problems. With the help of the Discrete Element Method software 3DEC, the random discontinuities were simulated and the instable blocks were exposed. The method and result can give reference to other engineering projects of its category.

Keywords: Underground cavern; discrete element method; stability of rock mass.

Table 1. Statistical results of joint elements.

Joint set	Dip direction (degree)	Dip angle (degree)	Average spacing (m)
J1	10-40	20-25	5.8
J2	70-80	24-25	7.5
J3	150-170	68-75	11.1
J4	230-270	75-80	14.3
J5	290-310	72-74	16.7
J6	350-360	68-75	15.1

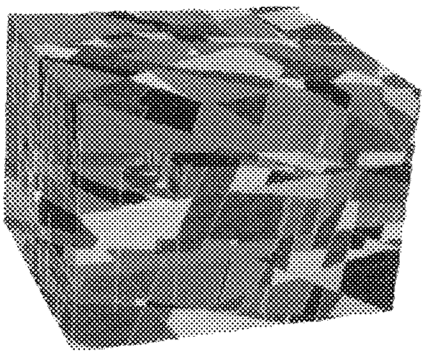

Fig. 1. The blocks distribution after adding the joint network.

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2.3. Field and Laboratory Studies

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STABILITY ANALYSIS OF LAVARAK UNDERGROUND POWERHOUSE CAVERN USING BACK ANALYSIS RESULTS

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Lavarak underground hydroelectric powerhouse station with 47 MW capacity is built 50 Kilometers North-East of Tehran in Iran. The depth of the cavern is 200 meters. The rock mass consists of a semi compacted conglomerate with clay matrix. The uniaxial compressive strength of the rock mass is 2.6 MPa which classifies it as a weak material with high modulus of deformation. Determination of rock mass parameters has been a goal which was followed in this project based on back analysis studies on a few number of monitoring instruments. In- situ rock stresses and the modulus of deformation are determined in this back analysis. The obtained results comply with the earlier studies performed by a couple of other consulting groups. Phase2 Program (Rocscience Inc.) was utilized for the present study. With the determined parameters, the stability analysis showed that the powerhouse cavern can sustain its stability and initial designed long tendons are not necessary. Elimination of these reinforcements resulted in considerable time and money savings.

Keywords: Stability Assessment; Lavarak Power Station; Phase2 Soft Ware.

MICROSEISMIC MONITORING AROUND LARGE UNDERGROUND STORAGE CAVERNS DURING CONSTRUCTION

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Brittle failure has been detected in over-stressed rock mass zone during the construction of underground oil storage caverns. The over-stressed rock mass zone is characterized by an excessive horizontal stress which is almost 3 times larger than the vertical component of the in situ stress. Physical signs of stress induced brittle failure became problematic as the bench excavation proceeded, which included a drastic increase in measured values of rock displacement, shotcrete stress and rockbolt stress as well as an increased frequency of popping sounds experienced by the underground workers.

In response to such problems associated with highly stressed rock mass, modifications were made to excavation designs and additional supports were installed. However, due to limitations of the existing monitoring method, a method of monitoring the overall behavior of the rock mass across the highly stressed region became necessary.

In this study, microseismic event monitoring system is applied to highly stressed rock mass around large underground storage caverns in order to ensure the structural stability of the caverns and the safety of the workers during excavation processes. The microseismic system consists of a data acquisition unit and a 3-dimensional array of 12 uniaxial accelerometers as shown in Figure 1.

The monitoring results indicate that seismic events are frequently induced by blasting activities as the excavation is followed by stress redistribution in the surrounding rock mass. Since the frequency of excavation-induced seismic events usually plateaus and disappears within the first three hours after the blast, special attention should be given to events occurring continuously after the three hours within the excavation damaged zone, as shown in Figure 2, as well as to those occurring without any specific relation to the blast.

Based on the accumulated microseismic event data, evaluation criteria are established for overall stability and integrity of the underground storage caverns, in which alerting levels of event frequency are determined for different ranges of magnitude. Interpretation of the monitoring data against the evaluation criteria enables overall stability assessment of the caverns and allows proper countermeasures to be taken.

Keywords: Microseismic monitoring; microseismic events; large rock cavern; underground storage.

KAERI UNDERGROUND RESEARCH FACILITY FOR THE VALIDATION OF A HIGH-LEVEL RADIOACTIVE WASTE DISPOSAL CONCEPT IN KOREA

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In Korea, an underground facility(KURF) for R&D related to high-level radioactive waste disposal in a geological formation is under construction in a granite body. The facility will be used for the validation of the safety, feasibility, stability, and appropriateness of Korean disposal system.

During the last years, site characterization, concept development, design, construction, and some in situ tests before and during the construction had been successfully carried out. Based on the geological, geotechnical, hydraulic, socio-economical conditions of the site, an optimum design of KURF could be suggested. The length of access tunnel length is 180m. There are two research modules, one is 45m long the other is 30m long, at the end of the access tunnel. To achieve the depth more effectively, a downward 10% slope was adapted. Fig. 1 shows the KURF layout. The construction of KURF was successfully done by using controlled blasting technique. Fig. 2 shows the blasting result at a research module.

During the construction phase, different in situ tests including hydraulic test and EDZ test had been performed. In this paper, important factors on KURF design and results from in situ tests during the site characterization as well as the construction will be summarized. The future plan for the operation, which is expected to be started late this year will also be discussed.

Keywords: Underground research tunnel, geological disposal, KURF, HLW disposal.

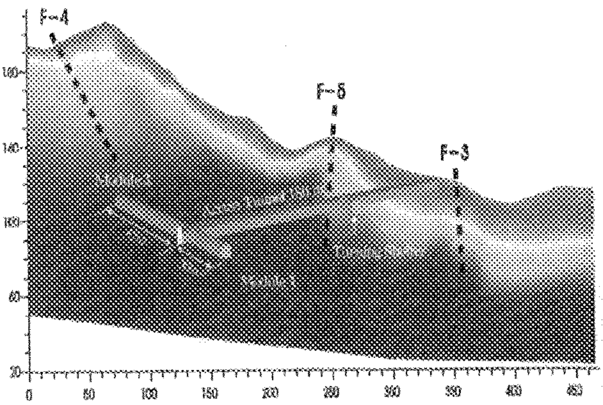

Fig. 1. KURF layout.

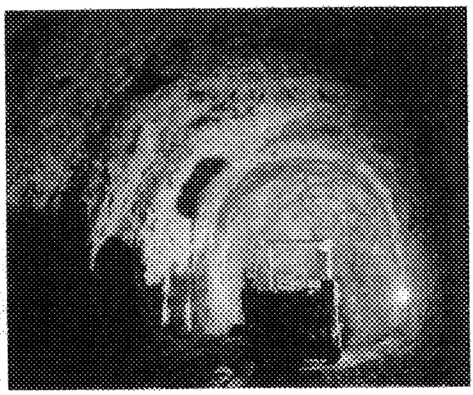

Fig. 2. KURF construction.

MONITORING AND ANALYSIS ABOUT STABILIZATION ON THE TAI'AN PUMPED-STORAGE STATION UNDERGROUND HOUSE

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The Tai'an pumped –storage station takes the important role in the electric network of Shandong province. The under-ground power-house lies in stiff and integrate granite expensive some fault. The dimension of the main power-house is 141.1×52×25.2m, and the area of the section is bigger in the domestic power-house. A lot of monitoring instruments are installed during instruction, including muti-point displacement detectors and anchor stress detectors and so on. Many information are received from those instruments. In this paper, based on the complete observation information of the pumped-storage station, the deformation datum of the rock mass and the stress of the embedded anchors are studied. And then the conclusion is the underground power-house are steady in principle. The excavation hasn't influenced on the power-house badly. The reading of the anchor stress detectors varied obviously at first and steadily finally.

At the same time, the numerical model of underground house is analyzed by using the finite difference method. During the computation, the simulation on the model is nonlinear and elastic-plastic. The characteristic of displacements and stresses of rock-mass is studied, and the dynamic variable process can be forecasted truly. FLAC^{3D} applies the contour map of min. and max. principle stresses, the vector map of the displacement and the distribution of the plastic zones.

At last, the displacement from computing and monitoring of some points in the rock are contrasted. The difference from computing and monitoring is less. The comparison of displacement of the points in the muti-point displacement detector are shown in the following table. That shows the simulation to wall rock in the power-house is correct and necessary. All of those can apply the reference to other stability analyses of large underground power-house.

Keywords: Case study, underground storage; numerical analysis.

Table Comparison of datum from observation and computation of M30.

Embedded depth		1.5m	3.5m	8.5m
Monitoring value	the forth layer excavated	3.1	3.18	2.73
	the fifth layer excavated	4.09	4.17	3.72
	the sixth layer excavated	5.26	5.09	4.8
Computing value	the forth layer excavated	2.31	2.01	1.24
	the fifth layer excavated	3.70	3.16	2.51
	the sixth layer excavated	5.18	5.06	4.29

DESIGN ANALYSIS OF EXPERIMENTAL LINED ROCK CAVERN FOR NATURAL GAS STORAGE IN JAPAN

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In this paper, we introduce the small experimental Lined Rock Cavern at the Kamioka mine of the ANGAS project and its design.

We investigated the design method for the LRC system in Japan in the previous project 1 (Nobuto *et al.*, 2003). According to the design method, the procedure of the design and numerical analyses were as follows:

(i) Stability of the rock mass against uplift force (pressure)
(ii) Stability of the test cavern and the support system under excavation
(iii) Behavior of the test cavern during operation (experiments)
(iv) Behavior of the concrete plug during operation (experiments)
(v) Discontinuous displacement between the cavern and the plug during operation (experiments)
(vi) Behavior of the steel liner during operation (experiments)
(vii) Stability of the manhole during operation (experiments)
(viii) Temperature variation of the cavern during injection and withdrawal

Based on the geological data and the design method, we designed the experimental LRC at the Kamioka mine shown in Figure 1. The details of the design and the numerical analysis are shown in the full paper.

Keywords: Lined Rock Cavern; Experimental Study; Natural Gas Storage; Concrete Plug; Steel Liner.

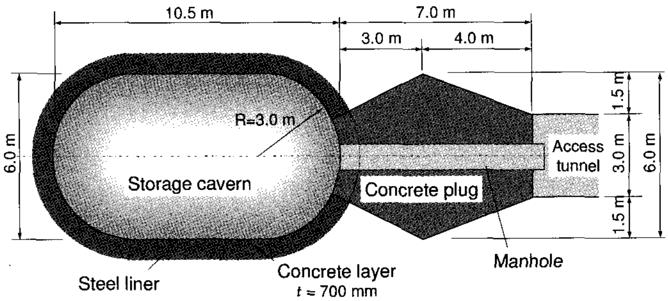

Figure 1. Schematic view of test cavern.

EXPERIMENTAL LINED ROCK CAVERN FOR NATURAL GAS STORAGE IN JAPAN

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Since 2004, we have been studying technologies for an underground natural gas storage system, a Lined Rock Cavern (LRC) gas storage system called ANGAS (Advanced Natural GAs Storage). In this paper, we introduce the project and discuss investigations of the rock mass properties based on the current work at the Kamioka mine in Japan. The experimental LRC and its design are also discussed in another paper related to this project at this symposium.

The purpose of the ANGAS project is to develop a suitable LRC system for Japan. Natural gas has been getting more important energy source in Japan. In the project, our goal is to ensure the design method for commercial types of the LRC system under the Japanese geological conditions of the limited distribution of hard rock and under its unique operation for shaving off the daily peak.

In order to confirm the validity of the LRC system and the key technologies (Fig. 1) for Japan, we have designed a small LRC (test cavern) located at the Kamioka mine in central Japan. The cavern is surrounded by the sedimentary rock mass called "Tedori layer" and mainly composed of sandstone and mudstone. Basic physical properties of the rocks, i.e. porosity, density at saturation, elastic modulus, uniaxial compressive strength, tensile strength and elastic wave velocity, etc., were investigated. The properties were evaluated mainly from laboratory tests of the core samples taken from the boreholes. Important mechanical properties for the design, such as a modulus of deformation, a creep ratio and a residual strain ratio of the in situ rock mass, were evaluated from the results of borehole loading tests and rigid plate loading tests performed in a part of the access tunnel of the cavern. In addition, initial stress of the rock mass was measured.

Also, we discussed the outlines of the test cavern and its conditions for design. The properties of the rock mass and initial stress conditions for design were evaluated from the above data.

We are now planning the system for measuring and monitoring during the experiments. The cavern is under construction and will be completed within 2006.

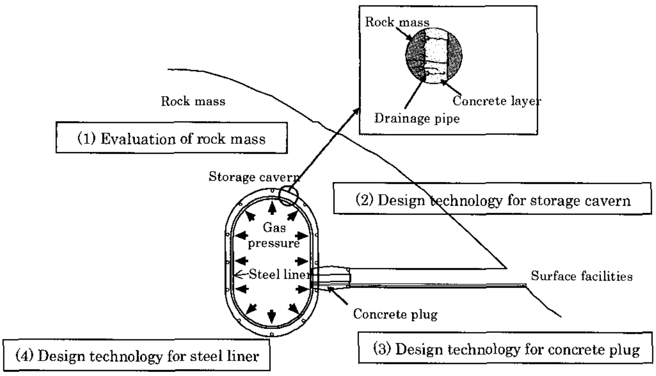

Fig. 1. Schematic view and key technologies of LRC system.

Keywords: Lined Rock Cavern; Rock Mass Properties; Experimental Study; Natural Gas Storage.

3. MINING

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INSTABILITY MODES OF ABANDONED LIGNITE MINES AND THE ASSESSMENT OF THEIR STABILITY

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The authors have been involved with some abandoned mines in Tokai Region of Japan and they have been investigating their performance and responses in long term and during earthquakes. In the first part, instability modes of abandoned lignite mines before the ultimate collapse are explained and their instability modes during earthquakes are described. In the next part of the article, Some stability assessment methods for abandoned room & pillar mines are proposed in order to investigate the effects of degradation and creep failure and earthquake loading. It is possible to assess the effects of some fundamental characteristics of the mines such as excavation ratio, overburden ratio etc and to estimate the collapse time of mines in long-term. The theoretical stability analysis method is based on seismic coefficient method for evaluating the stability of abandoned room and pillar mines during earthquakes. These stability analyses results indicated that many abandoned room and pillar lignite mines are prone to collapse in long term as well as during earthquakes. Therefore, some measures against the collapse or subsidence of these abandoned mines are urgently needed.

Keywords: Mine; stability; failure.

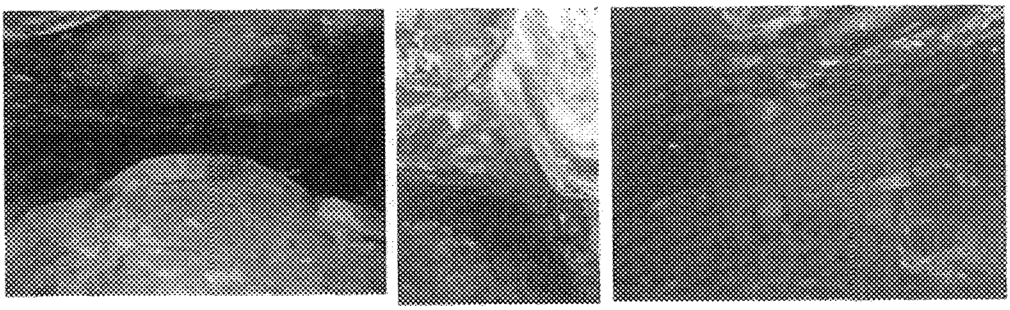
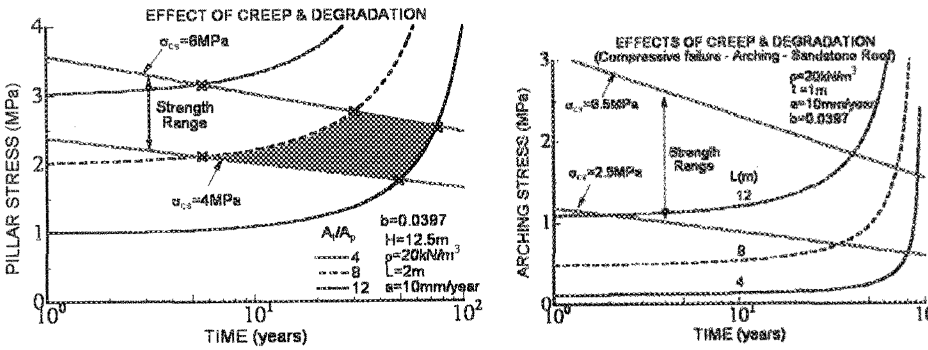


Fig. 1. Views of some instability modes in an abandoned lignite mine in Mitake town (Japan).



(a) pillar (b) roof

Fig. 2. Effects of degradation and creep loading on failure time of pillar and roof.

PHYSICAL SIMULATION OF FULL-SEAM MINING FOR A 20M THICK SEAM BY SUB-LEVEL CAVING MINING METHOD

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The practice of more than twenty years in China has proved that the mining 3.5m~5.0m thick coal seam by the fully mechanized mining with greater mining height and the mining 5.0m~15.0m very thick coal seam by a fully mechanized sub-level caving mining method are successful. If a coal seam thickness is more than 15m, a slice fully mechanized sub-level caving mining method is used generally. Then, is it feasible for a 20m very thick coal seam to be mined by the full-seam fully mechanized sub-level caving mining method with a greater bottom slice mining height? According to the condition of Suancigou mine No.6-1 coal seam, the caving characters of the top coal and roof, the expansion property on breaking and the stacking angle of the caved rock behind the powered support, the working loads of the powered support and the coal face recovery etc. of the full-seam fully mechanized sub-level caving mining method with greater bottom mining height for the 20m very thick seam have been systematically studied by using 1:30 physical simulation. The main testing results and conclusions are as follows.

(i) It is feasible in technology to use the full-seam fully mechanized sub-level caving mining method with greater bottom slice mining for the 20m very thick seam.

(ii) The first caving span of the immediate roof is 915mm (prototype 27.45m). The first weighting span of the main roof is 969mm (prototype 29.07m). Three periodic weighting spans of the main roof are respectively 216mm (prototype 6.48m), 702mm (prototype 21.06m) and 432mm (prototype 12.96m). And their average value is 450mm (prototype 13.5m).

(iii) According to the top-coal caving and drawing characteristic, the working face mining process may be divided into the initial mining phase, the transition phase and the top-coal normal caving phase. The face coal recovery n increases with the face advancing distance L. Its changing law is $n = 27.468\ln(L) - 59.47$. In the normal top-coal caving phase, the coal face recovery may reach 75%. The face coal recovery n increases or decreases periodically with the roof rock beam breaking periodically.

(iv) The circulation that the working loads of the support is more than the support setting load makes up 73% among the total circulations. The maximum support working loads occur when the first, second, and third periodic weighting of the main roof take place. 8500KN per unit working loads is suggested for the powered support in the actual working face under the simulation condition.

(v) There are the phenomena of the full top coal cantilevering towards mined-area and cutting down over the coal wall. The front-rear stability of the powered support must be considered when designing and electing support for the full seam mining very thick seam by the fully mechanized sub-level caving with greater bottom slice mining height.

Keywords: Full-seam mining, 20m thick coal seam, support loads, coal face recovery, top coal and roof caving.

NUMERICAL INVESTIGATION ON STABILITY OF THE ROCK PILLAR IN AN IN-SITU EXPERIMENT

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Using RFPA code, analyses have been carried out to investigate the stability of a rock pillar in a experiment for nuclear waste repositories, the numerically obtained stress field, temperature distribution, failure pattern of the pillar rock and associated AE events are all agree well with the in-situ data. Minor fracture initiation may take place in the vicinity of the boreholes after heating. Heating induces minor spalling at central pillar wall for 0.5 m sections below the tunnel floor, but the area of spalling is found to be limited. The 2D RFPA modeling indicated that microcracks/damages are localized and formed asymmetrically (distributed fractures around the holes) (as shown in Figure 1). Comparison of the failure mode and failure depth was also made between FLAC, PFC2D, FRACOD and RFPA2D results (as shown in Figure 2). These results confirm that at least some damages and micro cracking shall be formed in rock in the pillar area of interest, which was one of the main objectives in the pillar stability experiment. The core of the pillar remains intact for stress conditions corresponding to 120 days of heating which not only prove that the proposed technique provides a powerfully alternative and effective approach for the study on thermal-mechanical-damage coupling mechanism but also provide meaningful guides for the experiment design and associated engineerings.

Keywords: Numerical simulation; Rock pillar; Failure process; Thermal.

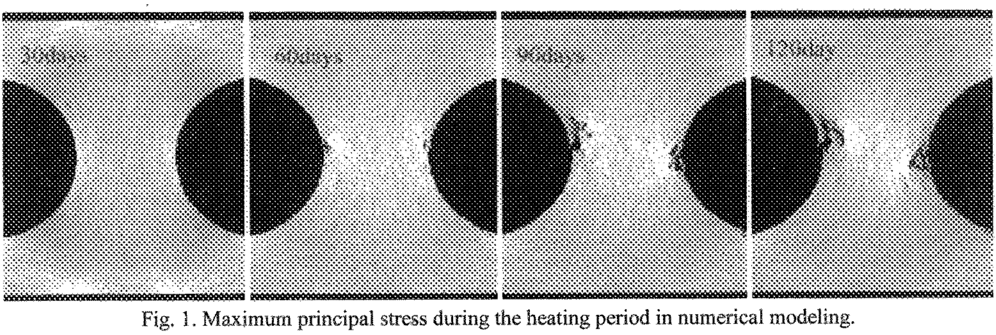
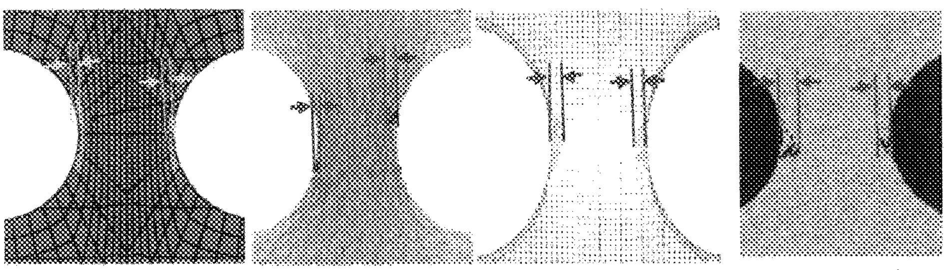


Fig. 1. Maximum principal stress during the heating period in numerical modeling.



(a) FLAC result (b) PFC result (c) FRACOD result (d) RFPA result

Fig. 2. Numerical results comparison.

STABILITY BEHAVIOUR OF ROADWAY INTERSECTION IN DEEP UNDERGROUND

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Coal mine roadway intersections are susceptible to ground control problem due to the inherent wide roof span, excessive stress and variable intersection geometrical shapes. At the present time, the stability problems of intersection in China coal mines have been paid attention, particularly in great depth of cover. Thus, a comprehensive research is conducted herein to investigate the effect of depth on the stability of the roadway intersection using 3D numerical modeling.

During the study, it is noted that the increments of rib deformation is up to 90%, which is greater than the increments of roof deformation (87%) and increments of floor deformation (84%) when the cover of depth varied from shallow to deep conditions. Also, the stress regimes for the normal span of the roadway and the large span of the intersection are compared to demonstrate the sensitivity of depth of cover on the stress regimes for the different geometry of the underground structures.

According to the study, the failure patterns have been identified, and some findings can be summarized as follows

(i) The stability of an intersection is strongly affected by both the depth of cover and the mechanical properties of surrounding rock masses.

(ii) Adverse stress conditions, in both horizontal and vertical stresses, are expected with an increasing of the depth of cover. Comparatively, the stress conditions of the intersection are much worse than the conditions of normal roadway, and the differences of stress regime increased with increasing of depth of cover. It implies that the geometry of underground structure plays an important role in stability, particularly when great depth is reached.

(iii) In shallow conditions, the roof stability is a major concern. In deep conditions, the rib stability also becomes an important factor, which strongly influences the stability of the intersection. Under current geological conditions, when the depth of cover reached 800 or more, the rib instability becomes major problem to be faced.

(iv) In shallow conditions, the rock mass distress is around 2.5m into roof strata, and in deep conditions, the distress is about 4.5m into roof and rib strata, which is 1.5times the height of the intersection. Accordingly, longer reinforcement elements are expected.

(v) When the depth of cover reached 800m, the failure concentrated around central roof strata of intersection, the ribsides and the rock pillar, which carried the major roof loads of the intersection. So, proper selection of the reinforcement method, parameters and locations are crucial for the stability of intersection in great depth.

According to the distress and deformation characteristics identified, the failure zones are determined, and the enhancement of reinforcement system for these zones are discussed and support guideline is provided.

Keywords: Rock Mechanics, Depth of cover, Intersection, Stability, Modeling.

EFFECT OF BACKFILLING ON STABILITY OF PILLARS IN HIGHWALL MINING SYSTEMS

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Final highwalls of open cut coal mines can form the starting point for other mining methods, such as underground or highwall mining. In its basic application, highwall mining is a technique utilized after the open cut portion of a reserve has been mined, sometimes prior to the introduction of underground mining.

Highwall mining is performed by a set of machinery positioned adjacent to the final highwall of an open cut as shown in Figure 1. The equipment is closely related to underground mining machinery, and is operated by remote control as it excavates the coal seam. This unmanned, remote operation contributes significantly to highwall mining's well-earned reputation as being one of the safest mining methods available today.

However, a large amount of coal tends to remain isolated and undeveloped as pillars due to previous indiscriminate mining operations performed by the use of an auger machine or a continuous miner. Moreover, the highwall mining causes highwall instability and surface subsidence.

Backfilling in the highwall mining would allow for a high coal extraction ratio while greatly reducing the threat of failure of the pillars and the highwall and damage caused by subsidence at the surface.

This paper describes the importance of backfilling system using wastes and discusses the effect of backfilling on pillar stability in highwall mining system by means of finite element analysis and laboratory tests.

Keywords: Highwall mining; Coal recovery; Backfilling; Coal pillar; Fly ash.

NEW METHOD OF ROOF-FALL HAZARD EVALUATION IN POLISH COPPER MINES

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Every year 30 million tones of copper ore are extracted in the three Polish copper mines. Mining operations are carried out by using the room and pillar method with advanced blasting technology. Roof support is exclusively of anchor type. A few dozen kilometers of galleries and chambers in each mine is a source of a roof-fall risk. All known methods of rock-mass roof state evaluation are applied, but none of them give information concerning the rock-mass state dynamic condition, usually resulting in the roof rock-mass-fall. Previous measurements have shown that roofs with velocity of stratification less than 1 millimeter per 24 hours are "safe". On the contrary, "unsafe" roofs reached the velocity up to several hundred millimeters per hour, one hour before roof-fall. Therefore, the roof's stratification velocity measurement *in situ* may give the reliable information about the current risk of roof-fall. Figure 1 shows the gauge designed for the task above. The gauge consists of the *top anchor*, integrated with the *bar* and the *rope*, permitting the anchor's strength reduction for quick installation in the roof borehole. The *measuring instrument* is integrated with the *bottom anchor*. It ensures detection of the roof stratification increase of the 1 micrometer value. Value of 0.6 mm / 24h, or less, is applied as criterion of the roof stability. It is equal to *ca* 6 μm per 15 minutes of *in situ* measurement. During the 3 years of testing the gallery roofs, all roof-state forecasts (stable - unstable) were documented as well as forecast's verification results (confirmed - unconfirmed). The roof was regarded as instable when at least one of the next events occurred: of 5 mm stratification alarm, the need of additional roof strengthening, or a roof-fall occurrence in the tested site. For 478 forecasts conducted, (a) 381 (79.71 %) of *confirmed roof stability* forecasts occurred, (b) 57 (11.92 %) of *confirmed roof instability* forecasts occurred, (c) 8 (1.67 %) of *unconfirmed roof stability* forecasts occurred and (d) 32 (6.69 %) of *unconfirmed roof instability* forecasts occurred. Cases (a) and (b) directly witness the methods for efficiency, cases (c) are false (potentially dangerous) *stability forecasts*, and cases (d) are false alarms. The described method is formally approved and applied in the Polish copper mines, especially in galleries characterized by "weak" roofs and with intensive haulage.

Keywords: Roof-stratification; Roof-fall; Roof-fall hazard.

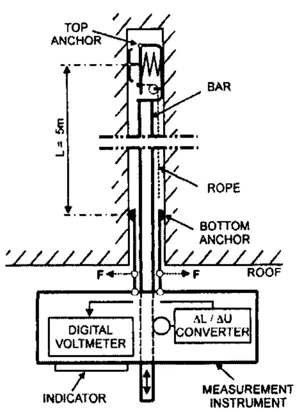

Fig. 1. Gauge for the roof stratification measurement.

DIMENSIONING OF SALT-ROCK PILLARS IN "POLKOWICE-SIEROSZOWICE" COPPER DEEP MINE BASED ON FEM

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Mining operations performed within the Northern part of the "Polkowice-Sieroszowice" deep copper mine are associated with a very thick (up to 200 m) and aerialy extended salt rock stratum. The understanding of the mechanism of interaction between the mine workings excavated within the salt-rock strata, required developing a multiparameter numerical analysis involving rheological aspects of the system behavior. The utilized laboratory based time-dependent parameters of saltrock deformation were verified using the monitoring data concerned with the time-dependent closure of one of the large salt caverns located in the Sieroszowice mine area, utilizing a back calculation procedure based on three-dimensional FEA.

Since in the area, new large chambers are planned to be mined-out, their safe dimensions had to be found. Therefore the 3-D bedded rock mass geomechanical model (Fig. 1) suitable for the considered area geological conditions was developed, utilizing the finite element method involving rock mass creep behavior as well as bed separation phenomenae.

In the paper there are presented numerical modeling results concerning the effect of salt chambers size on the entire system safety expressed by so called "safety margins" and conventional safety factors based on the Hoek-Brown theory of rock mass strength.

Keywords: Room-and-pillar; saltrock; stability; FEA.

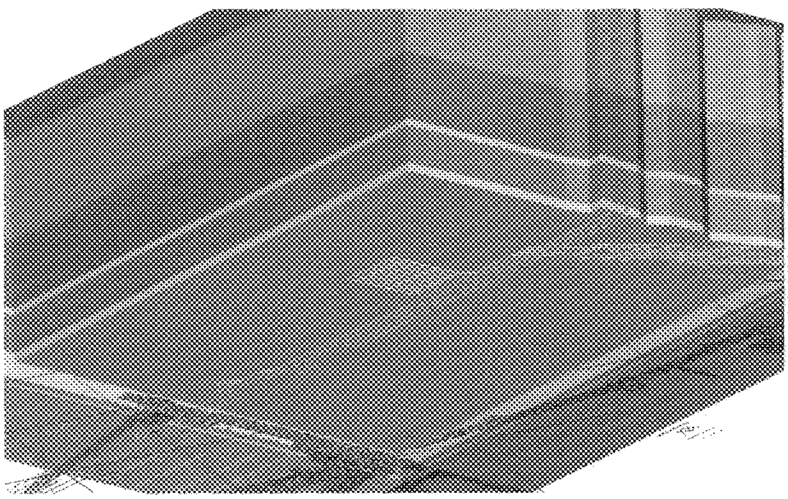

Fig. 1. Overall view of the FEM model representing rock mass surrounding salt chambers in the G-55 district in the Polkowice-Sieroszowice mine.

FAILURE MODES AND RESPONSES OF ABANDONED LIGNITE MINES INDUCED BY EARTHQUAKES

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There are many abandoned mines in Tokai Region of Japan and the authors have been investigating their performance and responses in long term as well as during earthquakes. The authors described briefly a real-time monitoring system and its application to the investigation of the stability and performance of an abandoned room and pillar lignite mine in Mitake town. During the monitoring period, two earthquakes relevant to the site occurred and their effects on the abandoned mine were observed both instrumentally and visually. Both geo-electrical potential and acoustic emission (AE) measurements are indicators of fore-coming instability problems related to seismic loading as well as degradation and creep of surrounding rocks and variation of surcharge loadings. The combinations of geo-electric potential measurements together with Acoustic Emission (AE) measurements can distinguish near-field and far-field events. A series of model tests and stability analyses on abandoned lignite mines was carried out to investigate the effects of earthquake loading. These stability analyses results indicated that many abandoned room and pillar lignite mines are prone to collapse during earthquake shaking. These results indicate that the shallow mines are prone to roof failure while the deeper mines are prone to pillar failure for actual strong ground motions during earthquakes. Therefore, some measures against the collapse or subsidence of these abandoned mines are urgently needed.

Keywords: Failure; mine; case study.

Fig. 1. A view of roof collapse in an abandoned lignite mine in Mitake town due to the 2004 Tokaido-oki Eq.

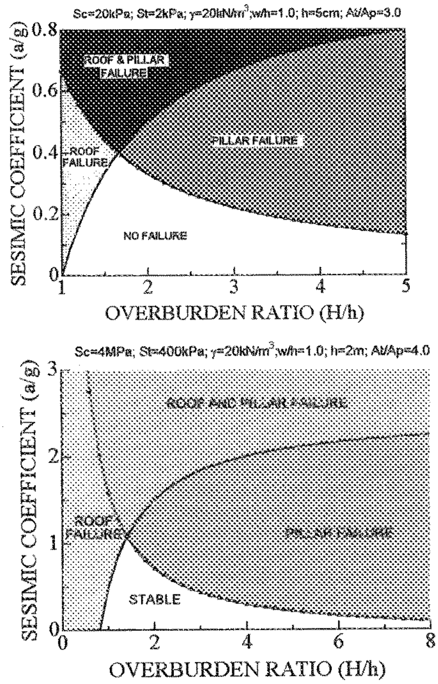

Fig. 2. Seismic stability charts for model and an actual abandoned lignite mines.

A DAMAGE MODEL OF ACOUSTIC EMISSION IN PILLAR FAILURE AND ITS NUMERICAL SIMULATION

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Based on the assumption that there exists a proportional relation between the number of microseismic events and microfractured elements, By using a developed numerical code, RFPA2D (Rock Failure Process Analysis), the process of the pillar rock burst is simulated and the simulated results reflect macroscopic failure evolution process induced by microscopic fracture and the spatial as well as the temporal distribution characteristics of acoustic emissions event hypocenter. Especially the heterogeneity of rock is considered which a good influence on the AE has resulted from the rupture of the rock.

Keywords: Rock burst; Damage model; Acoustic emission; Numerical simulation.

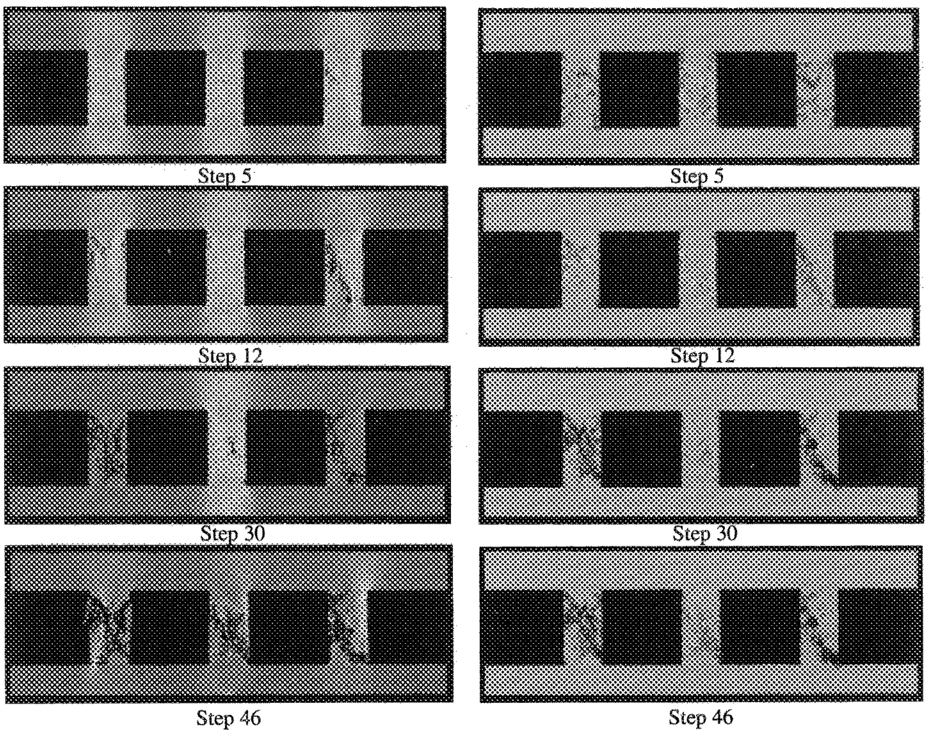

Figure 2(a). Pillar failure mode for shear stress distribution.

Figure 2(b). Pillar failure mode for AE distribution.

MEASURING ROCK MASS MODULUS OF DEFORMATION IN A STOPING-AFFECTED CROSS-CUT IN PONGKOR UNDERGROUND GOLD MINE

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Rock mass modulus of deformation is one of the important factors required for design work within the rock mass, especially the design of an underground structure. It can be determined indirectly from rock elasticity modulus measured in laboratory or rock mass quality or directly from *in situ* measurement.

In Pongkor underground gold mine, the first *in situ* measurement of rock mass modulus of deformation by using the Goodman's jack test was just recently carried out. The test was conducted in the Cross-Cut 6A Ciurug, Level 570. As the development and mining in Pongkor underground gold mine progress continuously, the measurement had to be carried out in an area that was affected by the underneath stoping activities. The effect of the stoping on the measurement results was investigated in this work.

Three boreholes were used for the test and they were drilled into footwall (Andesitic breccia rock mass) and into the Au-Ag orebody. In each borehole, three tests were conducted. In each test, eight measurements were conducted at eight different depths from the collar and at each depth four different loading directions were applied.

The underneath stope dimension in this work was larger than that of the previous work that also utilised the same boreholes. This larger stope produced higher induced stresses in the rock mass at the test locations and this higher induced stresses had to be encountered in the Goodman's jack test and consequently steeper gradient for the pressure-displacement curves were obtained. It was then revealed that there was a 20-30% increase in rock mass modulus of deformation when the stope advanced four metre vertically.

Keywords: Rock mass; modulus of deformation; underground mine.

CORRELATING APPARENT STRESSES PREDICTED BY MICROSEISMIC MONITORING AND TUNNEL DISPLACEMENTS MEASURED WITH CONVERGENCEMETER IN THE DOZ BLOCK CAVING MINE

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The stability of the extraction level excavations is one of the critical factors controlling efficient ore extraction in a block caving mine. If stability problems occur at the extraction level excavations, they are not easily remedied and normally have a severe impact on production. The stability of the extraction level installations can be examined by carrying out a geotechnical monitoring program.

PT Freeport Indonesia (PTFI) started to implement an intensive underground geotechnical monitoring program in the mid 90's to monitor displacements and stresses. The equipment included convergencemeter, multiple-point borehole extensometer (MPBX), and stress cells. Experiencing that convergence monitoring provided similar trends and information of extraction level loading history as other more sophisticated stress monitoring devices, PTFI then proposed the convergence monitoring as the main monitoring system in DOZ Mine, whereas MPBX and stress cells will only be used in special cases.

PTFI began the implementation of microseismic in 2002 with the installation of eight triaxial geophones at the extraction level of IOZ and DOZ mines. This first microseismic monitoring was successfully used to monitor DOZ cave propagation and rock mass response in the pillar between the DOZ and IOZ mines. Furthermore, having experienced nine rockbursts in DOZ Mine undercut and extraction levels in 2003, PTFI realised the limitation of the manual monitoring and proposed a new comprehensive microseismic monitoring for the DOZ mine.

Recently, PTFI is developing the the microseismic monitoring program and one of the development is the use of microseismic records in determination of a number of physical parameters, in particular, the apparent stresses built up in the rock mass on the boundaries of extraction level excavations. The predicted apparent stresses are then correlated with the tunnel displacements measured with the convergencemeter, because experiences in convergence monitoring revealed that the displacements data had similar trend with extraction level loading and damage histories.

The work conducted at three locations in forstertite rock mass and two locations in High Altered Locally Ore (*HALO*) rock mass showed that the cumulative apparent stress curves had similar graphical trend with the cumulative displacement curves. It can then be concluded that the microseismic monitoring system can be implemented for early detection of tunnel instability and early sign of support repair requirement.

Keywords: Block caving mine; microseismic; convergencemeter.

GEOTECHNICAL CHALLENGES IN THE DOZ BLOCK CAVE MINE

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PT Freeport Indonesia operates a copper-gold mine located in the province of Papua, Indonesia about 3,500 km east of Jakarta. Current ore production from the project is about 240,000 tonnes per day (tpd) of which about 40,000 tpd come from the DOZ Block Cave and the rest is mined from the Grasberg Open Pit. Ore production from underground mines will be gradually increased to reach about 200,000 tpd by 2018, replacing a large part of the production from the Grasberg Pit.

The DOZ Block Cave was originally designed at 25,000 tpd started at 2000, and is currently in the process of expanding to a sustained 50,000 tpd rate by 2007. The increase of production, complex geology, and the high rainfall in the area have resulted in significant geotechnical challenges in the design, scheduling and operation of the DOZ block cave including rock-burst potential, wet muck spills and differing degrees of fragmentation.

Rock burst potential has been identified in the Southwest of the DOZ, where the fine grained and stiff Ertsberg Diorite will be mined, especially since there are also several faults in the area. Management of wetmuck is another geotechnical challenge since the DOZ will be undercutting the old IOZ block cave, which is already connected to the surface subsidence area. Furthermore, coarse fragmentation is expected in the Ertsberg Diorite, which is expected to result in slower draw rates and lower drawpoint availability.

This paper outlines the geotechnical experiences during the mining of the DOZ Block Cave and the application of lessons learned in the management of the geotechnical issues to ensure safety of the miners and continued achievement of desired production rates from underground mining in the district.

Keywords: Block Cave Mine; Strain Burst; Wetmuck; Fragmentation.

MODELING OF MINING INDUCED DELAY OUTBURSTS IN TERMS OF MATERIAL DEGRADATION

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The delay outburst of coal and gas induced by mining excavation is a complex catastrophic unstable phenomenon that may involve the ejection of large volumes of coal, and often accompanied with gas such as methane, carbon dioxide or a mixture of carbon dioxide and methane. It has posed a great potential threat to facility operators and challenged many researchers from rock mechanics and rock engineering community. In this paper, a unique coupled gas flow and solid deformation numerical model named RFPA^{2D}-Flow is applied to simulate the evolutionary process of those catastrophic coal failures in underground collieries. Specifically, a finite element model, which incorporates the physics of gas flow in coal seam, the physics of coal deformation and creep failure in terms of material degradation, and the cross-couplings between them, is proposed. The model also incorporates the small-scale variability in deformation modulus and strength of coal and rock. Variability in modulus and strength is distributed the fine-scale resolution model according to a Weibull distribution where distribution parameter notes the level of heterogeneity. The numerical model is applied to simulate the whole process of coal and gas delay outbursts, including stress concentration, coal fracturing, gas pressure-driven expansion, and outburst. Numerical simulations indicate that delay outburst is a complex phenomenon involved interactions among gas pressure, stress and physico-mechanical properties of the coal and surrounding rocks and it can occur under a variety of conditions. The successful simulation of the whole coal and gas delay outburst process provides the basis to identify the outburst mechanisms, to parameterize the causative processes, and to define potential precursors to failure. Fig.1 gives an example of the mining induced delay outburst process and the associated stress field evolution.

Keywords: Delay outbursts; mining excavation; material degradation; numerical simulation; RFPA^{2D}-Flow.

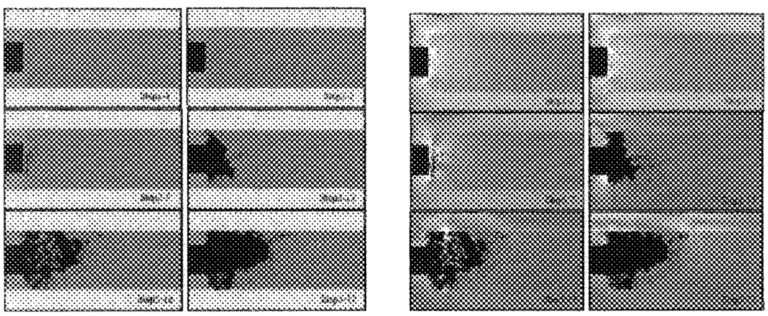

Fig. 1. Numerically simulated delay outburst process (a) and associated stress fields evolution (b).

STUDY ON CREEP BEHAVIOR OF COAL ROCK AND STABILITY OF TUNNEL THROUGH COAL-ROCK LAYER

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There are more than 8600 road and railway tunnels in China. And the total length of tunnels is about 4370 kilometers. The number and the length of Chinese tunnels stand first in the world. During the latest decade, Chinese civil engineering construction has made rapid development thanks to rapid economic growth. A lot of tunnels are being constructed to satisfy the traffic demands, but creep behavior of rock sometimes resulted in damage of tunnels. So the creep behavior of rock must be attached importance to. A tunnel, located in Chongqing China, through coal rock and sandstone, was taken as a case. And creep behavior of coal rock was taken the into account while analyzing the displacement of the surrounding rock and the stability of the tunnel.

Triaxial creep tests of coal rocks are carried out by the XTR01 electric-fluid servo compression machine, which is developed by Institute of Rock and Soil Mechanics of Chinese Academy of Sciences. And the loads are added by multi-step skill. By using the visco-elastoplastic rheological model, which can reflect the full creep process of coal rocks, the constitutive equation of coal rock is discussed. The creep parameters are determined by analyzing the test results. The comparison of the theoretical results and the test results show the good consistence between them.

Based on the rheological constitutive model programmed with VC++ and the experimental parameters, a two-dimensional explicit finite difference program FLAC (Fast Lagrangian Analysis of Continua) was employed in this study to simulate the behavior of the tunnel in the state of having and having no support system. The effect of different support time on the deformation of surrounding rock mass and the stability of the tunnel was analyzed as well.

The following conclusions could be drawn from the analysis results: the strength of the surrounding rock will decrease , and the deformation the tunnel will increase with the time if not supported; The creep rate will decrease with the support force decreasing, but the result is not obvious when the support force is more than a certain value. The displacement of having no support is more than that of having support. And the displacement will increase with the support time delaying, but the displacement will be little when supported 9 days later or more later(refer to Fig. 1). According to the time limit for the project and the displacement in different support time, the conclusion could be drawn that the 9th day is the optimum support time.

Keywords: Creep behavior; coal rock; stability; tunnel; support.

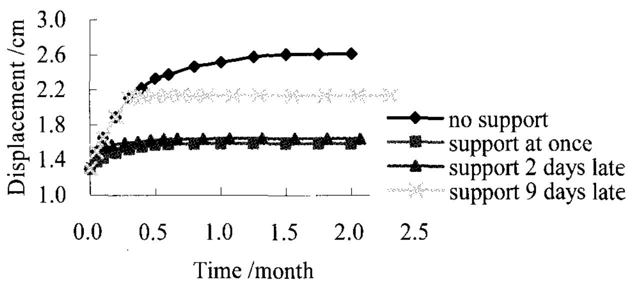

Fig. 1. Displacement-time curves with different support conditions.

3-D NUMERICAL SIMULATION AND CALCULATION MODELS DISCUSSION OF MINING SUBSIDENCE

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Ground subsidence due to coal mining are remarkable in many coal-producing countries, the mining subsidence hazard is more serious in China because of the dense population in coalfield areas. Mining subsidence can endanger land, buildings, traffics and communication lines. It also causes damages to ecological and social environment. So the research of prediction and control method of mining subsidence has very important meaning. Mining subsidence prediction is a key problem in coal mining. In this paper, the Finite Element Method is applied to predict subsidence, 3-D numerical calculation is done for mining subsidence of one mine, ground subsidence, and horizontal displacement, movement and deformation of the whole overburden are obtained in the course of coal mining. In addition, calculation results are discussed for different models including 2-D and 3-D models, with and without layer face in models, elasticity and elastic-plastic models. By the contrast among different calculation models, space effect and layer face effect in subsidence calculation are analyzed, calculation result shows that 3-D model with layer face effect is close to engineering fact.

Keyword: Mining subsidence; 3-D Numerical simulation; Calculation model; Space effect; Layer face effect.

DEVELOPMENT OF GIS-BASED SYSTEM FOR PREDICTING COAL MINING SUBSIDENCE AND ASSESSMENT OF ENVIRONMENT IMPACTS IN NORTH CHINA

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In China, there is about 2Gt of raw coal mined per year, which is one of the main parts of the total energy consumption. Consequently, subsidence due to underground mining is becoming one of the most important factors affecting the environment. In this paper, an integrated system has been developed within a Geographic Information System (GIS) platform, which included subsidence prediction model as well as environmental impact assessment model.

Probability Integral Method (PIM) is used to predict mining subsidence and the fuzzy logic model is applied to identify the classification of the mining-induced damage. Table 1 and Figure 1 present the damage classification results of calculation points in the case study.

Keywords: GIS; Coal Mining; Subsidence; Fuzzy logic; China.

Table 1. Damage classification results based on fuzzy model with the assumed mining design.

Calculation area	12.1km*7.5km=90.75km^2
Point number	121*75=9075 points
Mined area	16.2km2
Subsided calculation point	2129 points
No subsidence	1335 points (62.7%)
Class I	758 points (35.6%)
Class II	2 points (0.1%)
Class III	7 points (0.3%)
Class IV	27 points (1.3%)

Fig. 1. Damage classification based on fuzzy model after mining.

MASSIVE COLLAPSE OF PILLARS IN MINES

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In mines, pillars are often designed in regular arrays in a large area. When one pillar fails, the unbalanced load would be transferred to adjacent pillars causing these to be overloaded. This successive overloading process can lead to an unstable progressive domino effect so that large areas of the mine collapse massively. This type of failure occurred in Coalbrook coal mine in South Africa in 1960, which resulted in the collapse of 900 pillars and the loss of 437 lives. Renormalization group theory is an effective tool to describe the scale behavior of large scale system. In the paper, the renormalization group approach is used to study the systematic stability of the pillar groups, and the iterated cascading collapse function of varied scale is obtained. Then the critical probability of overall collapse is presented, which simplifies the identification of the systematic collapse of pillar groups to the calculation of the pillar failure probability. Finally, examples are offered to evaluate the stability of pillar cluster in Dabaoshan mine in China. The method offers a new view point to understand the overall collapse mechanism of the pillar groups and can serve as a reference for other application.

Keywords: Collapse; pillar; mine.

DYNAMIC SIMULATION AND OPTIMUM ANALYSIS ON CONSTRUCTION PROCESS FOR JOINT ARCH TUNNEL IN PARTIAL PRESSURE

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In this paper a 3D FEM model is established to simulate the construction process of Hong Jia-wu joint arch tunnel. The vertical displacement of arch crown, displacement and stress of middle wall, plastic area distribution of the surrounding rock, and stress of primary and secondary lining are obtained in different simulation condition. These calculated results are compared with the measured data in site. These two values are very close. This proves that the model and its parameter are correct. So we can analyze these values for different purpose.

Then we can get some valuable conclusion: a) it is more reasonable to construct the shallow underground hole firstly than the deep underground hole of joint arch tunnel. b) the construction method of cutting face is very important to the stability of the joint arch tunnel in partial pressure. By contrasting the plastic area of surrounding rock and arch crown's vertical displacement, we find that the three pilot hole method is better than the middle pilot hole method c) the partial pressure of middle wall is great. The stress in the top of middle wall is bigger than that in the bottom. The displacement of middle wall is very uneven and influenced obviously by the different construction order. d) in order to keep the surrounding rocks' stabilization the two cutting faces of joint arch tunnel must be staggered some distance. The distance that the latter construction face lags the former construction face is about double times diameter of the joint arch tunnel' single hole. e) the single hole's cycle length of joint arch tunnel is determined by the surrounding rocks and the supporting condition of tunnel. To the joint arch tunnel of this paper the reasonable cycle length is about 4m by computation. The above analysis results provide a scientific basis and technical guidance for the construction optimization of joint arch tunnel in partial pressure.

Keywords: Joint arch tunnel; 3D FEM; Optimum analysis; Partial pressure.

4. BLASTING AND DYNAMICS

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EFFECT OF STRESS LEVEL AND EXCITATION SIZE ON COMPRESSION WAVES IN JOINTED ROCKS

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The stress levels and the magnitude of excitation rarely affect elastic wave velocities in homogeneous media, such as an intact rock, while elastic wave velocities are more related to the magnitude of the excitation and stress level in particulate materials. Generally the wave velocity measured from the resonant frequency using steady state vibration decreases as the strain amplitude increases. However, a propagating compression wave front travels faster with increasing amplitude of source excitation in particulate materials, steepening the shape of the wavelet front. The compression front increases the mean stress in the particulate materials, which affect the stiffness of the medium and the velocity of wave propagation, leading to shock waves. Shock wave propagation in rock masses induced by blasting or any large amplitude source has drawn attention in underground mining and civil engineering. Although the effect of the excitation magnitude and stress level on compression waves can be of concern in seismic investigation practices, adequate laboratory simulations for this phenomenon are rarely performed. Thus, this paper explores experimentally the effects of excitation magnitude and stress level on compression wave velocity in jointed rock mass.

The compression wave velocity of jointed rocks is measured by detecting the arrival time with different stress levels and with different magnitude of excitation to identify how the state of stress and the magnitude of excitation affect the wave propagation in jointed rocks. The results show that the compression wave velocity increases with increasing source amplitude. Also, the compression wave velocity increases with increasing stress levels. Signals received from larger source excitation show an earlier arrival and a steeper rise, which is related to shock wave generation. It is found that the influence of excitation magnitude on the wave velocity decreases as the state of stress in jointed rocks increases. As the source amplitude increases, compression wave velocities become less dependent on stress level. Since the compression wave velocity in jointed rocks is governed by the stress level, this should be reflected in rock mass assessments that make use of the compression wave velocity. Large excitations are generated in the near field of most sources.

Keywords: Compression wave; jointed rock; stress level; large strain; magnitude of excitation.

WAVE PROPAGATION AND ATTENUATION IN DISCONTINUOUS ROCK MEDIA

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In this paper, the wave attenuation across the discontinuous geological media is theoretically and experimentally studied. In theory side, the classical method of characteristics for elastic wave propagation in elastic medium is introduced, and the method of characteristics is combined with the displacement discontinuity theory for behaviors of fractured rock mass, to solve for the wave transmission across the fractures. In experiment side, the standard Split Hopkinson Pressure Bar (SHPB) tests are carried out to simulate the wave propagation across the rock fractures. The effects of several parameters on the wave transmission are investigated experimentally, including the thickness of discontinuous layer and the moisture content within discontinuous layer. The variations of the ratio between the peak value of transmitted wave and incident wave with respect to thickness and moisture content are presented in Fig.1 and Fig. 2, respectively. It is found that the increase of thickness and moisture content will both attenuate the wave transmission across the rock fractures. In addition, the stress-strain relation of sand layer in SHPB tests is also derived for the further theoretical study.

Keywords: Wave propagation; Rock fracture; Split Hopkinson Pressure Bar.

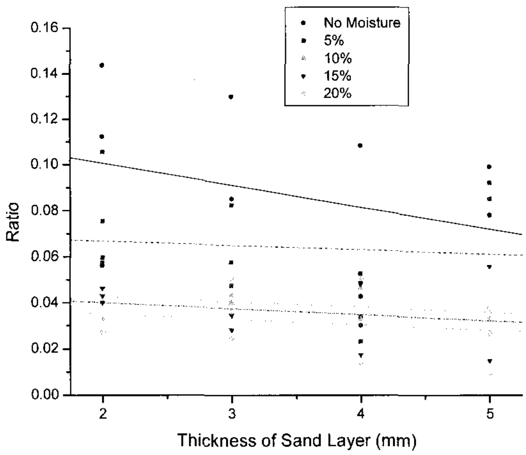

Fig. 1. Effect of moisture content on the ratio.

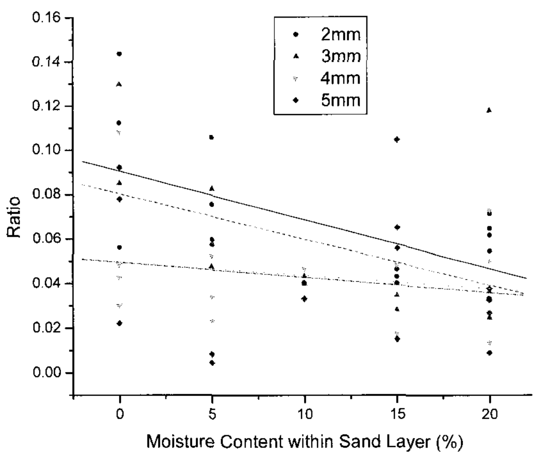

Fig. 2. Effect of thickness on the ratio.

TEST AND THEORY STUDY ON MIDDLE-DEEP CUT-HOLE BLASTING IN HARD ROCK TUNNEL

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With the coal situation is getting better and better, in order to quicken the mine deploitation speed, enhance the ratio of coal mining and tunnelling, Shuangyang Mine put money into technical renovation. It is one content to change the traditional manner of shallow hole blasting to the middle-deep holes and whole section blasting. Aiming at the problem of poor blasting effects that exists universally in hard rocks, The middling-and-deep-hole blasting tests that have been done in hard-rock tunnels at Shuangyang Mine, the blasting tests results of different kinds of digging forms have been got in this paper, and according to the dividing principle of equivalent charging structure of columnar charging, the cutting blasting parameters have been calculated theoretically. Through comparing, that compound-barrel digging is completely fit for middling-and-deep-hole blasting, and the utilizing rate is high. At last, some valuable achieves of tunnel blasting have been summarized, and presenting out the least space between holes that can abstain passivation effects of explosives . According to the construction fact of Shuangyang Mine, some concrete improving suggestions have been given, which can been referenced to other similar works.

Keywords: Digging blasting; digging form; middle-deep cut-hole blasting; hard rock tunnel.

CALCULATION OF THE BURDEN OF PERIPHERY BLAST-HOLES IN SMOOTH BLASTING FOR DEEP TUNNEL DRIVING

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With tunnels going into deeper ground, the residual stresses at tunnel depth become larger. So the effect of the residual stress should be taken into account when the smooth blasting parameters being calculated for driving the deep tunnel.

In recent years, the effect of the residual stress on rock blasting has been paid attention to. XIAO Zhengxue, *et al*'s conclusion is that rock fragmentation by blasting is attributed to the action of both static residual stress and dynamic stress from charge explosion, and DAI Jun has presented the calculation formulae of separation of holes for tunnel smooth blasting with the effect of both static and dynamic stress being considered, *etc.*

But, the calculation method of the burden of periphery blast holes is not been given yet till this time. In the present paper, the mechanism on fragmentation of burden rock of periphery blast holes in smooth blasting is investigated, based on the Livingston's theory of blasting crater, and then the calculation formula of burden of periphery blast holes is presented. The validity and applicability of the formula are also discussed, and the effect of residual stress analyzed as well.

From the studies, the conclusions are drawn that with the tunnels going to the larger depth, the effect of the residual stress has become so visible that it needs to be taken into account when the burden of periphery blast holes for smooth blasting is calculated, that for hard rock, the larger the residual stress is, the smaller the burden is, but for soft rock, the burden changes inversely, and that the effect extent is relevant to the physical and mechanic properties of rock and the tunnel radius.

The conclusion from the paper is of importance to the determination of the reasonable blasting parameters for driving a tunnel at depth so as to realize better smooth blasting result.

Keywords: Tunnel at depth, residual stress, smooth blasting, burden.

VALIDATION OF UDEC MODELING 2-DIMENSIONAL WAVE PROPAGATION IN ROCK

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The Universal Distinct Element Code (UDEC) was extensively applied and verified in 1-Dimensional wave (such as seismic wave) propagation in jointed rock masses. The numerical study in 1-D wave problem often started from verification of UDEC modeling 1-D wave propagation in jointed rock masses by comparing the numerical results with the analytical solution or the solutions from the method of characteristics. However, for a practical dynamic problem in a rock mass involving wave propagation from an underground explosion source (such as a blasting in a borehole or an accidental explosion in a tunnel), the problem becomes 2-Dimensional. For 2-D wave propagation in rock masses, only limited UDEC applications were reported in the literature for the analysis of some specific field cases. Probably due to its limited applications in 2-D wave problems in rock engineering, the reliability of UDEC in 2-D wave propagation in rock masses has not been thoroughly investigated. So far, literature review only gives one verification example in 2-D wave problems in rock, 'line source in an infinite elastic medium with a single discontinuity'. As a very fundamental part in the numerical study on 2-D wave propagation in rock masses, this paper aims to verify UDEC modeling 2-D wave propagation in rock. The case of harmonic wave from a cylindrical cavity in an elastic medium is chosen to perform the study.

In the numerical analysis in this verification study, a compressional harmonic wave is applied on the boundary of a cylindrical cavity with infinite length in an infinite elastic rock. Two cases, designated as case A and case B, are considered with following different conditions: (1) different radii of the cavities, (2) different frequencies of the input harmonic waves, and (3) different rock materials. In the UDEC models for case A and case B, the radial stress, the displacement and the velocity waves at several monitoring points are recorded and processed. By contrast, in the analytical calculation in this verification study, the radial stress waves at the corresponding monitoring points in the UDEC model are calculated from the equation given by Graff, while the displacement and velocity waves are calculated from the equations given by Hefny. Based on the above mentioned analytical equations, A MATLAB file is coded to calculate the analytical radial stress wave, displacement wave and velocity wave.

The amplitudes of the radial stress, the displacement and the velocity waves obtained from UDEC are compared with the calculated analytical solutions for the two cases. The comparison between the analytical solutions and the obtained UDEC results shows that, with small mesh size in the UDEC model, UDEC can simulate 2-D wave propagation in rocks with less than 2% numerical error under different conditions incorporated in case A and case B. It indicates that UDEC is capable of modeling 2-D wave propagation in rock with high degree accuracy.

Keywords: 2-Dimensional wave; analytical solution; numerical solution.

PENETRATION DEPTH OF LONG-ROD INTO GEOMATERIALS

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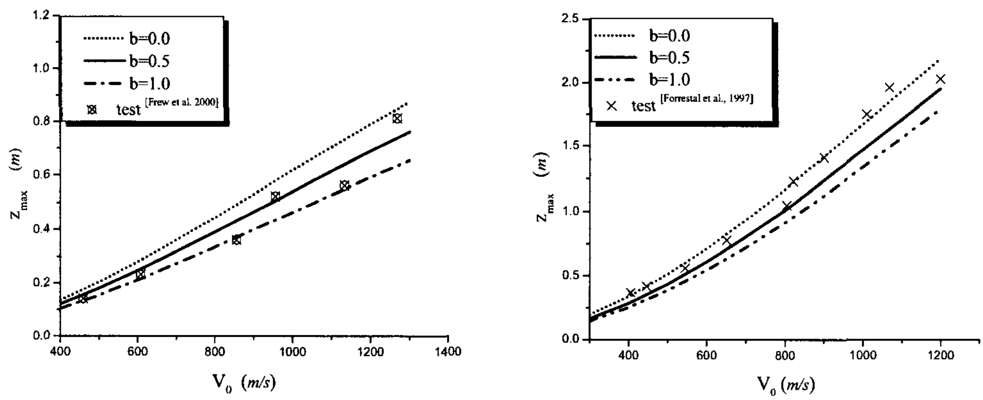
The investigation of the penetration not only has important realistic significance for improving the army defending capability, but also has momentous consulted worthiness for improving the attacking capability of high tech routine weapon.

During the penetration, one of obvious phenomenon in geo-materials is the dynamic fracture process. The damaged theory has been considered to be more suitable for the cleavage analysis of the geomaterials (Krajcinovic, 1996; Lorrain and Loland, 1983). When a brittle material is subjected to a tensile stress, it will be fail if the value of the stress is larger than its static strength (Yang et al., 1996; Ma et al., 1998). In this paper, the elastic-plastic and damage properties are considered together for geomaterial, and the static tensile strength is used as the point when the damage zone begins to appear.

How to define the failure criterion of the targets is critical for better analyzing penetration problems. The unified strength theory, which was suggested by Yu (1991, 2004), considers all of the components in stress space, covers a series of strength theories and can be used for different kinds of material.

In this paper, based on the cylindrical cavity expansion theory, the unified strength theory is applied as the failure criteria to modeling penetration, and the elastic-damage-plastic model of geomaterial target is considered. The relation between cavity radial stress and velocity at cavity-surface can be obtained by analyzing the distributions of stress and velocity of the target material. The attacking capability, penetration depth, of a long-rod can be assessed from the derived relation as the rod impacts and penetrates the geomaterial target with initial velocities of 300~1200m/s. By comparison with the test results, we can see that the final normal penetration depth not only depends on the mass and shape of rod, but also depends on the failure characteristic of target material. So a reasonable failure criterion should be adopted for the penetration model analysis.

Keywords: Penetration, Long-Rod, Geomaterial Target, Unified Strength Theory, Final Depth.



(limestone target) (concrete target)

Fig. 1. Relation between final depth and impact velocity of the rod.

STUDY ON THE STABILITY SAFETY OF DANGEROUS ROCK NO.2 IN SUOFENGYING HYDROPOWER STATION

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The dangerous rock No.2 in Suofengying hydropower station is located in the upper part of the intake in the right abutment. It is composed of 7 main vertical cracks and 4 soft layers. The bottom is on the soft rock. The height of the rock is about 180m and the total volume is about 78.6× $104m^3$. The possible failure modes are overturning, sliding along the bottom and sliding along the soft layers. It will bring the disaster if the failure is occur in any period of construction and operation..Some measures are used to prevent the failure of the dangerous rock. The continuous beam on the top and the pre-stressed cables through the cracks are used for the upper part. The anchor concrete beams and piles through the soft layers are used in the lower part. The piles against sliding of the bottom rock are used near and outside the dangerous rock No.2

The stability of the dangerous rock No.2 in the natural state and after reinforcement are studied by use of 3-D nonlinear finite element method. The thin layer element with the yield criteria of Mohr-Coulomb is used to simulate the soft layers. The anchor concrete beams and piles are simulated with beam element and the nodal displacements around the contact surfaces are expressed as the linear function of the central nodal displacements. The gradually strength decreased method is used to determine the most possible failure modes and the iterative method is used to determine the safety factor for the given failure mode.

In the analysis, the right of the dangerous rock No 2 is constrained horizontally, the front surface and back surface are restricted vertically, the bottom of the rock is fixed to simulate the effect of the mass. It is noted from the results that not only the horizontal displacement of the upper part of the dangerous rock No 2 is controlled by the prestressed anchor ropes, but also the horizontal displacement of the lower part of the dangerous rock No 2 is not developed because of the effect of the prestressed anchor ropes without piles.

The piles effectively act on reducing the horizontal displacement of the upper and lower of the dangerous rock No 2. The horizontal displacement is decreased from 0.13m to 0.002m. It is shown that the stability of the upper part is influenced by the stiffness of the soft basement.

It is noted from the displacement after reinforcement that there still about 3cm horizontal displacement in some parts of mudstone T1y3 though it is reduced. It is lead by the material characteristic of the mudstone itself, the grout is suggested to treatment the mudstone T1y3.

The stability of the dangerous rock No.2 in the natural state and after reinforcement are studied by use of 3-D nonlinear finite element method. The safety factor (K=1.0213) under the natural situation is obtained and it shows that the dangerous rock No.2 is in the limit state and some reinforcement measures are necessary. It reveals that the reinforcement measures used for the dangerous rock No.2 are effective.

Keywords: Stability; dangerous rock; 3-D nonlinear Finite element method; slide plane.

IN-SITU EXPERIMENTAL STUDY ON BLASTING VIBRATION WAVE PROPAGATION RULES IN COMPLICATED UNDERGROUND TUNNEL GROUP

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Three parallel tunnels with large sections and short distances to each other were excavated during the Xiluodu hydroelectric station construction in its right bank. There were several parallel and crossed working faces, which caused great blasting excavation interference and serious safety problems. Propagation laws of blasting vibration wave in the excavated tunnel and its adjacent tunnels were studied through in-situ experiments of blasting vibration in complicated tunnel group. The result showed that under the same blasting condition, when a tunnel was blasted, the propagation law of particle vibration velocity in the excavated tunnel was different from that in its adjacent tunnels. The propagation of the particle vibration velocity in adjacent tunnels had the magnification effect. The attenuation and the magnification effect were different when the blasting excavation type, the free faces and the confined effect changed. The propagation of the particle vibration velocity was stronger and the attenuation was faster when there were fewer free faces and more confined effects. This result could benefit the safety of the complicated tunnel group construction and help to improve the stability of surrounding rock and supporting design.

Keywords: Hydroelectric station; complicated tunnel group; adjacent tunnel; blasting vibration; in-situ experimental study; magnification effect.

BLASTING VIBRATION ANALYSIS IN SHALLOW BURIED TUNNEL EXCAVATION

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In order to control the harm of the blasting vibration, the new forecast method of blasting vibration and the real-time monitor was carried on to the project. The monitor plan, the monitor data and it's the analysis achievement of the blasting vibration are introduced in the paper. The weaken rule of the ground particle vibration velocity peak value was obtained in Taishan granite:

Horizontal particle velocity $$V = 85.2510\left(\frac{Q^{1/3}}{R}\right)^{1.5298} \tag{1}$$

Vertical particle velocity $$V = 204.5474\left(\frac{Q^{1/3}}{R}\right)^{1.7806} \tag{2}$$

BP neural network prediction model of peak amplitude of particle vibration velocity for the tunneling blasting vibration was proposed in ground. And compared with the real-time monitor data, the new forecast method is proved surpassed the traditional forecast method. In view of the fact, the blasting vibration characteristic of the shallow buried tunnel excavation is differently to other blasting type vibration characteristics. The aid of the wavelet transformation analysis, the distributed rule of the frequency spectrum and the energy was obtained on the typical vibration signals of the shallow buried tunnel excavation blasting. as following:

1. Horizontal and vertical main frequencies are 60.6391 and 65.2648 Hz respectively. Considered the above result and analysis of section 3that horizontal peak particle velocity is bigger than vertical one, we conclude that the damage of horizontal vibration on surface buildings could be worse than that of vertical one.
2. Small wave analysis shows that main frequency spectrums for all measured points are nearly the same. It is mainly due to the addition of each wave from each instantaneous delay explosion. By adjusting the delay time, we may change the particle velocity of blasting wave and reduce their frequencies.
3. Energy in horizontal direction is mainly in the second spectrum (30-60 Hz) with relatively narrow energy distribution scope. While energy in vertical direction is mainly in the 2nd (30-60 Hz) and 4th (90-120 Hz) frequency spectrums with relatively narrow energy distribution scope as well.

The research achievements will enrich the propagation theory of the blasting earthquake wave and play an important role in guiding the blasting construction of the tunnel excavation project and assuring the safety of ground buildings.

Keywords: Tunneling blasting, Blasting earthquake wave, Vibration monitoring, Wavelet analysis.

DYNAMIC RESPONSE OF SURROUNDING ROCKMASS INDUCED BY THE TRANSIENT UNLOADING OF IN-SITU STRESS*

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On the basis of the determination of explosion loading and excavation load during tunnel excavation with method of drilling and blasting, dynamic response of surrounding rock induced by blasting load and transient unloading of in-situ stress are discussed. The results show that the transient unloading of in-situ stress may be an important factor for the dynamic response of surrounding rock mass, and the maximum vibration of surrounding rock mass could be induced by either the blasting load or the transient unloading of in-situ stress caused by the initiation of the most outside rounds of breakage holes on excavation workface depending on the value of in-situ stress. For the case of low in-situ stress, the vibration in surrounding rock is induced mostly by blasting load. However, when in-situ stress increases, the PPV induced by the transient unloading of in-situ stress could exceed that induced by blasting load after certain depth in surrounding rock mass. If in-situ stress increases further, for example higher than 10MPa, the PPV induced by the transient unloading of in-situ stress could be much higher than that induced by blasting load. The vibration induced by excavation of inlet water tunnel with method of drilling and blasting at Pu Bu Gou Hydropower Project is presented as an example of the transient unloading of in-situ stress of rock mass, where the rock is granite with an initial stress about 3.0MPa, shown in Fig. 1. The monitored seismic data verifies the result of theoretical analysis.

Keywords: Tunnel; excavation; dynamic response; transient unloading; in-situ stress.

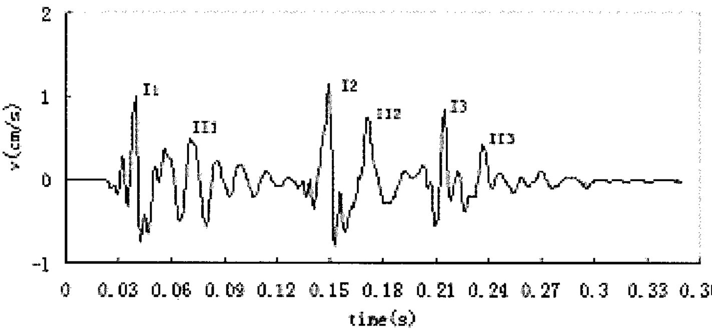

Fig. 1. Monitored vibration curve versus time (I1, I2 and I3: vibrations induced by blasting loads. II1, II2 and II3: vibrations caused by the transient unloading of in-situ stress).

RESEARCH ON FROZEN WEATHERED ROCK BLASTING TECHNIQUES IN SHAFT

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Blasting of frozen rock is under negative temperature and the frozen-tube is near, so the blasting is difference from the ordinary rock, and raises the blasting efficiency under the requirement of the safety of the freezing tube. Firstly failure criterion of freezing tube is established. Secondly blastability of frozen rock is analyzed. According to properties of frozen rock, blasting parameters are calculated and designed. Combining the blasting practice of artificially frozen rock of shaft, preparation works before blasting are proposed, such as the deviation calculation of freezing tube. In light of the characteristic of frozen rock and shaft area, drilling machine and construction machine are chosen. By blasting practice the blasting techniques protects the safety of freezing tube and raise the construction speed.

$$R=\frac{\sum_{i=1}^{n}(\sqrt[3]{q_i}\cdot r_i)}{\sum_{i=1}^{n}\sqrt[3]{q_i}} \qquad Q=\sum_{i=1}^{n}q_i(\frac{R}{r_i})^3 \tag{1}$$

Where R is equivalent distance, m; Q is equivalent charge, Kg; q_i is charge for ith blast hole, kg; r_i is the distance between freeze pipe and ith blast hole, m.

Under the prerequisite for pledging the safety of freeze pipe, Shock velocity by blasting should be controlled in certain critical value, freeze pipe would be broken if it exceed the critical value[6].

Shock failure criterion by blasting can be determined as:

$$K_m(\frac{Q^{1/3}}{R})^{\alpha}\le V_m \tag{2}$$

Where, V_{m} is critical value of shock velocity by blasting, m/ s; K_m, α are coefficients.

Table 1. K_m, α value for the difference kinds of rocks.

Types of rock	K_m	α
Solid rock	50~150	1.3~1.5
Medium strength rock Frozen weathered rock	150~250	1.5~2.0
Soft rock Weathered rock	250~350	2.0~2.2

The safety distance can further be deduced as:

$$R\ge Q^{1/3}\cdot(\frac{K_m}{V_m})^{1/\alpha} \tag{3}$$

Table 2. The blasting parameter of frozen weathered rock.

Home name	Hole numbers	Holedepth (mm)	Ring space (mm)	Burden (mm)	Hole space (mm)	Charge Kg/hole	Charge Kg/ring	Initiation sequence	Join line
Cut hole	1-6	2200	1200		630	2.50	12.0	1	Closed
Cut hole	7-18	2200	2400	600	630	2.50	19.2	2	reverse
Caving hole	18-36	2000	3800	700	660	1.50	21.6	3	parallel
Caving hole	37-60	2000	5200	700	680	1.50	28.8	4	
Periphery hole	61-100	2000	6200	500	500	1.00	32.0	5	
Total			q=1.9kg/m^3				113.6		

Keywords: Frozen weathered rock; blasting parameter; freezing tube.

ASSESSMENT OF BLAST-INDUCED VIBRATION IN JOINTED ROCK MASS

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Since blast-induced vibration may cause serious problem to the rock mass as well as the nearby structures and human beings, the prediction of blast-induced vibrations and assessment of their effects must be performed prior to actual blasting activities. For that reason, dynamic numerical analyses have been increased recently in order to analyze the effect of the blast-induced vibration. Most of the previous studies, however, were based on the continuum analysis unable to consider rock joints which significantly affect the wave propagation and attenuation characteristics. In addition, they almost adopted pressure curves estimated by theoretical or empirical equations as input detonation load, thus there were very difficult to reflect the characteristics of propagating media.

In this study, therefore, we suggested a dynamic distinct element analysis technique which uses particle velocity waveform obtained from a test blast as an input detonation load. In order to estimate the particle velocity waveform at the source of explosion, the measured particle velocities at surface were amplified with a compensation factor for the energy loss due to the traveled distance. A distinct element program, UDEC was used to consider the effect of rock joints. Joint network was generated statistically based on the geological survey result by using joint generation module in FracMan program. The 2-D joint traces for UDEC input was extracted by cutting the generated 3-D joint network. In order to verify the validity of proposed method and to determine the input parameter for numerical modeling, the test blast was simulated. The predicted results from the proposed method showed a good agreement with the measured vibration data from the test blast when applying 5% damping coefficient of rock mass. The predicted blast-induced vibration at the village near planned road tunnel construction site was less than the permitted standard, hence we may concluded that the blast design seemed to be done appropriately.

Keywords: Blast-induced vibration; jointed rock mass; distinct element method; estimated vibration waveform, dynamic analysis.

STUDY ON SPLIT-SECOND DELAY TIME OF PARALLEL CUT BLASTING IN ROCK DRIFTING

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In tunneling excavation engineering, the cut blasting is a main factor affecting the utilization ratio of the holes. The parallel cut blasting method with the large diameter empty hole is now widely used in tunneling blasting because of the application of the large-sized equipment. The cutting provides the free cavity for post-blasting. After blasting the rock in the cut area is broken, meanwhile, the broken rock fragments are mixed with the explosion gases into a two-phase fluid that is thrown out from the cut cavity, the time is needed in this process. If the split-second delay time between the cut blasting and post-blasting is so shorter that the broken rock pieces are partly thrown out, the blasting efficiency is bad. The fewer the rock fragments remain in the cut, the better the blasting result is. In this paper, the mechanical model of initiation delay time for parallel cut blasting is set up by analysis of rock fragments movement in the cut cavity, and the model is available to optimize the initiation delay time. The model is available to find out the quantitative relationship of clearage rate (the volume ratio of the remained rock fragments in the cut cavity) and time, and to optimize the initiation delay time between the crumbling holes and the cut-borehole. The numerical calculation shows that the interval delay time between the cut-boreholes and the crumbling holes should be taken in 17～25*ms* per meter of the borehole depth according to the rock quality. In engineering applications, the split-second delay time of parallel cut blasting in rock drifting should be taken according to the rock quality that when the depth of cut borehole is 4 *m*, the delay time is 70～100 *ms*, respectively, the depth 3*m*, the time 50～80*ms*, the depth 2 *m*, the time 30～50*ms*.

Keywords: Tunneling excavation engineering; parallel cut blasting; split-second delay time; mechanical model; large diameter empty-hole.

SEISMIC ANALYSIS OF TUNNELS - THE QUASI-STATIC METHOD

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A quasi-static method for the analysis of underground structures submitted to section distortion due to seismic shear waves is presented. The method consists of the determination of the peak ground shear stress at structure (tunnel or cavern) level, based on free-field ground profile analysis and application of that shear stress at ground surface. This imposes a uniform stress state in the ground, forcing the tunnel to deform.

Lining bending moments and thrust forces of a shallow circular tunnel in a terrain with varying deformability module are determined and compared with full dynamical analysis and available analytical closed form results. A good match between both simplified and the dynamic analysis is attained. Some useful conclusion are derived from the results

This method has been applied both in 2D and 3D structures with good results, avoiding the employ of time-consuming 3D dynamical analysis.

Keywords: Tunnel; seismic analysis; soil-structure interaction.

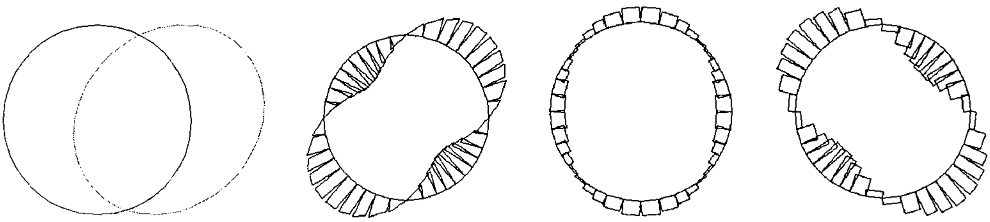

Figure 1. From left to right: deformed configuration; bending moment; shear force and thrust force on lining due to distortion of ground imposed by applied shear stress on free surface.

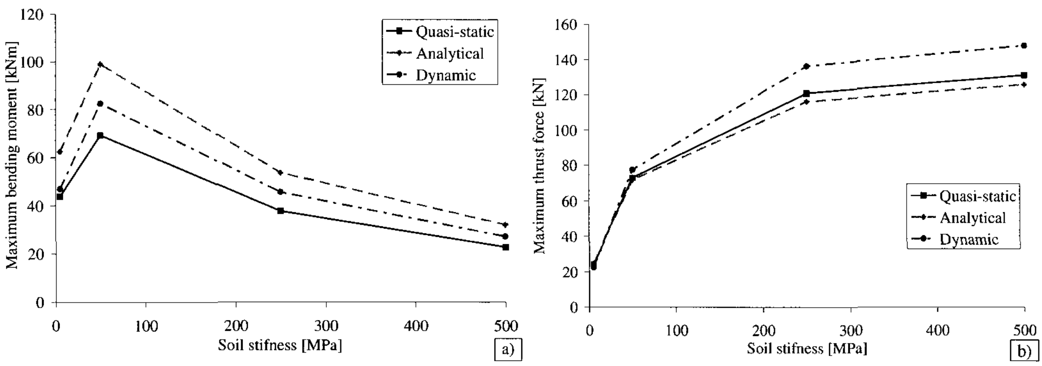

Figure 2. Evolution of maximum bending moment (a) and thrust force (b) with ground stiffness.

LONG HOLE BLASTING EXCAVATION IN SINGLE-TRACK RAILWAY TUNNELING

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The Kawarayu Tunnel on the JR Agatsuma line is a single-track railway tunnel, 1,870 m long with excavation sectional area of $28.8m^2$. To increase the excavation speed, long hole blasting was used. With this technique, the advance per blasting operation was planned to be about 3 m in the section of consolidated andesite. The side wall after the blasting appeared more smooth as the andesite became more consolidated with the progress of excavation by means of the long hole blasting (Photo 1). As a result, advance of 2.7 m per blasting operation was maintained, and the monthly advance exceeded the planned rate (Figure 1).

Keywords: Blast; tunnel; excavation face.

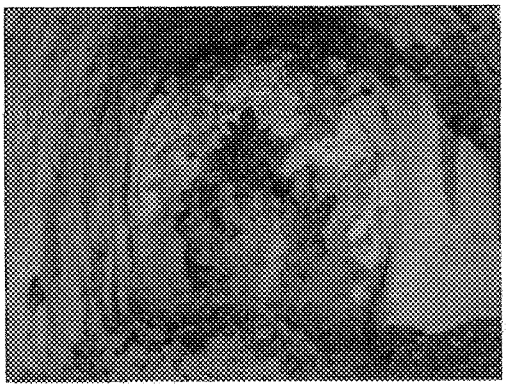

Photo 1. Face after test blast of center cut.

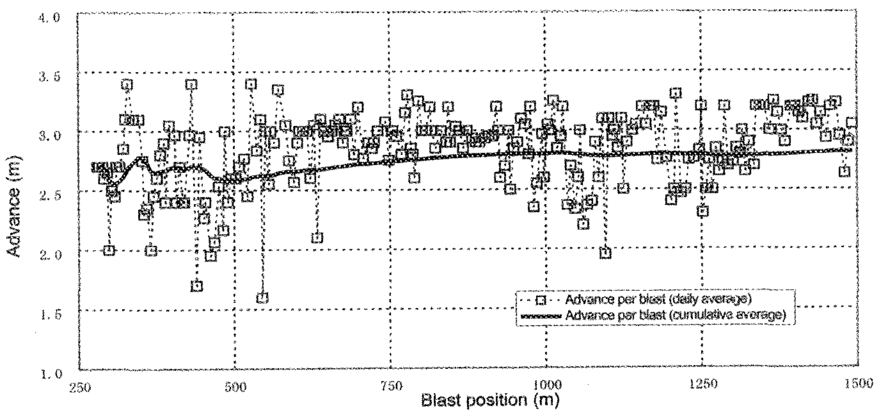

Figure 1. Advance per blast.

BALLISTIC PENETRATION AND PERFORATION OF GRANITE TARGET PLATES BY HARD PROJECTILES

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Sets of ballistic penetration and perforation experiments with 0.6m x 0.6m x 0.1m thick granite targets and 0.2kg, 20mm diameter hard projectiles were conducted. The projectiles were machined from Arne tool steel and oil hardened to a nominal Rockwell C value of 53. A compressed gas gun was used to launch the projectiles into the granite targets which have an average unconfined compressive strength of 163MPa. Projectiles of two different nose shapes were considered: flat-nose and ogive-nose of 3.0 caliber-radius-head (CRH). The effect of confinement on the response of the granite targets impacted by the ogive-nose projectiles was also studied. For every set of experiments, high-speed digital images were taken. The striking velocities were measured and the debris and damage on the granite targets were documented. Based on the dynamic spherical cavity expansion theory, a three-stage analytical model, as shown in Fig. 1, was proposed to predict the outcome of the ballistic penetration and perforation experiments. Predictions made with the analytical model are in good agreement with the recorded data.

Keywords: Penetration, perforation, granite target plates, hard projectiles, striking velocities, ballistic limit.

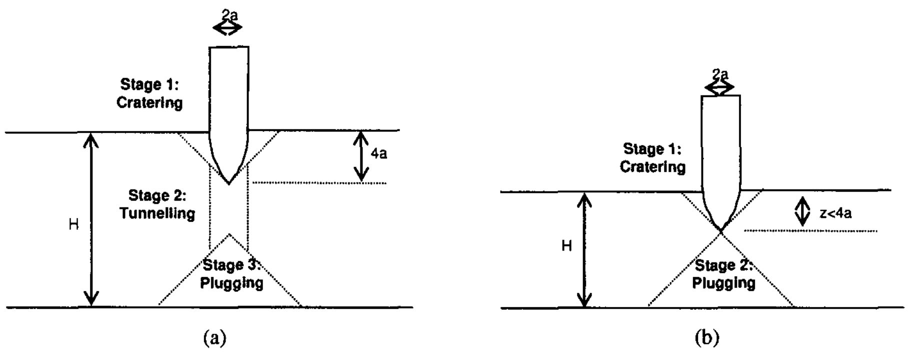

Fig. 1. Penetration and perforation model (a) with and (b) without tunneling.

OPTIMIZED BLASTING DESIGN FOR LARGE-SCALED QUARRYING BASED ON A 3D SPATIAL DISTRIBUTION OF ROCK FACTOR

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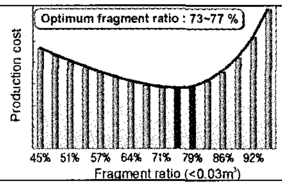
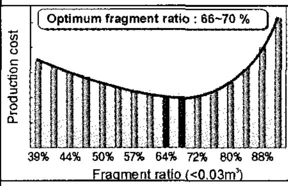
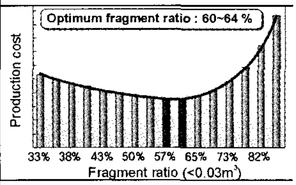
Rock fragmentation plays a critical role in large-scale quarrying operations because of its direct effects on the costs of drilling, blasting, secondary blasting and crushing. In this aspect, it is essential to consider rock fragmentation in blasting design. The optimum blasting pattern to excavate a quarry efficiently and economically can be determined based on the minimum production cost which is generally estimated according to rock fragmentation. By comparing various prediction models, it can be ascertained that the result obtained from Kuz-Ram model relatively coincides with that of field measurements. Kuz-Ram model uses the concept of rock factor to signify conditions of rock mass such as block size, rock jointing, strength and others. Rock factor is estimated from the geologic data such as block size of rock mass, rock jointing, strength, and others, and 3-D spatial distribution of it was predicted by a sequential indicator simulation (SIS) technique. The entire quarry was classified into three types of rock mass and optimum blasting pattern was proposed for each type based on 3-D spatial distribution of rock factor. In addition, the staged blasting plan was established from the analysis of blasting influence zone around the quarry for the purpose of improving constructability and economical efficiency.

The ratios of each blasting type are 19%, 57%, and 24% for TYPE-I~III, respectively. Blasting patterns for each blasting type were designed from the rock fragmentation analysis results, and the ratios of below fragment size 0.03 m^3 in each blasting pattern met the ranges of fragment ratio for achieving optimum productivity. The optimum patterns for each blasting type were designed from the analysis of rock fragment size and total production cost, and the 4 staged blasting plan was established for constructability and economical efficiency by the evaluation of blasting influence zone around the quarry. The vibration reduction methods were adopted in the blasting plan. The vibration-controlled blasting was necessary partly in western area at STAGE-1~2. The rock splitting and vibration-controlled blasting for minimizing ground vibration were applied near a sewage plant at STAGE-3.

Keywords: Rock fragmentation, Kuz-Ram model, rock factor, SIS, blasting influence zone.

Table 1. The optimum fragment ratio (<0.03 m^3) based on minimum production cost for TYPE-I, II and III.

<table>
<tr><th>Division</th><th>TYPE-I</th><th>TYPE-II</th><th>TYPE-III</th></tr>
<tr><td>Production cost-Fragment ratio</td><td></td><td></td><td></td></tr>
<tr><td>Optimum fragment ratio</td><td>73~77 %</td><td>66~70 %</td><td>60~64 %</td></tr>
</table>

PREDICTION OF FRAGMENTATION ZONE INDUCED BY BLASTING IN ROCK

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It is essential to predict a blasting-induced fragmentation zone beyond the proposed excavation line of a tunnel because the unwanted damage area requires extra support system for tunnel safety. Complicated blasting process which may hinder a proper characterization of the damage area can be effectively represented by two loading mechanisms. The one is a dynamic impulsive load generating stress waves outwards immediately after detonation. This load creates a crushed annulus and cracks around blasthole. The other is a gas pressure that remains for a relatively long time. Since the gas pressure reopens up the arrested cracks and continues to extend some cracks, it contributes to the final formation of the fragmentation zone induced by blasting.

This paper presents the simple method to evaluate the fragmentation zone induced by gas pressure during blasting in rock. The fragmentation zone is characterized by analyzing crack propagation from the blasthole. To do this, a model of the blasthole with a number of radial cracks of equal length in an infinite elastic plane is considered (Figure 1). In this model, the crack propagation is simulated by using two conditions only, the crack propagation criterion and the mass conservation of the gas where the adiabatic condition is coupled. As a result, the stress intensity factor of the crack generally decreases as crack propagates from the blasthole so that the length of the crack is determined. Gas penetration into the cracks significantly affects the extension of fragmentation zone in blasting in rock so that this factor is closely related to the blasting efficiency. Longer cracks are created in rocks with weaker properties. The number of cracks around blasthole has a little effect on the fragmentation formation because of the compensation by gas pressure. Gas pressure inside blasthole continues to decrease during crack propagation.

Keywords: Blasting, Crack Propagation, Fragmentation zone, Gas Pressure, Stress Intensity Factor.

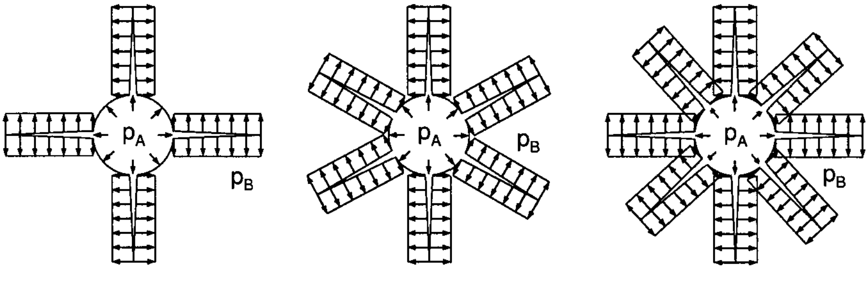

Fig. 1. Pressurized blasthole and propagating radial cracks (N indicates the number of cracks.).

STABILITY ASSESSMENT OF 290 LEVEL CAVE SUBJECTED TO BLAST-INDUCED VIBRATIONS

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Nanfen open-pit mine in Liaoning is one of the biggest iron ore in Benxi Iron Company, China. The 290 level cave is a major transport laneway in Nanfen open-pit. The base line of the pit bottom has reached 346m level in Nanfen surface mine. The importance of in situ assessment of stability of the 290 level cave for next mining sidestep (346-322m) design and construction has been met. Investigation in the Nanfen Surface Mine has shown that damage exists around 290 level cave and that the damage develops from the energy imparted to the rock by the blasting method and by redistribution of the in situ stress field around the 290 level cave.

Blast design parameters and blast performance which are considerably affected by 290 level cave in Nanfen open-pit Mine, were investigated, especially how the construction process will affect the stability of 290 level cave. It is concluded that the Nanfen mine contains structural elements and distinct textural varieties which differ significantly in their response to rock properties testing and to excavation. Data collected in this study demonstrate the highly inhomogeneity internal structure characteristic of apparently homogenous batholiths, and the influence of this inhomogentiy on rock mass response. Some measures are proposed to take so as to play fully the role of host rock in construction process. The practice showed that the engineering conditions of rock in Nanfen open-pit mine are very complex, and it is of importance to monitor rock's structural plane and deformation and to readjust timely the construction schedule and relevant technologies. It is stressed that the self-stability mechanism of key positions should be monitored and controlled carefully so as to ply fully the role of self-stability of host rock. What the measures proposed to take and relevant experience will benefit the construction of the other similar projects. Based on the engineering characters and rock mechanics, the main characteristics of stress induced brittle failure of the site are introduced during blasting construction. Various evaluation and measures are sought to stabilize 290 level cave. The results presented in this paper highlight how to optimize blast design parameters, and to assess the stability of 290 level cave in Nanfen open-pit mine subjected to blast-induced vibrations. It is to be expected that similar results will occur in same rock engineering also.

Keywords: Blast Damage; Optimization; Controlled Blasting; Mechanics Behavior, Stability of Surrounding Rock, Stress Field.

MODELING DYNAMIC SPLIT FAILURE OF GRANITE USING SMOOTHED PARTICLE HYDRODYNAMIC METHOD

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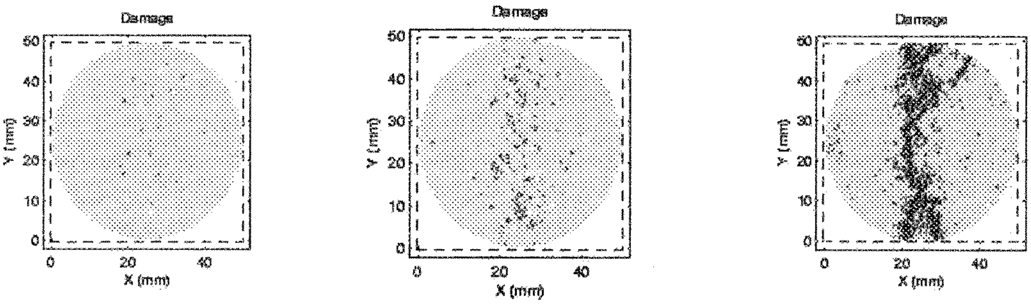
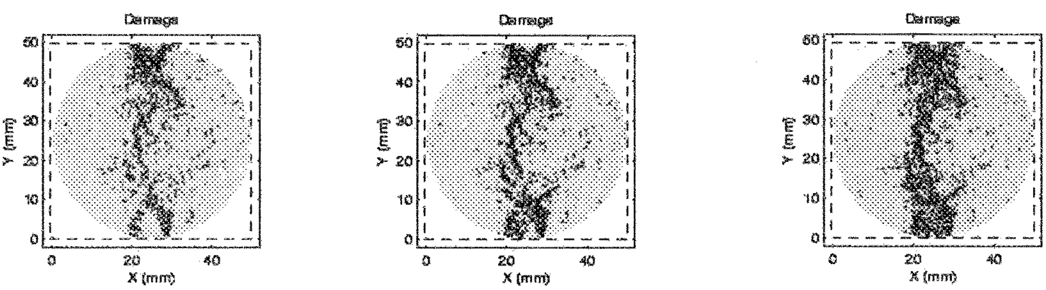
As the earliest mesh-free method, the smoothed particle hydrodynamics method (SPH) has been used to simulate the fracture and fragmentation process on brittle solids due to its distinct advantages in processing discontinuous problems over the conventional methods such as FEM since 1990s. In this paper, a program based on the SPH method was developed to simulate the dynamic split fracture process of granite. An elasto-plastic model coupled with the isotropic scalar tensile damage model depending on the equivalent tensile strain was developed to model the dynamic failure of brittle materials, such as rock and rock-like materials. A statistical approach based on the Weibull distribution law was adopted to model the material heterogeneity and investigate its effects on the mechanical response and the fracture process. A series of numerical simulations on the Brazilian splitting test for the granite sample were performed using the developed program. The dependency of the material dynamic properties on the material heterogeneity was investigated. Differences of the fracture process for these cases were compared and analyzed. Results showed the developed program was very suitable to model these dynamic split failure processes.

Keywords: Mesh-Free Method, SPH, Ealsto-Plasticity, Tensile Failure, Rock Fracture, Heterogeneity.



step 1801 step 2601 step 12401

Fig. 1. Rock fracture process in Case 1 under loading Case A.



step 1801 step 2601 step 12401

Fig. 2. Rock fracture process in Case 2 under loading Case B.

BLASTING VIBRATION EFFECT IN EXCAVATING A MULTI-ARCH TUNNEL

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At present blasting is still widely applied in tunnel excavation, the harmful effect of blasting vibration exists inevitably. The Dongkoutang tunnel in Hunan Province , China, was a multi-arch one and was excavated by blasting. The blasting vibration effect on the stability of the surrounding rock and tunnel structure and on the safety of buildings above the tunnel was concerned. In the tunnel construction, the left hole was excavated after the right hole had been completed. The excavation blasting of the left hole might affect the right hole. By using the dynamic FEM calculation program and by measuring the particle vibration velocity of the surface-ground and the central wall of the tunnel, the characters of the blasting seismic waves were studied.

In simulation, three types of surrounding rock were taken into account and the effect of the different diving sequence of the left and right cave was also considered. The results of calculation indicated that the Young's modulus played an important role in the blasting vibration question of the tunnel. The enhancement of elastic modulus increased the spreading speed of the seismic wave; however, it reduced the Peak Particle Velocity. The results of calculation showed that the arch crown and nearby regions of the tunnel were the dangerous area, the stress concentration appeared at the above area, where the PPV may exceed 10 cm/s and the vibration time was longer than that of other area. During blasting the left hole, the pieces of sprayed concrete and broken stone of the right hole fell from the above regions occasionally, which indicated that the value of PPV was large enough and showed that the simulated results corresponded to objective reality.

Numerical simulation of the blasting vibration was conducted during the right cave was excavated. The Peak Particle Velocity (PPV) of the surface around the buildings was 3.28 cm/s in horizontal direction and 5.05 cm/s in vertical direction, which exceeded the limit of 2 cm/s to brickwork according to the Chinese Safety Regulations For Blasting. So the houses above the tunnel had to be removed before blasting excavation. The results of the vibration monitoring of the above-mentioned site were 2.7 cm/s in horizontal direction and 5.1 cm/s in vertical direction, which showed the calculation was correct. And in the simulation, the importance of the delay interval in the blasting vibration effect was also studied. The PPV value would be decreased largely, if the delay interval was 100 ms or greater.

Keywords: Multi-arch tunnel; blasting vibration; peak particle velocity; dynamic FEM.

STUDY ON BLAST-INDUCED DAMAGE IN BEDROCK EXCAVATION

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ThisDrill and Blast is a main method for excavations of bedrock in engineering such as hydraulic and nuclear power station project. During blasting excavating, ground vibration caused by explosion will result in damage to surrounding rock. Study on size and extent of rock damage is of importance to ensure the integrity of foundation bedrock. For this case, four groups of sonic wave tests have been conducted in surrounding rock during blast excavating in Lingdong Nuclear Power Station (LNPS) project, Guangdong, China. Sonic wave speed was measured at the same point along the depth of testing holes before and after blasting. Change rate of sonic wave speed can thus be obtained and analyzed. By the theory of elastic waves and the assumption that the density and Poison's ratio of damaged rock are approximately equal to those of undamaged, a damage threshold of D_{cri}=0.19 was presented on condition that critical change rate of sonic wave speed is prescribed as η=10%. According to this relationship the damage zone size of each explosion test has been achieved (Fig. 1). It's found that surrounding rock in the middle part of charge column is most badly damaged and with the depth increasing from the bottom of charge column, damage variable rapidly reduces to a negligible quantity. In addition, in the area adjacent to ground, the degree of rock damage is less. At the same depth, the damage variable decreases rapidly with the horizontal distance from charge hole. On the other hand, damage variable decreases very fast with the depth. It can be concluded that, in column charge explosions, damage variable attenuates more rapidly in vertical direction than that in horizontal direction. Damage zone radius of rock is approximately 3 times of damage zone depth.

Keywords: Damage zone; nuclear power station; sound wave test; rock blasting.

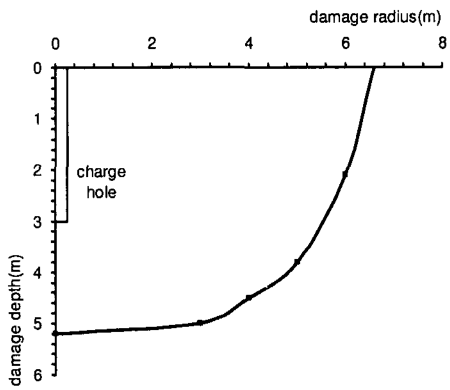

Fig. 1. Damage contour of rock mass under blasting.

DYNAMIC UNIAXIAL COMPRESSION TESTS ON A CEMENT MORTAR

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Rock and cement mortar (rock-like material) are brittle and heterogeneous materials. They are widely used as civil engineering construction materials and as host materials for tunnels and underground structures. The dynamic properties of rock and rock-like materials are different from their normal static properties, and are the basic information in assessing the stability of rock structures under dynamic loads.

Researches on the rate effect have been conducted primarily through experiments, especially under uniaxial compression loads. This paper reports a series of dynamic uniaxial compression tests on a cement mortar. A total of 17 uniaxial compression tests at five different loading rates (10^{-1}, 10^{1}, 10^{2}, 10^{3} 10^{4} MPa/s) were conducted. The tests at very low loading rates of the magnitude of 10^{-1} MPa/s can be treated as quasi-static tests, which are equivalent to the standard for quasi-static rock deformation tests specified in the ISRM Suggested Methods for Rock Characterization, Testing and Monitoring (1981). When the loading rates are over 10^{0} MPa/s, the tests are considered as dynamic tests.

The experimental results indicate that there is no significant increase in the Young's modulus and the Poisson's ratio of the cement mortar with the loading rate. Whereas, the compressive strength increases from 33.5 to 44.0 MPa (approximately 30% increment) corresponding to the increase of loading rate from 10^{-1} (a quasi-static loading rate) to 10^{4} MPa/s (a high loading rate). The relationship between the uniaxial compressive strength and the loading rate can be expressed as:

$$\sigma_{dc} = 1.76\log(\dot{\sigma}_{dc}/\dot{\sigma}_{sc}) + 34.3 \tag{1}$$

where $\dot{\sigma}_{dc}$ is the dynamic loading rate; $\dot{\sigma}_{sc}$ is the quasi-static loading rate, 34.3 (MPa) is the uniaxial compressive strength of the cement mortar at quasi-static loading rate 1.76 is a constant for the material. This conclusion is in consistency with the findings obtained by Zhao et al. (1999) from similar tests on cylindrical rock specimens (Bukit Timah granite) with sizes of 30 mm in diameter and 60 mm in length.

Keywords: Dynamic uniaxial compression tests; cement mortar; loading-rate effect.

ROCK RESPONSE UNDER BLAST LOAD BY THE DISCONTINUOUS DEFORMATION ANALYSIS

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In order to evaluate the response of rock mass under blast loads, the discontinuous deformation analysis (DDA) is used as the numerical tool to carry out the numerical simulation. In this paper, the blast effect on the rock mass is studied. The numerical results show that the DDA method is able to provide some valuable predictions for blast effect in the rock medium. This preliminary study forms a foundation for future analysis on coupled system of buried structure under blast load.

The following figures show the rock mass response under a buried explosive detonation by the DDA simulation.

Keywords: Blast; deformation; discontinuity.

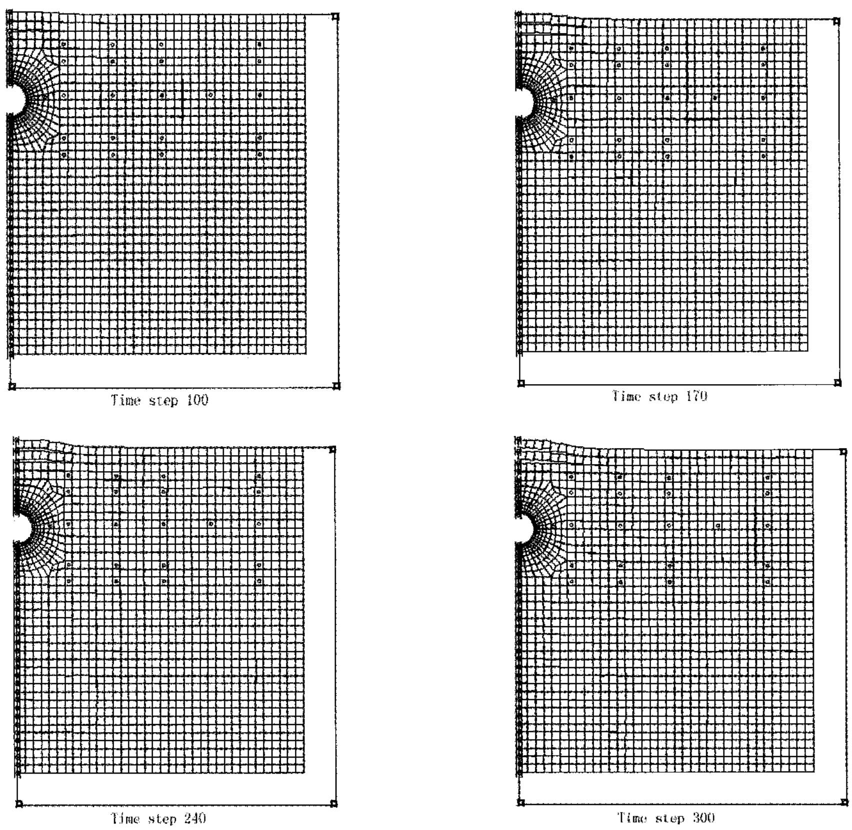

Fig. 1. Block deformation at various time steps.

DYNAMIC SHEAR PROPERTIES OF BRITTLE MATERIALS SUBJECTED TO CYCLIC LOADING

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Shear testing of brittle materials (modeled by mortar) under static and cyclic loading is studied in the present paper. Following the description of shear equipment in the Vibration laboratory of Hohai University which is based on MTS and allows the shear tests to be carried out under cyclic loading conditions including frequency of 1Hz, 2Hz, 3Hz and 5Hz, the test phenomenon is described in detail and the results of each kinds of tests are given, with attention being paid to the difference of the mechanical behavior between static and dynamic tests.

The objective is to find changing discipline of the key strength including peak strength, residual strength and the strain versus the shearing frequent applied during the tests. It is shown that:

(1) Shear position was in the middle of the specimen and only one shear surface appeared just as we planed in advance whether under static conditions or under dynamic conditions.

(2) Normal displacement and normal loading decreased slightly at first and then it increased constantly until the specimen failed. After the failure point, the displacement would decrease appreciably and keep a certain value at last

(3) Shear stress-displacement curves under static tests is similar to the strength envelope curves under cyclic tests. Both of them can be divided into four parts: elastic stage, elastic-plastic stage, failure stage and slide stage. A simple constitutive equation can be used to model the static stress-displacement curve and cyclic strength envelope curve.

(4) Under cyclic tests, the dynamic peak strength is greater than the corresponding static value. According to Mohr criterion, the peak strength could be defined by two factors: cohesion c and the angle of internal friction φ. For the first group of specimen, the dynamic value of c was almost double the static value and φ increased nearly 30%. The two factors also increased greatly for the second group of specimens. While the peak strength varies inconspicuously with the frequency changing.

(5) The residual strength varies in the same manner as peak strength under static and cyclic loading. The strength value will increase with the frequency increasing but the changing trend is slightly and inconspicuously.

(6) The dynamic to static peak strength ratio has a tendency to decrease as the static shear strength increases or normal loading increases.

(7) The strain according to the peak strength increases with the peak strength increasing.

Some opinions about shear failure mechanism and difference between static and dynamic conditions are given at last.

Keywords: Cyclic shear test; brittle materials; shear property; MTS.

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5. SUPPORT AND REINFORCEMENT

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DESIGN OF ROCK SUPPORT SYSTEM FOR SUB-SEA DOCK EXCAVATION

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Aker Kværner has commissioned the construction of a new dock at their naval yard in Egersund on the south-western coast of Norway to the contractor AF Spesialprosjekt.

The new dock will have a length of 100 m, a width of 40 m and an excavated depth of 7 m below mean sea level, with vertical rock walls. The docksides are designed to support very heavy loads (up to 1000 kN/m) from cranes used for assembly of drilling platforms that will be constructed within the dock.

The bedrock at the site is anorthosite, an igneous rock with a high content of feldspar. The rock is jointed, and a geological survey of the area revealed that for all the three sides of the dock, joints with strike parallel to the excavation line and with a dip of 45° – 70° towards the excavation must be expected, with inherent risk of instability and failure of the dock walls during excavation.

The contractor AF Spesialprosjekt commissioned Norconsult AS to undertake the overall design of the dock, including design of a rock reinforcement system that would ensure the integrity of the dock walls both during excavation and under full load conditions during subsequent operation.

A major restriction was that the reinforcement system had to be installed prior to start of the excavation work and had to be installed from the surface above sea level.

The solution was a system of fans of rock dowels installed in boreholes drilled from a position 50 cm outside the excavation contour, with dowels slanting 80°, 65° and 45° from the horizontal into the dock wall.

The dowels were 32 mm rebar dowels installed fully grouted in the boreholes.

Fans of 3 rebar dowels were installed every 1 m along the dock walls. The dowels were installed to a depth of 1,5 m beyond a design surface oriented parallel to the dock wall with a dip of 45° towards the excavation and cutting the foot of the wall. Tests performed at the site indicated a friction angle of 45° for the dominating joint sets justifying the above assumption.

The verification of the stability of the dock walls was based on Norwegian guidelines for rock stability calculations, common practice within rock mechanics and certain evaluations based on interaction behaviour in the sliding planes. Utilization of the dowels with regard to the overall stability and sliding were calculated based on typical dowel theory considering the rebar dowels crossing the joints and taking into account the corresponding cohesion and friction effects within the jointed surfaces.

In total 6 anticipated cross sections along the crane track were studied utilizing two-dimensional calculation models. The final results were presented as safety factors representative for the overall stability under design load conditions.

Keywords: Design; rock support; excavation.

NON-DESTRUCTIVE EVALUATION SYSTEM OF THE TUNNEL CONCRETE LINING USING WAVELET TRANSFORM ANALYSIS AND NEW ACOUSTIC TAPPING MEASUREMENT

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There are a number of aged railway tunnels constructed of concrete lining in Japan. These tunnels have some problems in their structural and material integrity. Under such a situation, an accident such as concrete spalling occurred in a Shinkansen tunnel in 1999. An effective non-destructive test method for such problems is hence expected. A hammering test is carried out at the first stage of the tunnel inspection. Though it is very easy to make the test, the results are qualitative and the reproducibility is scarce.

Then we have developed a new system which improves those problems. The system is mainly composed of three elements; a tapping method, a measuring technique of sound, and an analysis method of sound signal. The tapping method uses a steel ball shooting tool driven by some elastic rubber cords. Its motion looks like a slingshot. This shooting tool generates useful high-frequency sound waves, and suppresses the associated noise most in the tools on which we have tested. As the measuring technique of sound, we use a microphone with a special noise-proofing hood that is pressed against the lining. The hood has a cork cylinder which enables to catch without so distorting the characteristic of sound. The sound waves are converted into digital data on the computer. As the analysis method of sound signal, we use the wavelet transform analysis. The transform data contains meaningful information about the concrete lining.

By using the evaluation system, we can approximately estimate several state parameters of the lining concrete, such as thickness of the lining and position of defects like cavities in and behind the lining.

Keywords: Non-destructive testing; Tunnel concrete lining; Acoustic tapping; Microphone with a noise-proofing hood.

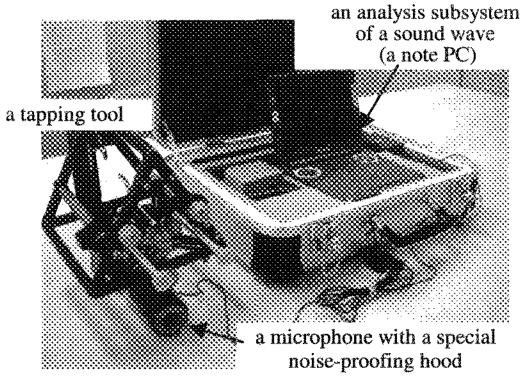

Fig. 1. Non-destructive Evaluation System of the Tunnel Concrete Lining.

BENEFITS AND COMPARISONS OF PRE-REINFORCEMENT APPLICATIONS IN TUNNELLING

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The construction of tunnels under complex conditions, such as loose unconsolidated ground, fault zones and areas of shallow overburden in urban regions is a challenging problem which often leads to ground subsurface settlements and collapses.

Pre-reinforcing ahead of the tunnel face is an upcoming technique to improve the rock mass behaviour before the actual advance takes place.

This paper describes the most striking advantages of pre-reinforcement in general and shows a comparison between spiling with Self-Drilling-Anchors and common pipe-roofing.

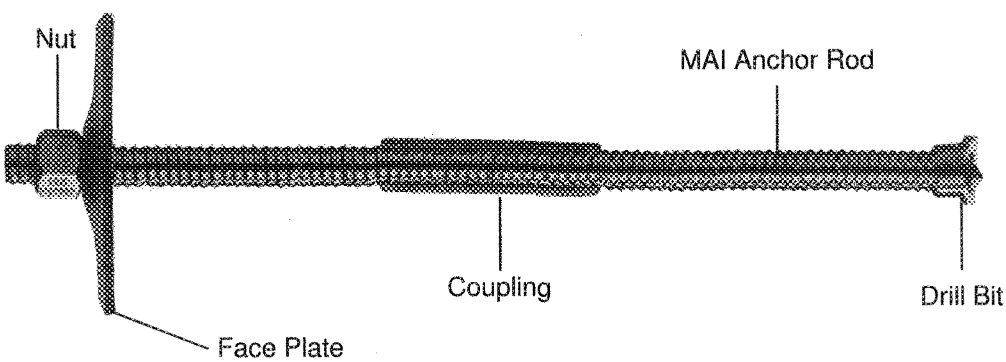
The two methods will be compared in the face of the construction costs, the advance speed, the construction time and the influence on the rock mass behaviour.

Pre-reinforcement enable the load redistribution in the unsupported excavation area without failure in accordance to the fundamentals of the New Austrian Tunnelling Method.

Summing up, the most striking advantage of spiling with Self Drilling Anchor as ahead support is the higher action flexibility to frequently changing rock mass conditions during construction in terms of the variation possibilities of the lengths, the capacity (diameter of the rods) and the amount (pattern) of the spiles (anchors) from one round to the next which effects on costs and construction time.

The increased use of pre-reinforcement in tunnelling calls for design rules which are based on the support characteristic of pre-support systems.

Keywords: Pre-reinforcement; Pre-support; Self Drilling Anchor; Pipe-roofing; forepoling; spiling.



Components of the MAI-Self Drilling Anchor (SDA) System

NUMERICAL ANALYSIS FOR BETTER UNDERSTANDING MECHANISM OF SUPPORT EFFECT ON GROUND STABILITY BY USING DISTINCT ELEMENT METHOD

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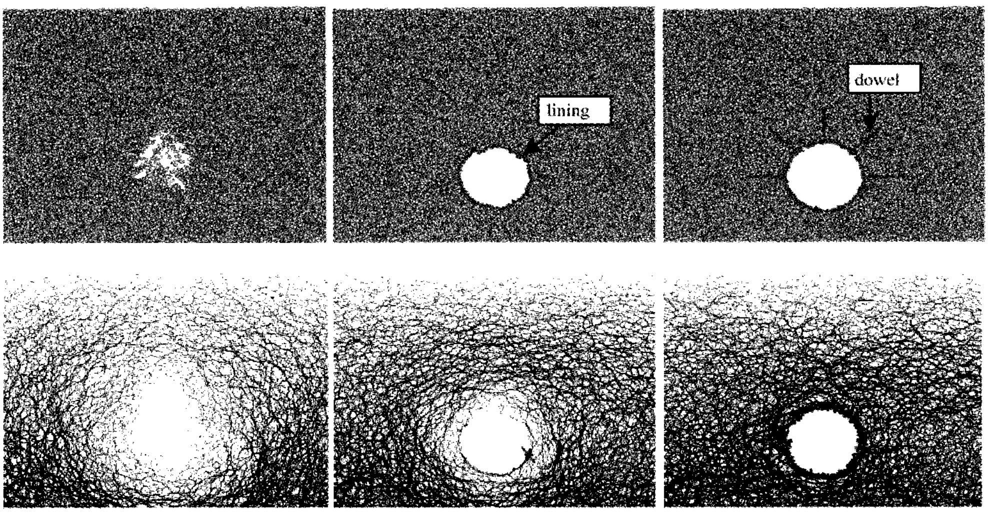
When a tunnel is excavated in a shallow ground, a loss of the stability of the tunnel causes settlement of the ground surface. To remain the ground stability, it is important to choose proper supports or reinforcements. However, effects of the supports and reinforcements on the ground stability are not enough clear to establish systematic methods to choose the proper supports. Thus, a number of numerical simulations of the tunnel excavation have been carried out in order to clarify how supports and reinforcements affect the stability of the ground around the tunnel.

In general, analytical methods based on a continuum mechanics are normally used for analyses of the ground stability. However, it is difficult for the continuum analyses to simulate failure such as separation of material or slip. These phenomena are very important to discuss the ground stability. Therefore, we adopted the distinct element method (DEM) to perform the simulations.

Figure 1 shows distributions of particles and contact force after a tunnel is excavated in the shallow ground. As shown in Figure1, the tunnel will collapse unless supports are installed. Figure 1 also shows that the excavation of the tunnel reduces the contact force around the unlined and the lined tunnels, while, for the tunnel supported by both a lining and dowels, the contact force is concentrated around the tunnel. This concentration of the contact force may be regard as a ground arch.

Keywords: DEM; support; ground stability; tunnel.



(a) Unlined tunnel (b) Supported by lining (c) Supported by lining and dowels

Fig. 1. Distributions of particles (top) and contact force (bottom).

FULL-COLUMN GROUTED ROCK BOLTS AND SUPPORT PRESSURE

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Rock bolts and shotcrete is a very popular support system in almost all the tunnelling and underground projects. The support system is designed to take care of the expected pressure. The support pressure being shared by the rock bolt is an important issue, which also helps in deciding the thickness of the final shotcrete/concrete lining for balance of the support pressure.

It is known that pre-grouting improves the rock mass quality. Similarly, full-column grouted rock bolts help in two ways, (i) it reinforces the rock mass around the underground opening creating a reinforced rock rib and (ii) the grout percolates through the cracks and fissures and thus improves the rock mass around the opening and thereby reducing the ultimate support pressure. It may be highlighted here that the improvement in the rock mass because of full-column rock bolt grouting is found to be more in poor rocks and it may be practically negligible in good rocks. In very good to exceptionally good rocks (represented by rock mass quality Q value from 40 to 1000), on the other hand, the joints are tight and there shall not be further scope of improvement in rock mass quality because of rock bolt grouting.

Table 1. Original and modified value of rock mass quality for three rock classes.

	Poor Rocks	**Fair Rocks**	**Good Rocks**
Original Q Value	1.0 to 4.0	4.0 to 10.0	10.0 to 40.0
Modified Q (Q_{mod}) value after rock bolting	3.0 to 10.0	10.0 to 20.0	20.0 to 40.0

A study has been carried out to find out the improvement in Q to obtain the modified rock mass quality Q_{mod} because of the grout of full-column grouted rock bolt in case of poor to good rocks represented by Q value between 1.0 and 40.0. The expected change in the rating of various parameters of Q-system has been suggested in the paper. Incorporating the suggested changes, the modified rock mass quality Q_{mod} is given in Table 1. The Q_{mod} is found to be up to 3 times more than the original Q for poor rocks (Q = 1), whereas in good rocks (Q = 40) the Q and Q_{mod} remains the same (Table 1).

In poor rocks, the support pressure obtained using Q_{mod} is less than the support pressure obtained from the original Q. Hence, the final shotcrete thickness would reduce. The study is found to be useful in reducing the supporting cost and time.

Keywords: Pre-grouting; full-column grouted rock bolts; support pressure.

CONCRETE SEGMENTAL LINER INSTRUMENTATION TO QUANTIFY STRESSES INDUCED BY GROUND FREEZING

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Construction of the world's second largest high-grade uranium orebody requires a novel approach to mining methodology and underground development infrastructure. Block ground freezing of the orebody and production level is required due to very poor ground conditions, inflow potential, and hydrostatic pressures at a horizon of 480 metres below surface.

A single shield TBM with a fibre re-enforced concrete segmental lining was used to develop access the lowest level below the orebody. Freeze holes drilled from this elevation would induce liner stresses at the hole collars and upper set of tunnels. Instrumentation was embedded within the segmental liner to estimate the field stresses verses the modelled stresses.

This paper compares modeled liner stresses with initial field results based on concrete segmental instrumentation. The instrumentation design within the liner rings and casting of the instrumentation is also discussed.[2]

The type and quantity of instrumentation cast per ring were two radial pressure cells and ten-strain gages.[2] The instrumented rings were placed in areas of weak rockmass strength, with high permeability, and high water content.[3]

Keywords: Instrumentation; ground freezing; ground support, liner stress; concrete segmental liner, uranium mining.

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TESTING EQUIPMENT FOR ROCK UNDER COUPLING LOADS

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Under the influence of engineering practice or nature effects, such as mining in deep level, earthquake, blasting, stratum sliding etc, the underground rock usually experiences the coupling loads of dynamic stress and the static stress of ground stress. Conventional researches of rock mechanics are mainly about cases that rock is in the condition of only static load or dynamic load. Some preliminary tests on INSTRON servo-controlled material testing machine have shown that rock under coupling loads behaves differently with rock undergoing static or dynamic load separately. However, the maximum strain rate of specimen under coupling loads on INSTRON is less than $10^0\ \varepsilon^{-1}$, which sometimes is regarded as quasi-static load. In order to achieve coupling loads with specimen strain rate larger than $10^0\ \varepsilon^{-1}$, a new equipment has been developed which allows specimen to be subjected to coupling loads of 0-200MPa static load and dynamic load with 10^0-$10^3\ \varepsilon^{-1}$ specimen strain rate. Then the theoretical analysis is carried out to show that the wave equations are still applicable for the new apparatus. A description of the apparatus and the operating methods are also introduced. Finally, the tests of silt rock with various coupling loads are carried out and some test results are given to show the robust of the new equipment.

Keywords: Rock test; load; displacement.

FEM ANALYSIS AND DETECTION FOR STRUCTURAL DAMAGE OF TUNNEL LINING

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Yuyuan Tunnel located in Zhangjiajie City, Hunan Province, China, which is a new tunnel, 2714 m in length. Many cracks appeared on the lining after the tunnel construction had finished. This paper reports mainly on the detection of structural damage of the lining detection and a three-dimensional FEM analysis of a segmental lining embedded cracks to discover the effect of cracks on the mechanical property of the lining.

To improve the capabilities of the structural damage inspection in the tunnel lining, ground penetrating radar (GPR) was employed in this study. Many field measurements in order to detect the distributing of structural damages for the tunnel lining were achieved. The damages of concrete structure were located, and the distribution and depth of damages were inspected and estimated by GPR. The Finite Element Method (FEM) was used to analyze the stress distribution in the damage area (e.g. crack tip) based on the GPR investigations, and the damage evaluation of tunnel structure was discussed.

This study yielded the following findings:

(i) Ground penetrating radar was successfully used to delineate the locations of damages of the tunnel lining. It has been confirmed that GPR is an accurate and efficient method for detecting the tunnel lining damages. The investigations showed that structural damages (e.g. uncompacted concrete and cracks) were occurred in several sections on the arch of tunnel lining, and some ill phenomenon, such as concrete desquamating, pockmarked face and blanch, appeared on the surface of lining. The results of the investigation can give guidance to dispose the cracks and to repair the tunnel lining.

(ii) On the basis of the GPR investigation results,a three-dimensional FEM analysis for structural damage of tunnel lining were carried out. It has been confirmed that the stress concentration occurred at the foot of sidewalls for the integrity lining and the stress concentration occurred at the tip of cracks for the lining with cracks. The effect of the cracks along trend of the tunnel was the most serious on the mechanical property of the tunnel lining. The numerical simulation and computation showed that the cracks in the arch of lining are easier to extend than those in other localities of the lining.

Keywords: Tunnel lining; FEM analysis; nondestructive testing (NDT); structural damage; ground penetrating radar (GPR).

GROUND REACTION CURVE FOR A PHENOMENOLOGICAL DAMAGE MODEL

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Continuum Damage Mechanics has provided great results for numerous materials (metals, polymers, concrete, rocks...) and loadings (monotonic, creep, fatigue and brittle failures). Different degrees of complexity are encountered in usual constitutive modeling, modeling accounting or not for plasticity, dilatancy, induced anisotropy, porosity and/or cracks closure. One focus here on the ability of Marigo simple damage model to deal with underground engineering in rocks mechanics, i.e. here to allow for the calculation of the Ground Reaction Curve. The non-linearities of rock materials are phenomenologically modeled by use of the isotropic damage formalism.

The hodograph method allows to solve the mechanical problem of unloading a circular excavation in a damageable rock mass. A new closed-form solution of the Ground Reaction Curve is then derived for a specific two parameter damage law. A validation of the solution by the semi-analytical transfer matrix method is proposed. The identification procedure for the damage parameters uses a conventional CD triaxial test.

The "Bois de Peu" tunnel, close to Besançon in France, provides a practical application to the present calculations concluding to the possibility to use Damage Mechanics for design in underground work.

Keywords: Damage, convergence-confinement, Ground Reaction Curve, transfer matrix method, hodograph method, non-linearities.

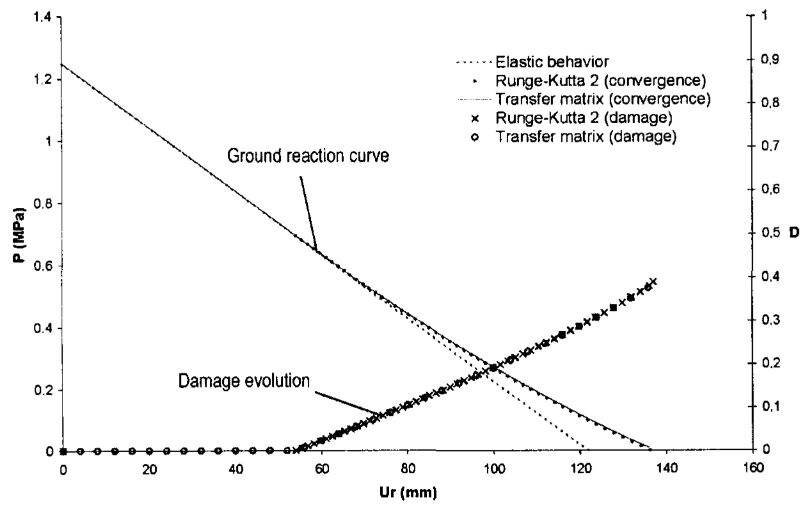

Figure 1. Comparison between the two resolution methods for the validation example.

THE EVALUATION OF THE EFFECT OF LONG FACE BOLTING BY 3D DISTINCT ELEMENT METHOD

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In recent years, continuum analysis approaches, FEM for example, have been widely used for mountain tunnel design and preliminary ground behavior assessment. Continuum approaches have limitations, however, when applied to local behavior such as face collapse. Therefore, application of the discontinuum analysis approach must be considered as well. In this research, the authors simulated tri-axial compression tests, using as parameters the particle size, the void ratio and the arrangement of particles, by applying the distinct element method for granular media. This paper also discusses how the face behavior varies depending on the arrangement of long face bolts by means of the distinct element and continuum analysis methods.

Keywords: Three-dimensional distinct element, tri-axial compression tests, long face bolting.

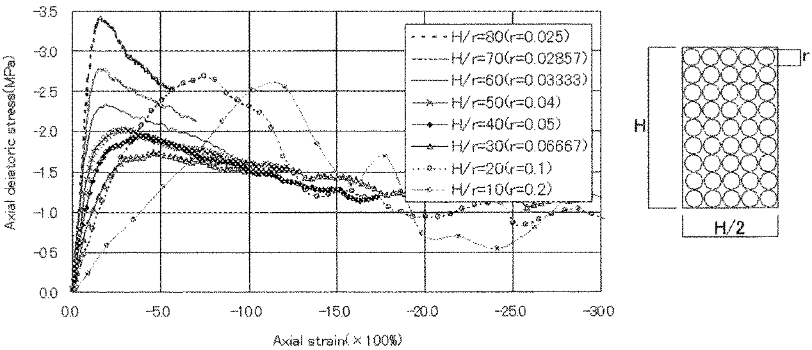
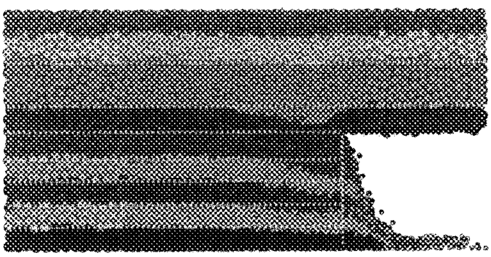
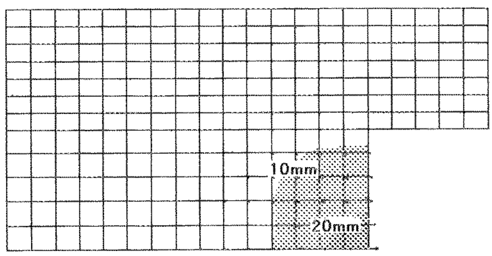


Figure 1. Analysis result(Tri-axial test).



distinct element method



continuum analysis method

Figure 2. Condition of the tunnel face.

MECHANISM AND MEASURES OF COARSE AGGREGATE SPALLING IN AGED TUNNEL CONCRETE LININGS

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A number of defective conditions such as coarse aggregate spalling are visible on cast-in-place concrete tunnel linings covered with accumulated soot emitted by steam locomotives. It is apparent that such defects have mainly occurred at construction joints or honeycombing existed since the beginning of the concrete construction. We conducted a study into the mechanism of such defects; hence, this paper describes the results of the study and countermeasures to cope with such defects.

The findings obtained from the study are as follows:

(1) Coarse aggregates at honeycombing lose adhesive force and spall when the surrounding mortar is eroded by acidic water. This water is produced by acidic oxide from soot on the lining surface dissolving in leakage or condensation. Figure 1 shows the mechanism of coarse aggregate spalling.

(2) Smoke damage actualizes not only in the thinning of brick lining joints but also in the phenomenon of coarse aggregate spalling at honeycombing in cast-in-place concrete linings.

(3) The authors devised a repairing agent using a potential hardening-type material as a countermeasure against coarse aggregate spalling phenomenon.

(4) The authors believe that the repairing agent devised is effective in preventing coarse aggregate spalling.

Keywords: Concrete tunnel lining; smoke damage; acid deterioration; honeycombing; countermeasures.

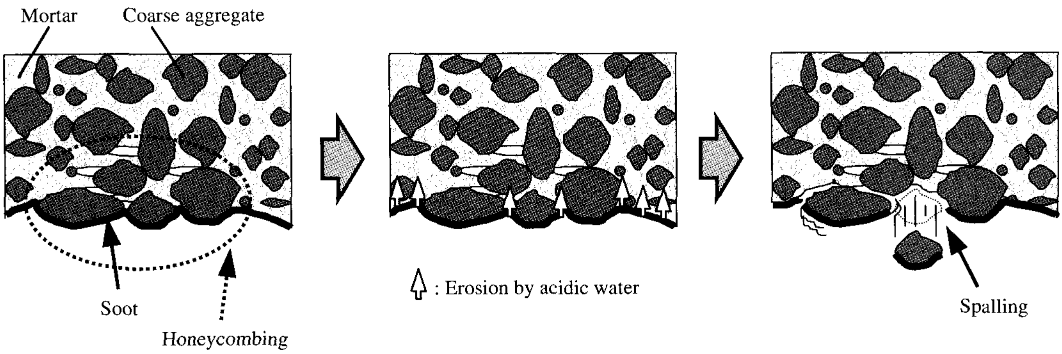

Fig. 1. Mechanism of coarse aggregate spalling (cross-sectional view of honeycombing)

EFFECT OF CONTACT ROOF ZONE ON THE PERFORMANCE OF LONGWALL POWERED SUPPORTS

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Longwall mining has proved its potential of being safe and highly productive for deeper coal deposits. The interaction of powered roof supports with the roof is a key issue in design of longwall panels. Numerical modeling has evolved to be a suitable tool for the study of various aspects of longwall mining system. An attempt has been made in this paper to understand the behavior of the immediate roof vis-à-vis powered roof support system. A case study of two adjacent longwall panels was taken up in an underground coal mine in Kothagudem Area of Singareni Collieries Company Limited, Andra Pradesh, in southern India, with coal roof and stone roof. Three workable seams are occurring within this mine. Of the three coal seams, the top seam is being extracted by mechanized longwall retreating method. Strata above the top seam comprised of massive sandstone. Panel-A was worked in the middle section with coal roof condition at a depth of 200m, with 770m x 150m dimensions. Panel-B was worked in top section of the same seam with stone roof condition at a depth of 215m and its dimensions are 420m x 150m.

A numerical model was developed to find out the distribution of vertical, horizontal and shear stresses at different locations like within the face, above canopy, above unsupported span, at the front legs and rear legs of supports etc. Field studies were also taken up to assess the loading pattern of powered supports during extraction in both the panels, using various instruments. Borehole extensometer with four anchor points was installed in a borehole drilled from surface up to the coal seam in Panel-B with stone roof condition. The borehole was at a distance of 30m from the starting face line along the centre line of panel. This facilitated monitoring of falls, strata behavior inside goaf and the separation of different roof beds. Remote convergence recorders were installed in between the mid face chocks after a face retreat of 12.0m, in order to detect the local falls. Leg pressure surveys were conducted by fixing pressure gauges on each leg of the chock and continuous pressure recorders at strategic chock legs to understand the load transfer onto powered supports. Load cells and convergence recorders were placed in gate roadways to understand the roof behavior.

Modeling studies revealed that vertical stresses are responsible for the failure of immediate roof beds in panel with coal roof. Results have indicated higher vertical stress coming on to front legs during the time of main fall. The coal roof failed regularly due to stress as the face progressed. This phenomenon was confirmed by the results of field monitoring. Overhanging of roof was observed till main fall in the panel with stone roof condition. Under stone roof condition, vertical stress was higher on rear legs. Modeling studies predicted the main fall between 44m-48m face progress with coal roof and 40m for stone roof condition. This was confirmed by the field instrumentation and observations.

Keywords: Coal roof, stone roof, powered supports, numerical modeling, field monitoring, main fall.

COMPARISON BETWEEN NUMERICAL ANALYSES AND ACTUAL TEST IN FIELD FOR PRESTRESS ANCHORS (MONOBARS)

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Siah Bishe Pumped storage power plant with capacity of 1000 MW is the first pumped storage power plant in Iran that located at north of Iran. This power plant has large caverns. These caverns considering geological properties are located in a sedimentary layers and igneous masses. Thickness of sedimentary layers is from some centimeter to about 1 meter.

In above totally about 1200 monobars with capacity of 890 and 300 KN, with length of 8,10,15,20,30 meter and bond length of 4 and 5 meters has been installed in Siah bishe pumped storage power plant (The monobars are from Annahute Company).

In this paper numerical analysis has been done with finite difference method. The behavior and movement of rock behind anchor pads is investigated separately for monobars on field, also movement of anchor pads during stressing have been measured by micrometers. The behavior of rock around bond length has been modeled too.

Results of rock movement behind anchor pad in numerical modeling and actual stressing are very close in comparison. The pull out tests data are compared with numerical analysis and result present in this paper too.

Keywords: Monobar; Anchor; Numerical analysis.

SHEAR REINFORCING EFFECT OF RUST PROOFING EXPANSIVE ROCK BOLTS

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The instantaneous reinforcing effect by the expansive steel pipe rock bolt is expected since its expansion due to application of hydraulic pressure in the steel pipe generates the friction force. However, the corrosion of the steel pipe has been of concern. To overcome the corrosion problem, high corrosion resistance hot dip coating steel plate "ZAM (Zn-6%,Al-3%,Mg)" has been developed recently, RPE (Rust Proofing Expansive) rock bolt has also been developed by using ZAM coated steel plate. In addition, the inferior shear strength than the conventional massive rock bolt has been of concern. In this paper, the corrosion resistance test and the shear test have been performed in a laboratory.

New ZAM (Zn-6%,Al-3%,Mg) plated expansive rock bolt is regarded as fulfilling the function required of rock bolt over the past 20 years (See Photo 1). From the shear test, the shear rigidity of expansive rock bolt has almost the same ability at all type of rock bolt and its shear strength is less than that of the massive rock bolt (See Fig. 1).

Keywords: Rock bolt; shear test; corrosion resistance.

Specimen	Observation position	Before the test	After SST 500hour (WOP is 265h)	After the CCT 60 cycles (WOP is 30c)
Without plating	Non-welded			
	welded			
ZAM plating	Non-welded			
	welded			
Zinc Plating	Non-welded			
	welded			

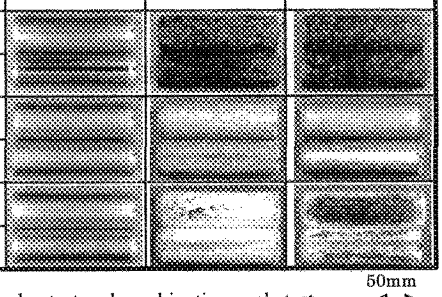

Photo 1. Results of salt splay test and combination cycle test.

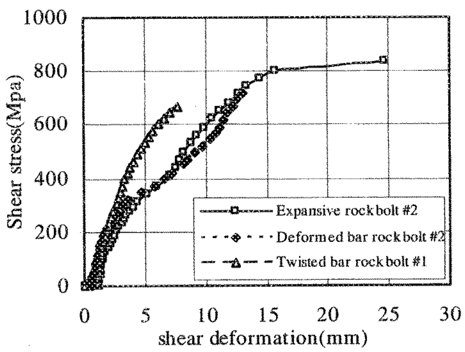

Fig. 1. Shear stress deformation curve results in high strength block.

DETERIORATION MECHANISMS OF TUNNEL LINING CONCRETE

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Railways in Japan started to use concrete for tunnel linings around 1910. To properly control and maintain concrete lining material in the future, it is important to investigate the causes of concrete deterioration and take appropriate countermeasures. The authors of this report surveyed several deteriorated tunnels and discovered the following:

(1) The lining concrete of railway tunnels deteriorated due to chemical reaction with (i) acid and (ii) sulfate ions, as well as (iii) the fact that a substitute material had been used for the cement contained within the concrete (Table 1).

(2) Concrete deteriorates due to the action of acid pulverizing and reducing the strength of concrete. The acid originates from acidic ground water and the smoke emitted by steam locomotives in the past.

(3) Deterioration due to the action of sulfate ions results from the formation of ettringite as the ions concentrate at the carbonation front and cause cracks.

(4) Concrete also deteriorates because diatomaceous earth (a cement substitute material) was used due to a shortage of cement when the tunnels were constructed. Since the compressive strength becomes extremely low in the carbonated areas of such concrete, this carbonation would appear to be the cause of the deterioration.

Keywords: Tunnel; lining concrete; deterioration; acid; sulfate; diatom earth.

Table 1. Deterioration of tunnel lining concrete.

kinds	year of construction	deterioration conditions	cause of deterioration
tunnel A	around 1927	Concrete has weakened through pulverization, and presents a brown layer.	acid originates from the steam locomotive smoke in the past
tunnel B	around 1950	Concrete has weakened through pulverization, and presents a brown layer.	acid originates from the steam locomotive smoke in the past
tunnel C	1923	Concrete has weakened through pulverization, and presents a brown layer.	acid originates from acidic ground water and the steam locomotive smoke in the past
tunnel D	1937	Fragment layer etc. was observed near the surface, and a white substance was present.	the action of sulphate ions
tunnel E	1924	Fragment layer etc. was observed near the surface, and a white substance was present.	the action of sulphate ions
tunnel F	1928	Concrete surface had weakened through pulverization. No brown layer was present.	diatomaceous earth was used due to a shortage of cement

FLOOR HEAVE ROADWAY PRESTRESSED ANCHOR AND INVERTED-ARCH COMBINED SUPPORT DESIGN AND ITS NUMERICAL ANALYSIS

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How to control floor heave effectively and economically is an urgent problem in deep soft rock roadway. In deep mine under -850m level, the failure of pump chamber was severe, especially the floor heaved worse. The former methods of controlling floor heave were summed up. According to research of controlling floor heave methods, the floor heave reasons of pump chamber was analyzed and the new control floor heave method of prestressed anchor and inverted-arch combined support was brought forward. Through 3-D numerical simulation by FLAC^{3D}, the reinforced effect of pump chamber supported by prestressed anchor and inverted-arch combined support system was analyzed. The numerical result shows that prestressed anchor and inverted-arch combined support can control floor heave in deep mine soft rock roadway effectively.

Keywords: Floor heave; Prestressed anchor; Inverted-arch; Combined support; Numerical analysis.

EXPERIMENTAL STUDY ON THE INFLUENCE FACTORS OF CABLE BOLT REINFORCEMENT

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The influencing factors to anchoring force of resin-grouted cable bolts are numerous and complicated. The anchoring mechanism of cable bolts has not been studied perfectly. In practice, the cable bolt supporting parameters are always designed empirically. So it is inevitable that the supporting parameters of cable bolts are always not suitable to the roadways' conditions. As the worst case, the reinforced surrounding rock may go into failure. Based on the survey results that more and more coal mine roof rock mass is bearing fracture water, some laboratory cable-bolt pull-out tests were carried out. Four factors are considered in this paper. They are embedment length, bore-hole diameter, resin capsule packaging and water. Water is highlighted as an important factor first time. A 7-wire high-strength stranded cable was used in the tests. The research results show that there are four models of cable load-distribution. The law of axial force distribution along the cable bolt is obtained, and it is some different from the conventional Mindlin distribution. The achievement is valuable to the design of roadways supporting parameters and gives insight into the bond mechanism of cable bolts.

Keywords: Rock reinforcement; cable-bolt; bond strength; influence factor; pull-out test; water.

AUXILIARY METHOD TO STABILIZE CUTTING FACE OF MOUNTAIN TUNNEL

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Recently, there are many tunnels whose portals are located on the slope mountain with un-equivalence pressure or at the crush zone with small overburden. For these portals, Pipe-roof method, Umbrella method etc. are applied as the auxiliary method for tunnel stability. Lately, fore-poling method (PU-IF) and AGF method in which general Tunnel-Jumbo can be used are applied in many unstable sites for effective fore-piling.

For applying these methods, economy, site execution and stability at the cutting face should be examined based on the measurement data during construction. In this paper, two sites adopting such auxiliary methods for stability of cutting face are presented.

In Shoukawa Tunnel, which is under thin overburden with unsymmetrical earth pressure, the vertical fore-poling bolt method was utilized to avoid the ground settlement, collapse at the cutting face and landslide. The vertical fore-poling mechanism was evaluated through the monitoring system for slope stability by using back analysis with field measurement data.

Another site in this presentation is Kitasuma Tunnel, a long twin bore tunnel with a center-pillar. Design of the center-pillar is important since 1km of this tunnel is located at urban area. Based on the result of FEM analysis, auxiliary methods of pre-grouting and PU-IF were applied before excavation for supporting ground surface.

Keywords: Auxiliary method; Fore-poling method; Unsymmetrical earth pressure; Stability.

REINFORCING ANALYSIS OF NEW PRESTRESSED ANCHORED ROPE BASED ON INTERFACE ELEMENT METHOD

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The improved prestressed anchored rope is proposed because of the fact that the cement mortar before the bearing plate of traditional type is adapt to be destroyed and lead to the instability of the structure (Fig. 1) The main characteristic of the improved prestressed anchored rope lies in the steel tube welded before the bearing plate so as to overcome the defect. Focusing on the new type of prestressed anchored rope proposed before, the refined models are established in this paper based on the interface stress element method of discontinuous mechanics. The characteristics of cement mortar, rock mass or soil body outside and materials on interface between different media are considered. The model can also imitate the non-linear behaviors of various materials. Through the numerical calculating analysis of the representative problems, the stresses of different media of the prestressed anchored rope in various cases are obtained. Evaluation has been made on the reinforcement of the new type of prestressed anchored rope, which lead to some valuable conclusions. The research results show: 1) The normal stress of steel tube is higher than that of cement mortar, showing that the steel tube can protect cement mortar to some extend. 2) The normal stress of cement mortar increases as the strength of outside medium becomes lower, which indicates that the reinforcement effect of the prestressed anchored rope is more obvious in the lower strength medium. 3) The normal stress of cement mortar is mainly concentrated on the root of the anchor rope, while shear stress of interface between cement mortar and outside medium on the root and head of the anchor rope. These parts are prone to be destroyed. 4) The method proposed in the paper can well reflect the discontinuous deformation of various materials and is an effective method for reinforcing analysis of prestressed anchored rope.

Keywords: Prestressed anchored rope; cement mortar; rock mass; interface element method; reinforcing analysis.

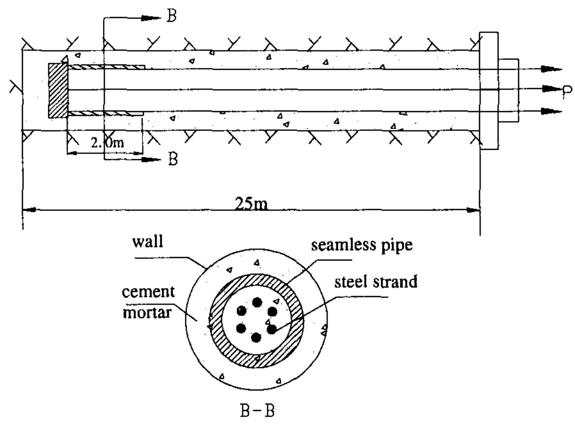

Fig. 1. Improved prestressed Anchor rope.

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6. ROCK MASS

6.1. General

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A PROPOSAL FOR THE MODIFICATION OF RQD (MRQD)

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The RQD of Deere (1964) is considered as a quick and economical factor in most rock engineering classifications such as Q, RMR and even in calculation of some Rock Geotechnical parameters and support design. Besides its high applicability, RQD has some deficiencies, which should be considered: a) RQD method does not give any information for spacing more than 10 cm (4 inches); b) In case of 9 joints per meter, if 1 or 2 joints are added, the RQD will decrease from Excellent to Very poor class. Which shows non realistic sensitivity of RQD to joint numbers; c) Rock masses with 11,20,and even 50 joints per meter have the same RQD. This is true, also, for fragmented or crushed rock masses, as well as cavities, RQD of all these rock masses are equal to zero. But, there is a distinct difference in Rock Quality for all above cases. So MRQD is suggested to correct and modify it for the best usage. This parameter could describe more completely the differences of various cases. The modified version of RQD (MRQD) is explained according to percentage of weak zone which includes discontinuities, core washed, core loss, fragmented and crushed zone, vuggy zone, and cavity. The relation between weak zone and modified RQD (MRQD) is: MRQD=100-W.Z.Based on MRQD, rocks are classified in to 5 classes. Using MRQD in Rock Engineering Classification is more appropriate for calculation of Rock mass Geotechnical Parameter.

Keywords: Rock classification;, RQD; rock mass.

ROCK MASS CHARACTERIZATION AND ROCK MASS PROPERTY VARIABILITY CONSIDERATIONS FOR TUNNEL AND CAVERN DESIGN

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In rock engineering practice, data, because of the huge cost involved in acquisition, are often incomplete and hence contain uncertainties. Uncertainties are inherent and unavoidable in the rock mass classification process. Uncertainties can be divided into several types: (a) uncertainties attributed to inherent randomness, natural variation, etc.; (b) uncertainties attributed to lack of data, knowledge about events and processes; (c) uncertainties due to our inability to understand decision objectives. Common sources of uncertainties in rock engineering include the spatial and temporal variability of the rock mass properties; random and systematic errors in data mapping, logging, testing, and monitoring; analytical and numerical model simplification; human omissions and errors.

To quantify the effects of these uncertainties on tunnel and cavern stability predictions, it is necessary to utilize probabilistic methods. In the present study, a quantitative, probabilistic approach to use the *GSI* system for rock mass classification is presented. It employs the block volume and a joint condition factor as quantitative characterization factors. The approach is built on the linkage between descriptive geological terms and measurable field parameters such as joint spacing and joint roughness, which are random variables. It allows the evaluation of both the means and variances of strength and deformation parameters, using Monte Carlo or point estimate method (Fig. 1). One example is given to illustrate how to consider the variability of the rock mass properties and reflect the quantitative risk assessment results in tunnel and cavern design. In this fashion, the design engineers are able to accomplish the challenging tasks for achieving a balance between the safety and economy in the design of rock support systems.

Keywords: Rock mass classification; *GSI* system; Rock mass strength; Variability; Cavern.

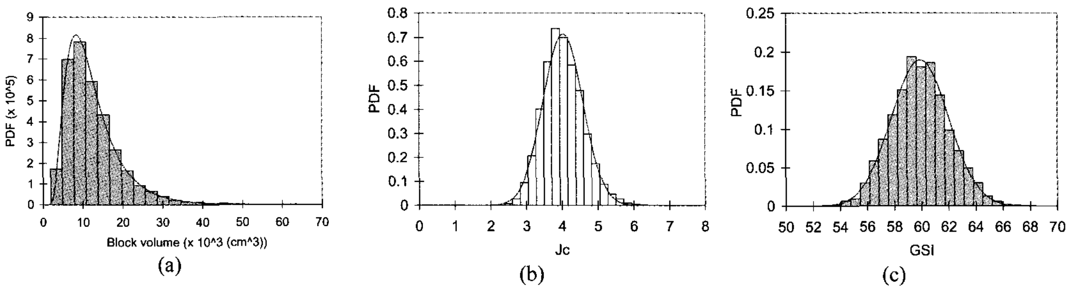

Fig. 1. (a) Block volume V_b; (b) J_c; (c) *GSI* distributions simulated using @RISK.

STRAIN-DEPENDENT PERMEABILITY TENSOR FOR COUPLED M-H ANALYSIS OF UNDERGROUND OPENING

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Most existing studies on hydraulic conductivity tensor for fractured rock mass are either stress-dependent or elastic strain dependent. As a result, material nonlinearity and post-peak dilatancy of fractures, which accounts for drastic variation of hydraulic conductivity, are not well considered in formulation of the permeability for disturbed rock mass, especially for the surrounding formations of deeply-buried tunnels with high in situ stress and high pore water pressure. In this study, by characterizing rock masses as an anisotropic continuum with one or multiple sets of critically oriented fractures, a methodology is developed to address the change in permeability resulting from engineering disturbance. An equivalent non-associative elastic-perfectly plastic constitutive model with mobilized dilatancy is presented to describe the global nonlinear response of the rock system. By separating the deformation of fractures from the equivalent medium, a strain-dependent permeability tensor is formulated, which not only considers the normal compressive deformation of the fractures, but more importantly, integrates the effect of post-peak shear dilatancy of fractures. This work differs from the existing studies on permeability tensor for fractured rock masses in that the formulation is total strain-dependent, rather than stress or elastic strain-dependent, thus exhibiting an advantage in describing the reality of fracture dilatancy, change in hydraulic properties and coupled mechanical-hydrological (M-H) process. The formulation has been implemented in a FEM code, and has been applied to investigate the coupled M-H effect of a deeply-buried underground tunnel excavation.

Keywords: Permeability tensor; Strain; Fractured rock mass; Hydromechanical coupling; Post-peak dilatancy.

APPLICATION AND RESEARCH OF SEISMIC INVESTIGATION METHODS TO PREDICT ROCK MASS CONDITIONS AHEAD OF THE FACE

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Tunnelling is a risky business. Any activity where it is expected to proceed according to a predefined plan and cost where the major influence, the geology, is at worst almost totally unknown and at best interpolated from boreholes intercepting only a minute percentage of the total tunnel volume has to be considered a high risk. However tunnels are still needed and more often the alignments are being dictated by the requirements of the client and not restricted by the geology or the available excavation technology.

Geophysical methods for investigating the ground, especially down the hole tools have long been used in the oil industry. Surface techniques using Seismic reflection and refraction can provide useful coverage or the ground conditions along proposed alignment however they are restricted by the effective depth and the ability to pick out sub vertical features.

The ideal geophysical investigation system is one that is operated from within the tunnel looking forward ahead of the face to provide an early warning system for changes in the rock mass especially sub vertical features which can lead to catastrophic events such as the intersection of fault and fracture zones. Early identification allows the organization of logistics and processes to be put in place to evaluate and mitigate the risks.

The TSP (Tunnel Seismic Prediction) is such a system for this application. This paper looks at the suitability of the TSP system to hard rock tunnelling in areas of high overburden and complex geology. With the evaluation of P-waves and S-waves TSP provides useful results to predict rock quality and significant features (reflectors) ahead of the face and is appropriate for production orientated jobsite applications. It presents a TSP case study form the current job site of the 57 km long Gotthard Base Tunnel crossing the Swiss Alps. Here, a downfall occurred at the Multifunctional Station Faido due to an unexpected major fault zone which is crossing the tunnel tubes under unfavorable tunneling condition. A comparison between the results of two TSP surveys and the geology being excavated afterwards realizes the optimal use of this method and how tunneling works will become more predictable in both costs and risks.

Keywords: Excavation; rock test; rock mass.

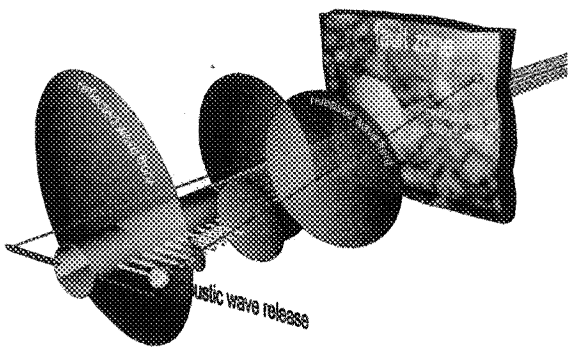

Fig. 1. Principle of the Tunnel Seismic Prediction Method: acoustical waves being generated by small explosive charges in the tunnel side wall are being reflected back at discontinuities in the rock mass.

ANALYSING OF THE REPRESENTATIVE LENGTH OF ROCK MASS SUBJECTED TO LOAD BY IN SITU LOADING TESTS TO EVALUATION OF ROCK MASS DEFORMATION MODULUS

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The *in situ* deformation modulus is an important engineering parameter required for designing many structures in and on rock, from underground openings to foundations. Among the rock mass properties, the deformation modulus has a vital importance for the design of rock engineering projects, because the deformation modulus is the best representative parameter of the pre-failure mechanical behavior of the rock material and of a rock mass. The in situ loading tests intended to measure the rock mass deformability seek to cover a representative part of the rock mass including the matrix rock and some fissures. This paper has reviewed all large in situ tests, to evaluate of the deformation modulus of the rock mass. The classification of in-situ loading tests for the deformation modulus determination has been proposed, basing on the type of opening and the loading condition, along with a scheme to measure the representative length of each loading test. Analyzing 9 case studies, it has been clarified that there is a considerable variability in results obtained from different loading tests, as the modulus value increases with the representative length. From the variability in results, it can be concluded that there is presently no one single method that is absolutely reliable. From the fact that the better results are provided by the tests which can cover a larger volume of rock mass, it has been discussed that the volume effect on results definitely needs further developments of higher-quality loading tests.

Keywords: Deformation modulus, representative length, in situ loading, higher-quality loading, conventional loading.

ROCK MASS QUALITY EVALUATION BY FRACTAL DIMENSION OF ROCK MASS DISCONTINUITY DISTRIBUTION

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The character of rock mass discontinuity is the dominant factor of rock mass quality. The rock mass discontinuity, owing to its property of self-similarity, can be addressed with the fractal theory although it is irregular in space and complex in network structure. Basic concepts of the fractal theory and the calculating method of the fractal dimension are introduced. The present study has testified the fractal characteristics of rock mass discontinuity distributions based on the calculation of the fractal dimensions of discontinuity distributions in the roadway surrounding rock masses at an underground mine with the application of the Box-dimension method. By analyzing the fractal dimension values, this investigation shows that the higher the value is, the denser the discontinuity distribution, the longer the discontinuity traces, and the lower the rock mass quality, which suggests the feasibility of taking the fractal dimension of rock mass discontinuity distribution as the index of rock mass quality rating. By comparing the fractal dimensions calculated above with the corresponding rock mass grades classified by *Standard for engineering classification of rock masses*, this paper proposes a scheme of rock mass quality classification, which takes the fractal dimension of the discontinuity distribution as the classifying index.

Keywords: Rock mass quality; rock mass discontinuity; fractal dimension; fractal theory; roadway surrounding rock mass.

RISK EVALUATION OF WATER INRUSH DURING SHAFT EXCAVATION IN FRACTURED ROCK MASSES

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Constructions of underground facilities are planned and carried out on the assumption that rock mass properties, such as hydro-geological structures are investigated previously. A construction cost is also estimated based on the rock mass properties obtained in the phase of feasibility study, F/S. It should be noted, however, that geological conditions obtained prior to the cost estimation phase always involve uncertainties, for it is practically impossible to foresee geological conditions of the whole construction site in the phase of F/S. Therefore, a risk of cost overrun will emerge if the actual condition of rock masses is worse than expected. Especially, at large-scale construction site, such as high-level radioactive waste disposal plants, variations of the construction cost caused by emergence of unforeseeable geological conditions might be enormous.

A water inrush from fractured zones is one of primary risk factors that cause cost overrun during the construction of underground facilities. This water inrush risk becomes serious especially in case of shaft excavations, for the deeper the tunnel excavation is, the higher a hydraulic pressure around the tunnel is.

From such a viewpoint, this study aims to propose a risk evaluation method for unexpected water inrush during shaft excavations in discontinuous rock masses. In this study, large faults of which hydro-geological structures are predicted in the phase of F/S are modeled deterministically. In contrast, other discontinuous planes are generated stochastically using discrete fracture network approach in order to represent heterogeneous rock masses. The amount of water inrush, which flows in excavated shaft, is calculated by using a seepage analysis, and then an influx classification is performed to associate the discharge to the cost. Moreover, Monte Carlo simulations are carried out to obtain variations of the cost statistically. Finally, the water inrush risk is evaluated as quantitative values by applying prevailed theory in the field of financial engineering, such as the risk curve and Value at Risk, VaR.

In addition, a development of a risk reduction method is investigated. In details, draining water by using pre-borings with the Decision Tree analysis is adopted to evaluate a risk of returns on investment quantitatively.

Keywords: Construction cost; water inrush; Value at Risk; risk reduction.

SIMULATION OF FRACTURE MECHANICS FOR ROCK MASSES UNDER VERY LOW TEMPERATURE CONDITIONS

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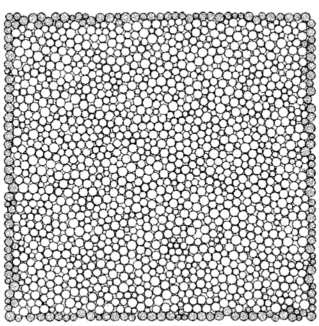
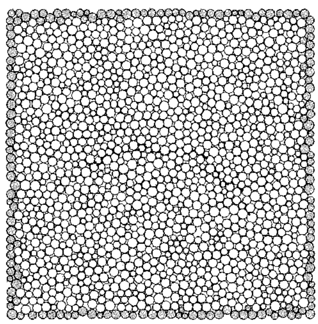
Failures of underground storage were due to thermal stresses generating cracks in the rock masses and thermal cracks contributed to induce gas leakage and to an increase in heat flux between LNG and the rock masses. At storage cavern scale almost all rock masses and especially hard rock masses are fissured. These already existing natural cracks will act as relaxation structures when the temperature is lowered. So it is very important to figure out the fracture mechanisms for rock masses under very low temperature conditions.

Since the conventional numerical codes such as FLAC and UDEC do not have the ability to simulate the development of cracks in rock masses, the Particle Flow Code PFC2D was selected, which was applied successfully for modeling of brittle failure observed around AECL's Mine-by test tunnel.

The present study considers greatly simplified geometries and was purposed to investigate the separate effect of different physical conditions such as jointing, in-situ stresses, and frost heave pressures on the formation of new cracks and the development of tensile stresses caused by the general rock shrinkage during cooling. On the basis of the results from thermo-mechanical modeling using a PFC2D code, it is certain that the presence of joints and frost heave pressure had a pronounced effect the formation of new cracks in the rock mass. The results found in this study are not entirely comparable with the observations obtained from real sites due to complicated geological and groundwater conditions. However, it could be possible to estimate the mechanisms of fracture using PFC2D models as shown in this paper.

Keywords: Thermal cracks; fracture mechanics; underground storage; thermo-mechanical modeling; PFC2D.


(a) No crack (-80°C)


(b) Fracture length: 0.5m (-60°C)

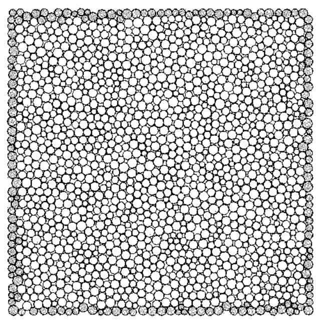

(c) Fracture length: 1.0m (-50°C)

Fig. 1. Fracture distribution with regard to frost heave pressures (K=0.5, vertical stress=1.3MPa).

NEW APPROACH FOR PREDICTION OF BEARING CAPACITY IN ROCK AND ROCK MASS

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The assessment of bearing capacity is always essential for safe and economical design of foundations in rock and rock mass. The field methods for estimation of bearing capacity are cumbersome to execute and time consuming too. So, laboratory and theoretical approaches are frequently preferred over the field methods. In this paper, the theoretical solution originally proposed by Bell (1915) has been implemented for prediction of bearing capacity of intact rock, jointed rock and rock foundations using failure criteria suggested by (Rao, 1984, Rao, 1998 and Ramamurthy, 2001). The formulated expressions of bearing capacity are tested at four rock mass sites and results obtained are compared with other existing methods. The results show that the proposed bearing capacity equations simple in form, predict values comparable with these methods. It is interesting to note that Hoek and Brown criterion (1988) under estimates the bearing capacity whereas GSI approach of Hoek et al. (1995) over predicts the same.

Rocks are inherently strong and stable to withstand to the loads imposed by usual foundations. However, due to the presence of weak planes, joints and other discontinuities, the strata become weak and require correct assessment of its bearing pressure while constructing on such materials. These problems become more severe especially while dealing with the foundations of tall buildings, arch dams, bridges, nuclear power plants and other such heavy and sensitive structures (Rao et al. 2002). Therefore, in most cases of rock structures, the ultimate bearing capacity of the footing is not limited with characteristics of the homogeneous and isotropic rock mass but with that of joints and bedding planes, which form the rock foundation.

A number of bearing capacity equations are reported in the literature, which provide explicit solutions for the ultimate bearing capacity. Most of them consider the mode of failure as well as to some extent material properties of the rock strata.

The strip footing resting on rock with wide rock joint spacing, due to some plastic reaction will redistribute the stresses and its bearing capacity may be estimated similar to intact rock using the classical theories of Terzaghi, Prandtl, Hansen, and others. Heavily fractured rock with closed spacing of joints may be treated as dense granular mass. But the limitation in this case is the measurement of the engineering properties i.e. shear strength parameters (c and ϕ). Very few studies have been carried on rock foundation that explains the effects of discontinuities on modes of failure and bearing capacity. The works reported in the literature are for closely fractured rock that suggests the procedure similar to soil foundation (Hoek and Brown, 1980 and Bell,1915. Several empirical methods are available in the literature which have successfully predicted the strength of rock containing a single plane of weakness (Jeager and Cook, 1979) and that of rock mass (Rao, 1984 and Rao, 1998); however very few of them have been utilised for assessing the bearing capacity of rock mass.

In this paper, using a semi empirical strength criterion for rock mass (Rao, 1984,1998 and Ramamurthy, 2001), an equation for bearing capacity of rock foundation has been developed assuming the wedge to be defined by straight lines to form active and passive zones satisfying the equilibrium and failure criterion similar to Hoek and Brown (1980), and Bell (1915). Further the expression of the bearing capacity has been extended to the jointed rock as well as to rock mass.

An actual field case has been selected to demonstrate the applicability of the proposed relationship. The bearing capacity predicted using the proposed equation is compared with the values obtained using several other procedures suggested in the past for different sites.

Keywords: Rock Mass; Bearing Capacity; Model Study.

PREDICTION OF GROUND CONDITION AND EVALUATION OF ITS UNCERTAINTY BY SIMULATED ANNEALING

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At the planning and design stages of an underground space or tunnelling project, the information regarding ground conditions is very important to enhance economical efficiency and overall safety. In general, the information can be expressed using rock mass classification such as RMR (Bieniawski, 1984) and Q-system (Barton et al., 1974) and with the geophysical exploration image. Rock mass classifications using borehole data can provide direct information of the rock mass in a local scale for the design scheme. Oppositely, the image of geophysical exploration can provide exhaustive but indirect information. These two types of information have inherent uncertainties from various sources and are given in different empirical scales and with their own physical meanings. In this study, simulated annealing (SA) was applied to overcome the shortcomings of kriging methods or conditional simulations using just a primary variable. Using this technique, the RMR and the image of geophysical exploration can be integrated to construct the spatial distribution of the RMR and to evaluate its uncertainty. A practical procedure of SA technique for the uncertainty evaluation of the RMR was suggested and also demonstrated through an application, where it was used to identify the spatial distribution of RMR and quantify its uncertainty. For a geotechnical application, the objective functions of SA were defined using statistical and spatial models of RMR and the correlations between RMR and the geophysical image. The use of SA technique for predicting the ground condition prior to planning and design of underground excavation has been demonstrated in this paper. It has been found that the SA technique is quite capable of integrating various types of data and optimizing a spatial distribution of a primary variable with constraints, which are designed by statistical and spatial modelling. The applicability of SA technique has also been demonstrated by constructing a map of RMR and evaluating its uncertainty to determine an optimized layout or alignment of tunnel by comparison among a few of candidate alignments. The SA technique can also be used to update an already predicted ground condition by using the new information obtained form a proceeding excavation.

Keywords: RMR, simulated annealing, spatial distribution, uncertainty.

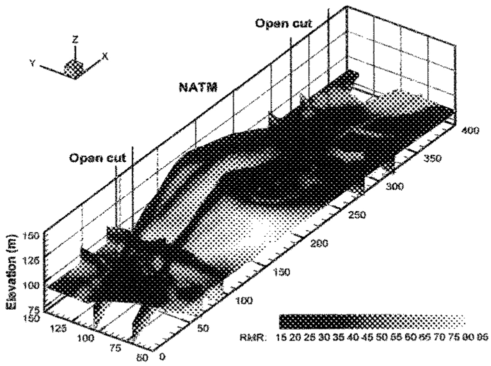

Fig. 1. 3 dimensional spatial distribution of RMR by Simulated Annealing technique.

CONSIDERATIONS ON LONG-TERM STRENGTH OF JOINTED ROCK UPON THE HOMOGENIZED CRACK PROPAGATION

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Strength of rock is deeply associated with a stress-dependent interactive crack growth from tips of neighboring compressible and incompressible joints involved within a rock mass. A new scheme is proposed in order to evaluate the quasi-brittle fracture of jointed rock and to discuss the relation between the interdependent crack growth and the strength failure.

In this article, assuming a formation of kinks at the joint-tips, the open crack propagation from tips of kinks is analyzed as the pure mode I crack propagation ($K_{II} = 0$). In particular, to clarify the effect of interdependent crack extension from neighboring joints, the homogenization scheme is successfully applied along with a compressible dilatational joint model. Homogenized models of joints are firstly described along with a modeling of compressible joints and dilatational sliding condition, introducing the relative compressibility ratio (B) of joint, along with the coefficient of friction (μ) and the dilatancy angle (ϕ_d).

Homogenized crack propagation analysis is carried out by means of the SC-DDM, using infinite models subjected to the pressure (p) at infinity. The mode I stress intensity factor (K_I) at tips of propagating cracks are precisely analyzed to evaluate the critical applied pressure (p_c) when K_I reaches to its critical value (K_{IC}), namely the crack propagates with a critical rate (v_c). Subsequently the critical pressure is analyzed as a function of the crack elongation, and the crack elongation rate at various pressure levels is evaluated upon the so-called Paris's law. From such a crack elongation rate analysis, the time of quasi-brittle delayed fracture (t^*) caused by a static fatigue is formulated as follows:

$$t^* \propto (p/T)^{-\frac{1}{n}}. \tag{1}$$

where p is the applied pressure, T is the tensile strength and n is a constant called the stress erosion index.

Upon the homogenized crack propagation analysis, it is discussed that the stress-dependent interactive crack extension in rock is deeply influenced by the joint spacing and the compressibility of joint. By the crack elongation rate analysis upon the Paris's law, it is concretely shown that the most important parameter to estimate the long-term strength of rock is the stress erosion index (n), and that the present analysis is a promising tool for the analysis of rock strength characteristics.

Keywords: Rock strength; Compressible joints; Homogenization; Pure mode I crack propagation.

MODIFIED ROCK MASS CLASSIFICATION SYSTEM FOR PRELIMINARY DESIGN OF ROCK SLOPES

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Considering lacking of suitable system in characterizing heavily jointed rock masses in slopes, attempts were made in this study to propose a modified rock mass classification system for preliminary design of rock slope. For this, the modified Geological Strength Index (GSI) was employed as a basis, and the relevant geotechnical data were collected from different rock slope sites in IRAN. Among the studied sites, 8 sites were selected and their stability was analyzed by means of the CLARA scientific software. To establish the proposed classification system, besides the GSI, five more parameters as: uniaxial compressive strength, rock type (lithology), slope excavation method, groundwater condition and earthquake force, were considered, and their relative effects on stability of fractured rock slopes were studied. By analyzing sensitivity of stability of the studied rock slopes to the above five parameters as well as GSI value of the rock masses, a logical basis was presented for determining the proposed rating system values. Then the weight of each parameter was defined by statistical analysis of the sensitivity analyses results, where rounded mean values were fixed as rating values incorporated in to the classification of the rock masses.

In the proposed rock classification system, a rating is allocated to each parameter and an overall rating for the rock mass is evaluated by summing the rating of all parameters. This overall rating is denoted as Slope Stability Rating; SSR. The interpretation of this new rating system (SSR) for the stability of the rock slopes has been achieved by preparing a number of design charts for different values of safety factor (Fig.1). Finally, by comparing the actual stability condition of the studied rock slopes with the stability evaluating results using SSR system, the accuracy and reliability of the presented design charts were approved.

Keywords: Rock mass classification system, Slope, Design, Iran, GSI, SSR.

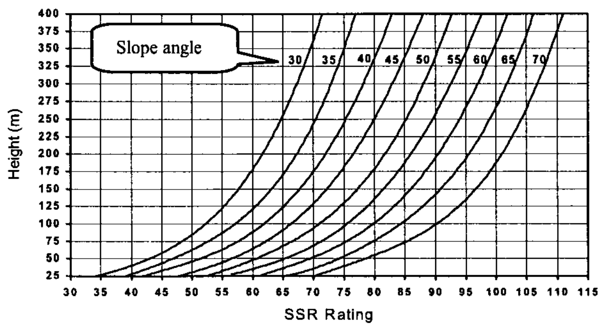

Fig. 1. Rock slope design chart, based on SSR values of rock masses (F.S=1.5).

ENGINEERING BEHAVIOUR OF SIMULATED BLOCK MASS MODELS

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One of the major problems confronting designers of engineering structures in rock is that of estimating its strength. In field rock mass may be consisting of two to four dominating sets of joints, which make the rock mass highly anisotropic in strength and deformational response. So, it becomes very important to know the strength and deformational properties of rock mass than that of intact rock while considering design for foundations, slopes, and underground structures. The importance of intermediate principal stress σ_2 on strength response of intact and anisotropic rocks has been advocated by many scientists in the past based on their laboratory and field experiences. The scanning of literature shows that this area has received still not much attention as far as study on rock mass behaviour is concerned. To assess the engineering response of rock mass under the influence of intermediate principal stress, a physical model study was undertaken in the present work. Cubical rock mass specimens of size 15 cm with three orthogonal joint sets were prepared in laboratory using sand-lime blocks of unconfined compressive strength 13.5 MPa and testing was performed in the True Triaxial System developed recently.

The rock mass geometries and magnitudes of horizontal principal stresses (σ_2/σ_3 ratio) along with their directions were changed to simulate variety of field conditions occurring at site. The inclination of continuous joint set-I was varied as θ = 0°, 20°, 40°, 60°, 80°and 90°, keeping the staggering of joint set-II equal to 0.5 of small cubical element of rock mass specimen and joint set-III was vertical and continuous. The horizontal stress ratio σ_2/σ_3 was increased from 1 to 5.2. The engineering properties viz. strength, modulus, modulus and failure pattern are found to be influenced by interlocking level of joints, σ_2/σ_3 ratio and direction of σ_2 with respect to dip direction of critical joint set-I. The observations show the shearing of intact material and joints in rock mass specimens with θ = 0°, 20°, 80° and 90°. The shear planes developed on YZ or σ_2 face dips along σ_3 directions and the fracture dip increases with increasing σ_2. Similarly, rock mass with θ = 40° and θ = 60° exhibited the joint dilation and shearing of some blocks. These modes of failure shifted from sliding, dilation to shearing with increasing σ_2/σ_3 ratio. The maximum enhancement in strength upto 309% was seen due to σ_2 in rock mass specimens with θ = 60°. At θ = 90° this enhancement is lowest (26%). Similarly maximum enhancement in modulus and modulus ratio is 560% and 186 % respectively, which corresponds to θ = 60°.

Anisotropy in strength, modulus and modulus ratio properties are also seen, which reduces with increasing level of intermediate principal stress.

Keywords: Simulated block mass; True triaxial stress; Anisotropy; Physical modeling; Intermediate principal stress.

EVALUATION OF ROCK MASS QUALITY AND ITS APPLICATION

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In rock engineering, the rock mass can be classified as loaded or unloaded according to its mechanical stress state and history. The rock is in an unloaded state during excavation of steep high slopes, whereas it is in a loaded state in the tangential direction and in an unloaded state in normal direction in deep underground excavations.

For high slope deep excavations and the large deep underground tunnels, major factorsare the depth, the energy release magnitude and rate, the development of tensile stress zones, and the unloading associated with the establishment of the secondary stress condition after the release of energy associated with the primary excavation. After the tunnel is advanced, the tangential direction of the secondary state of stress is a loading condition and the radial direction is an unloading condition increasing greatly the deviatoric stress. Due to this unloading during high slope and the tunnel excavation, the qualities of the rock mass deteriorate and the degree of unloading of the rock mass is different in different unloading zones. It is necessary to divide the unloading zones into different areas and analysis the mechanical features of each area.

The goal of research on the problems of slope and underground engineering is keeping the construction stable. Numerical methods are available to address some of the issues associated with stability, but if the mechanical parameters for the intact and damaged portions of the rock mass are not chosen appropriately, the analysis may be in vain or even seriously misleading. Thus selecting suitable parameters is a major focus of the application of numerical methods to analyse this class of rock engineering problems.

The quality of the rock mass is one of the problems of most concern to the rock engineering. One should account for the deterioration that a rock mass undergoes during unloading, which leads to a decline in rock mass quality. How to choose appropriate mechanical parameters for the rock mass is a problem for rock engineers, and we take the RMR method and a method of estimation to evaluate the quality of an unloaded rock mass, calculating the RMR value of the unloaded rock mass with the established relation between RMR value and deformation modulus, allowing us to assess mechanical parameters and evaluate the rock mass quality in a manner that can be used as a reference for other projects.

After the excavation of the dam abutment slope along the center-line of the double-arch dam at the Baihetan hydropower project, the rock mass quality is deteriorated by the de-stressing, and this is accounted for in the unloading rock mass mechanics theory based on the applications of a Hoek-Brown rock mass failure criterion. According to the relation between RMR value and deformation modulus that Serafim and Pereira (1983) put forward, the research program makes use of the deformation modulus after excavating and unloading to calculate the RMR value in the affected rock mass, thereby estimating the m, s, c and ϕ values in the 21 identified regions within the rock mass. These estimated parameters are used for engineering design and to guide construction work.

Keywords: Unloaded rock mass; deformation parameter; quality evaluation; reference.

COMPLETE STRESS-STRAIN CURVE FOR JOINTED ROCK MASSES

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The presence of discontinuities within rock highly perplexes the prediction on the stress-strain relationship of rock masses, especially for its post-peak behavior. For the analysis of engineering problem dealing with rock materials, it is essential to well modeling its complete stress-strain curve. Based on the concept of equivalent continuum, this paper deduces a three-dimensional, nonlinear complete stress-strain curve that integrates the mechanical behavior of intact rock and discontinuities into a jointed rock mass through representative volume element. The mathematic model accounts for the effects of joint spacing, orientation, number of joint sets and properties of joints and intact rock. The validity of the proposed nonlinear constitutive law has been tested by comparing the results predicted to analytical solution and existing model. Applicability of the model has also verified by comparing with the outcomes of laboratory tests. It is believed that the proposed model provides a better representation for the complete stress-strain behavior for regular, well persistent jointed rock mass.

Its application shows that the number of joint sets and their attitudes play key roles on the complete stress-strain relationship of rock masses. The isotropic complete stress-strain curves of intact rock material become highly anisotropic as the joint sets number is less than four. The difference of strength and secant deformation modulus of rock masses can reach several decuples with various joints attitudes. The complete stress-strain curves get back to isotropic if the presence of joint sets is greater than four.

The stress states also affect the complete stress-strain curves of rock masses, and the influence is swayed by the relativity between the direction of principal stresses and attitudes of discontinuities. The proposed nonlinear constitutive law is able to account the effect of middle principal stress. The calculated results show that the effect of middle principal stress on strength and secant deformation modulus of rock masses is of important as joint strike is perpendicular to the middle principal stress and parallel to the minor principal stress.

Keywords: Jointed rock masses; constitutive law; complete stress-strain curve.

A PARAMETRIC STUDY ON FLOW OF GROUNDWATER IN FRACTURED-POROUS MEDIA: 3D SIMULATION

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A typical rock mass consists of intact rock containing embedded planar discontinuities (e.g. faults, joints, lithological contacts). Flow modelling in intact rock can be relatively straightforward, however, the presence of randomly oriented existing discontinuities as well as the emergence of new joints by the propagation of existing discontinuities due to disturbance of the rock mass (e.g. unloading during excavation) can make flow modelling in jointed rock a rather complex process. The presence of interconnected fractures provides a preferred pathway for fluid flow through porous media and thus consideration of the effect of fracture proves to be a fundamental aspect of fluid flow modeling. With the recent rapid expansion of the mining industry, problems arising from the presence of water in mined rock have become particularly pertinent. To meet the demands of the mining industry, numerical techniques for the analysis of groundwater problems have attracted considerable attention from a number of researchers. However, due to requirements for more faithful approximations to physical behaviour, numerical methods for the accurate prediction of fluid flow behaviour in porous media are still developing.

This paper discusses numerical modelling of groundwater flow around and into a tunnel in a fractured porous medium, carried out using the commercial finite difference code FLAC-3D (Itasca, 1996). A FISH, in built in FLAC, function was written and used for the purpose of estimation of the total volume of water ingress to the tunnel. The results of the numerical modelling are systematically analysed to investigate the effects of the distribution of insitu stresses between horizontal and vertical components, the influence of differing hydraulic boundary conditions and the effects of the scale of boundary blocks on the efficiency of the numerical model. In the analysis, fluid flow is modelled using a porous flow approach. The effects of the distribution of insitu horizontal to vertical stresses in the rock mass, hydraulic boundary conditions, and the scale of boundary blocks on total water flow toward a model cavity are investigated. The findings of this study show that the induced stress field has a significant influence on flow-deformation characteristics, depending on the orientations and interconnectivity of the joints. Beyond an insitu horizontal to vertical stress ratio of 1.0, the change of joint flow rate is marginal for the trialled models. Moreover, the study suggests that the most appropriate block size to be used for flow calculation in a fractured rock mass is 10-12 times the maximum dimension of the excavation.

Keywords: Flow; groundwater; porous media.

6.2. Theoretical and Numerical Analyses

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SIMULATION OF STRATIFIED ROCKS USING COSSERAT MODEL

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Stratified rocks are commonly encountered in coal mining, which is much weak in stratified direction (usually in horizontal direction) compared to the normal direction (vertical direction). The layered rock typically fails in opening and shearing off at the interfaces and consequently in bending of intact rock and its shearing and tension failure. Such a failure mechanism cannot be modelled by using conventional models.

The conventional continua-based models assume rock as a continuous medium and thus neither opening nor shearing off is not allowed inside the element. The discrete element model can involve rock joints physically in the model, but as the joint space is usually as close as 0.2-0.3 m, it is not practical to involve all the joints in the computational model.

COSFLOW, which has been recently developed by CSIRO Exploration and Mining, is an efficient tool to simulate overburden movement in various geological and mining conditions. COSFLOW incorporates the layered Cosserat continuum formulation. This provides major benefits over conventional continuum numerical models of sedimentary rock in that the code is able to efficiently simulate rock breakage and slip as well as separation along bedding planes. In addition, COSFLOW couples fluid flow with strata deformation and stress, and can be used to simulate the effects of mining on ground water systems and predict gas emission during longwall mining.

Using Cosserat model, a parametric study on bed separation due to longwall mining is carried out by varying longwall advance, rock properties, mining height and panel width. From the parametric study, several major findings are obtained.

- A separation typically experiences a dynamic process from motivating, opening, expanding, tending to close, closing, further closing and closed along with the longwall advance. The occurrence and development of separations varies in space and time. Some separations are completely closed while the others may still remain open after the mining stopped.
- A separation developing at an interface can be described as follows. Assuming an interface between two units, once the lower unit starts to fail, the incompatibility between the two units appears and the separation motivates. With the rock failure extending upwards in the lower unit, the separation is increasing in size. When the whole lower unit is failed while the upper unit still remains at an elastic status, the separation reaches its maximum in size and then the lower unit tends to decelerate its downward movement. Along with the longwall advance, the rock failure extends upwards to the upper unit and the upper unit accelerates its downward movement. This results in the separation tending to close until the upper unit is completely failed. If there is another unit existed above the upper unit, the same process will occur, which causes the previously opened separations to close further.

Keywords: Simulation; stratified rock; mining.

APPLICATION OF MESHLESS METHOD FOR BEHAVIOR ANALYSIS OF JOINTED ROCK MASS

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In analyzing the mechanical behavior of engineering materials, the methods based on meshes such as finite element (FEM) or finite difference (FDM) when large deformations and large number of discontinuities are involved, are not suitable. The conventional process for dealing with moving discontinuities in these methods is to remesh the domain in each step of the process which leads in reduction of speed and degradation of accuracy and complexity of the calculations. To overcome these problems the meshless methods are introduced and applied which are found very suitable for problems with change of geometry. There are a number of meshless methods, such as the Element Free Galerkin (EFG) method. In this paper EFG method is used for the mechanical analysis of intact rock mass and jointed rock mass, the penalty method is used for essential boundary condition and good results were obtained. Moving least squares approximation has been used to construct shape function for element free Galerkin method. Moving least squares approximation produces shape function that does not possess the Kronecker delta function. The penalty method is effective approach for enforcing the essential boundary condition that is used in this paper. The method based on meshes such as FDM and FEM have other difficulties such as the bad geometry, because of null of domain or joint creation. Also the time of calculation for EFG method is less than that for the methods based on meshes such as FDM. The displacement field of each block is constructed by equilibrium equation from an array of points distributed in the blocks. These nodes scattered in the problem domain and on the boundaries of the domain. It is found that the present method is more suitable for modeling discontinuities of rock mass in two-dimensional and particular three-dimensional domains than conventional methods based on meshes

Keywords: Element Free Galerkin Method; Jointed Rock Mass; Meshless Method.

ELASTIC-PLASTIC ANALYSIS OF JOINTED ROCK MASS USING MESHLESS METHOD

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Analysis of rock masses with plenty of weakness planes and joints with numerical mesh-based methods encounters difficulties. In the analysis when even only a few meshes are involved, mesh generation can consume more time and effort compared to the construction and solution of the discrete set of equations. Continuous remeshing of the domain in order to avoid the break down of the calculation due to excessive mesh distortion is other concern. These problems lead in reducing accuracy and increasing complexity in computer codes. To overcome these shortcomings recently the meshfree methods have been developed and found applications in different engineering fields. The real behavior of jointed rock mass is nonlinear and elastoplastic. In this paper the application of mesh-free (Element Free Galerkin) method in analyzing the nonlinear and elastoplastic behavior of jointed rock mass is presented. Moving least squares approximation has used to construct shape function for element free Galerkin method. Moving least squares approximation produces shape function that do not possess the Kronecker delta function. The Lagrange multiplier and penalty method are used for enforcing the essential boundary condition. The penalty method is effective approach for enforcing the essential boundary condition that is used in this paper. This analyzing is new search and is obtained good results. In this method, there is no need to use meshes or elements for field variable interpolation. The nodes scattered in the problem domain and on the boundaries of the domain. The nodes remain constant while the geometry of the domain is changing, therefore saving the time and computational effort in analyzing process. The minimum total potential energy principle states that for a structure system that is at equilibrium, the tptal potential energy in the system must be stationary for variation of admissible displacements. A few examples of application of the method in predicting the plastic behavior of jointed rocks are presented and its suitability is highlighted.

Keywords: Nonlinear analysis; Elastoplastic analysis; Element free; Jointed rock mass; Meshless.

STOCHASTIC SIMULATION OF ROCK MASS PROPERTIES USING A MODIFIED GENETIC ALGORITHM

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Engineers use the borehole rock mass classification data and geophysical site investigation results for the rock mass classification. There have been many cases where the borehole-logging data is not acquired in the field because of the cost and difficulty of access to a tunnel site due to geographical features or an existing facility. Subjectivity of the engineers results in a variety of rock mass classifications according to locality and the limited information from the borehole data. Kriging is one of the most widely used interpolation methods in geostatistics. Despite its wide use, the kriging map flattens out the local details of the spatial variation with small values being overestimated while underestimation with large values occurs. This type of selective bias is a serious shortcoming due to the loss of the distribution features of the original data.

In this study, a hybrid genetic algorithm (GA) was used to find the optimal image reproducing both the sampling histogram and the semi-variogram model. Additionally, we desired to preserve the data values. GA performance could be improved by designing and changing the genetic operator, which is critical when dealing with large full-field simulation models, such as reproduction, crossover, mutation, and selection. Most of the past research in geotechnical engineering used a simple GA with a one-dimensional chromosome and a simple genetic operator, such as a one-point or two-point crossover and the random change of a bit of chromosome.

This paper describes in detail the formulation of a modified GA to estimate the rock mass rating (RMR) in the section No. 302 of the works of the Seoul Metropolitan Subway Corporation (SMSC). An effective initial solution was determined by comparing the random number generating method with the sequential indicator simulation (SIS). A two-dimensional chromosome was designed to maintain the spatial correlation of the original logging data, and a modified genetic operator and local optimization algorithm were used to improve the efficiency of the genetic algorithm.

The modified GA found the RMR estimation efficiently by using a 2-dimensional chromosome, improved GA operators, and a 2-OPT (two-point optimization) local optimization algorithm. The objective function could consider heterogeneity as the horizontal and vertical directions, and reproduced the semi-variogram of the input data. Through the analysis of results from 30 equally probable simulations, the RMR value could be presented in the form of a probability distribution function and the uncertainty of estimation could be successfully quantified. The reliability of the simulation was analyzed by split-sample validation.

Keywords: Rock mass rating; stochastic simulation; genetic algorithm; sequential indicator simulation; optimization; geostatistics.

NUMERICAL ANALYSIS ON ROCK FAILURE MECHANICS UNDER LOADING AND UNLOADING CONDITIONS

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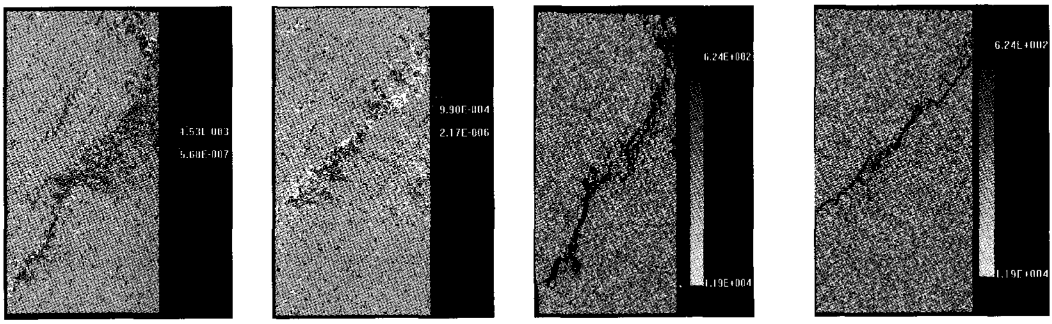
During engineering practice, it has been noticed that the mechanical characters of rock under loading and unloading conditions are different. Unfortunately the mechanical parameters are still taken as constant during excavation, which falls short of engineering practice. In this paper, the mechanical characters of intact rock mass under loading and unloading conditions were analyzed by using the RFPA^{2D} numerical method. Simulation results show that the failure of intact rock mass under unloading conditions is mainly caused by tension stress, while failure under loading conditions is mainly caused by both shear and tension stress. Angle between fractured surface and the direction of σ_1 under unloading conditions is bigger than that under loading conditions, which means that the intact rock mass are easier to fail under unloading conditions. The increasing of acoustic energy under unloading conditions is more stable compared with that under loading conditions before failure, which indicates that the unloading intact rock mass will take on a sudden failure. Results also show that the peak strength under unloading conditions is smaller than that under loading conditions. It can be concluded that localized damages caused by unloading, which in turn decrease the mechanical parameters of rock mass are the main reasons resulting in the failure of rock mass.

Keywords: Rock mass; Unloading; Numerical simulation.



AE picture under unloading condition | AE picture under loading condition | Fractured surface under unloading condition | Fractured surface under loading condition

Fig. 1. AE pictures and shear force pictures under loading and unloading conditions. The white circle positions stand for compression failure, whereas the red circle positions stand for tension failure. The size of circle stands for the amount of energy.

APPLICATION OF A FUZZY MODEL TO ESTIMATE THE ENGINEERING ROCK MASS PROPERTIES

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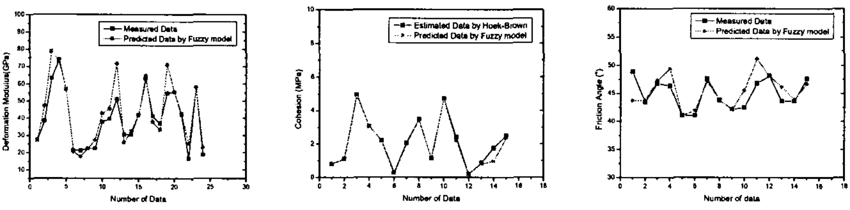
The engineering rock mass properties such as deformation modulus, cohesion and friction angle are required as the input parameters of a numerical analysis for the design and stability analysis of the underground structures. These values are conveniently obtained throughout the site investigations, in-situ and laboratory tests and rock mass classification systems. There, however, have been few researches on the evaluation of uncertainty and imprecision that can be included in determining the engineering rock mass properties. Some attempts have been made to increase the reliability of these values by using the statistical method such as Monte Carlo simulation, but it is not sufficient owing to the large differences between different simulations. Therefore, it is necessary to develop a reliable evaluation method for the engineering rock mass properties. In this study, a prediction model by using the fuzzy inference system is proposed for the evaluation of the input parameters. The fuzzy model used herein is the modified Takagi-Sugeno type model using a data clustering method of which the optimization criterion is the minimum RMSE (Root Mean Squared Error). The data sets for learning and verifying the fuzzy model are collected from a domestic tunnel construction site in Korea and they consist of the several categories including laboratory test results and rock mass classification values. The comparisons between the results obtained from the empirical equations and that from the fuzzy models are made. It is concluded that the performance of the fuzzy model is satisfactory, and also the ability for the prediction of fuzzy model is better than that of the empirical equations for the deformation modulus because the fuzzy model is trained by the data of the specific tunnel construction site. Sufficiently large database, however, is required for training the fuzzy model for the more accurate prediction and wide availability.

Keywords: Engineering rock mass properties, deformation modulus, empirical equation, TS fuzzy model, fuzzy clustering.



(a) (b) (c)

Fig. 1. The plot of measured and predicted values for each output variables in training the models: (a) Deformation modulus; (b) Cohesion; (c) Internal friction angle.

AN ANALYTICAL STUDY ON ELECTRICAL RESISTIVITY-BASED ROCK MASS CLASSIFICATION

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Rapid increase of demands for various natural resources owing to industrial revolution has led many researchers to find effective methods to detect those varied in underground. One of prominent methods is a geophysical exploration using electrical resistivity. Although the electrical resistivity-based geophysical exploration has been commonly used for site investigation, the majority of such application methods are restricted to coarse descriptions of underground conditions and only a few researchers have tried to relate electrical resistivity with rock mass classification systems.

The Q-system is commonly used as a representative rock mass classification system in modern rock engineering because it properly captures the important characteristics of a rock mass. In this paper, electrical resistivity is correlated to the Q-system through theoretical analyses. The analyses are based on Coulomb's law and Gauss' law considering electrical characteristics of constituent parameters for rock mass classification such as joint roughness, joint alteration, joint set number and RQD. Q-system is related to electrical resistivity as follows:

$$\rho^{Q}_{mass} = C \times \frac{QR \cdot NR \cdot RR}{AR} \tag{1}$$

where QR, NR, RR, AR are the coefficients of normalized indices related to each parameter in Q-system and the constant C is the electric resistivity which depends on the type of rock.

The analytical results show a strong correlation between electrical resistivity and Q-value. However, large data scattering suggests that direct conversion from electrical resistivity to rock mass classification is inappropriate. This issue is addressed through a set of proposed guidelines for classifying rock mass by using electrical resistivity.

Keywords: Electrical resistivity; rock mass classification; Q-system.

APPLYING THE THEORY OF SEISMIC INTERFEROMETRY TO GEOLOGICAL SURVEY USING ARTIFICIAL SOURCES IN TUNNELS

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The reflection response can be obtained by the cross-correlation of the transmission responses, which is called "seismic interferometry". The method intends to image subsurface geological structures using underground noise as a transmission wave field observed on the surface and to simulate pseudo shot records as many as the number of the receivers corresponding to data of reflection seismic survey.

We introduce a result of our field test done toward practical use of the seismic interferometry. We carried out the field test with underground moving sources in a hilly area where there exist two tunnels. For the effective seismic interferometry, long term recording data with many receivers is desirable. For this field experiment, we newly developed the PC-based seismic recording system which can continuously record the transmission data of 96 receivers simultaneously for arbitrary time period As the seismic source, Digi-pulse and air-knocker hit the surface of the tunnel road at equal intervals in the driveway tunnel and the sidewalk tunnel respectively, moving from the entrance to the exit of the tunnels. The truck kept running in the driveway tunnel intently and drove 10 times round trips. The record lengths of each source are 1,840s (Digi-pulse), 2,140s (air-knocker) and 1,600s (truck). A survey line with 96 receivers was located on the hill just above the sidewalk tunnel.

In order to image the subsurface structures, the conventional data processing was applied to the shot records simulated by cross-correlation of the observed data. Obtained subsurface images being properly data-processed are clear enough to estimate the subsurface structures. In stacked sections, geological structures parallel to the surface inclining to the east side can be clearly visualized, and they are identical to the result of the reflection seismic survey. Our result strongly supports the applicability of the seismic interferometry to image subsurface structures from the passive record observation of the noise generated by social activities.

Keywords: Seismic Interferometry, Cross-correlation, Moving sources.

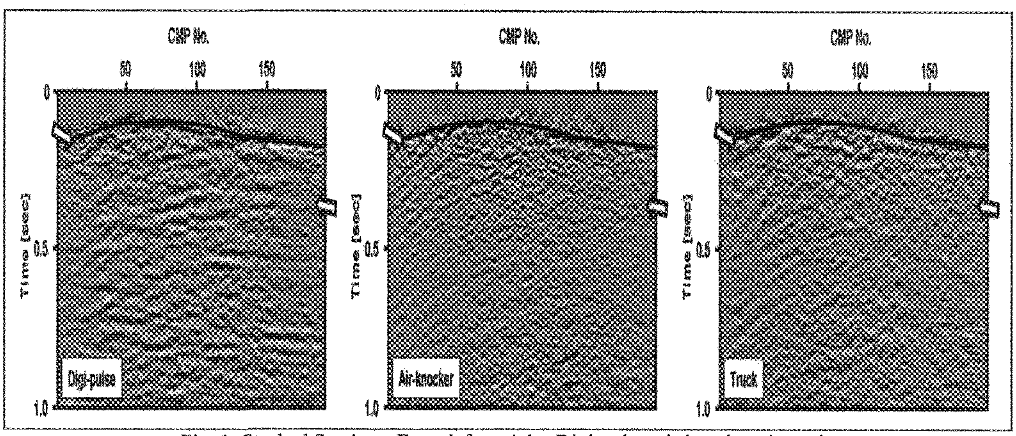

Fig. 1. Stacked Sections. From left to right: Digi-pulse, air-knocker, 4t truck.

GEOTECHNICAL CONCERNS DURING THE DEVELOPMENT OF THE AB TUNNELS IN PT FREEPORT INDONESIA

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The Ali-Boediarjo (AB) Tunnels are part of the Common Infrastructure Project being undertaken by PT Freeport Indonesia (PTFI) in the province of Papua, Indonesia to provide access to several deep underground orebodies which are expected to replace the current Grasberg Open Pit. These 6.8 m by 6.5 m fully arched twin tunnels are designed to last for at least 30 years as part of the long-term development of facilities for the underground projects. The tunnels are 24 meters apart and are connected by crosscuts at 200 meter intervals and are developed using a conventional drill and blast method.

Developments in shotcrete technology prompted PTFI to use early strength shotcrete as the primary temporary support supplemented with weld-mesh and split-sets where required. The use of early strength shotcrete as the temporary support allows early and safe re-entry to the face and removes the installation of permanent support from the development cycle thus speeding up the rate of advance of the tunnels.

Permanent ground support designs for the AB Tunnels have been published in the feasibility study report. The ground conditions expected in the tunnels were categorized into four classes from Class A (Very Good) to Class D (Poor) and the applicable ground supports were planned by estimating the conditions in different parts of the tunnels. During the daily inspections, the ground conditions and the ground support requirement were re-assessed and necessary modifications were recommended by the geotech engineers. Ground support designs for wide span areas, such as cross-overs, magazines and intersections were not dealt with in the feasibility study and were undertaken as special cases based on the excavation dimensions and observed ground conditions.

The development plans focused on maximizing advance rates through the use of state-of-the-art equipment. In the absence of extensive geological and geotechnical information, the use of pilot drill holes and daily inspections have helped in the safe development of these tunnels. The use of in-cycle shotcrete as temporary ground support has also helped PTFI achieve high average advance rates in the development in the tunnels.

This paper outlines some of the geotechnical concerns faced during the development of the AB Tunnels and actions taken to improve the safety and productivity at this important infrastructure development.

Keywords: Stability; case study; tunnel.

NUMERICAL MODELING OF JOINTED ROCK MASS: A PRACTICAL EQUIVALENT CONTINUUM MODEL

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The paper deals with numerical implementation of Practical equivalent continuum model, using Fast lagrangian analysis of continua both in 2D and 3D using *FLAC* and *FLAC3D*, respectively. In this model the jointed rock properties are represented by a set of empirical relationships that express the properties of the jointed medium as a function of the properties of intact rock and joint factor along with confining pressure dependent Duncan and Chang hyperbolic model. Model, is implemented in *FLAC and FLAC3D* using *FISH* functions available in respective codes. The corresponding verification is based on simulation of triaxial compression tests by comparing the experimental and numerical results. Fig. 1(a) shows the axi-symmetric specimen and *FLAC3D* specimen for numerical simulation. Fig. 1(b) shows the explicit models for axi-symmetric and *FLAC3D* analysis with 3 joints. In *FLAC* the cylindrical specimens are considered as axi-symmetric problem where as in *FLAC3D* as 3-dimensional problem. The results obtained from the numerical models, match well with the experimental results (see Fig. 2b). The equivalent continuum results are also compared with explicit modeling of jointed rock specimens and the results were in close agreement (see Fig. 2a).

Keywords: Numerical model; FLAC; Joint factor; Equivalent continuum; Explicit model.

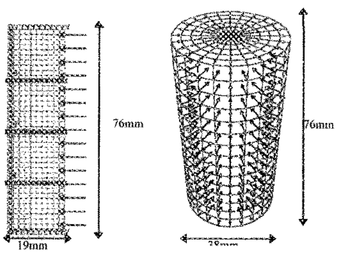

Fig. 1(a)

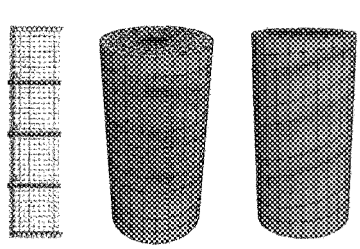

Fig. 1(b)

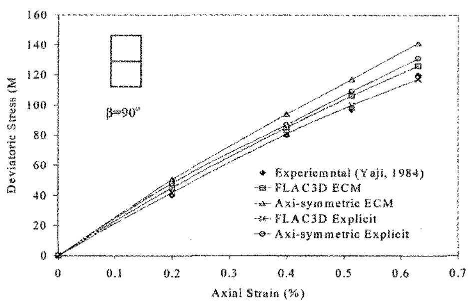

Fig. 2(a)

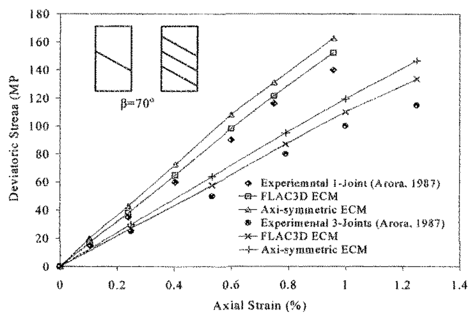

Fig. 2(b)

APPLICATION OF EXTENDED FINITE ELEMENT METHOD TO CRACKING ANALYSIS OF ROCK MASSES

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Discontinuities are very important for rock engineering as they directly affect safety, but numerical simulation of arbitrary discontinuities is always a difficult problem, especially for tracking dynamic crack. In the past, there are two methods to model crack growth, one is that crack growth direction is pre-assigned, the other is that computation region is re-meshed when crack grows. The above methods are apparently not very good for crack growth. For static discontinuities, the discontinuities are dependent on the mesh in the finite element analysis, so meshing is time-consuming. The extended Finite Element Method (XFEM) is a new numerical method for displacement discontinuity problems, it may conveniently model static and dynamic crack. The improved extended Finite Element Method, which can directly evaluate stress intensity factors without extra post-processing, is used to discretize the governing equations, allowing for the modeling of cracks whose geometry are independent of the finite element computation mesh. In the improved extended finite element method, the finite element approximation of the nodes surrounding the crack tip is enriched with not only the first term but also the higher order terms of the crack tip asymptotic field using a partition of unity method. The crack faces behind the tip(s) are modeled independently of the mesh by displacement jump functions. The analysis of crack growth is dramatically simplified with the improved extended finite element method. When crack faces are open, the crack faces are free. When crack faces are closed, the contact conditions are considered on the interface formed by the crack faces. First, one SIFs problem is presented to demonstrate the accuracy of the method, then two crack growth problems are simulated. Numerical simulations illustrate that the method can effectively model the crack growth and discontinuity problems, and it has prospective practical merits in rock engineering.

Keywords: Improved XFEM; jump function; asymptotic crack-tip displacement fields; crack growth.

A 2-D NATURAL ELEMENT MODEL FOR JOINTED ROCK MASSES

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Rock masses usually include a great number of joints, making the mechanical behavior of the rock mass complicated. The existence and behavior of joints in a rock mass is well known to control the mechanical behaviors of the jointed rock mass. Establishment of an analysis method for rock masses containing discontinuities is one of the most important issues in rock mechanics. Many numerical methods have been proposed, such as Discrete Element Method (DEM), Discontinuous Deformation Analysis (DDA), Finite Element Method (FEM), and several others. All these methods require discretization of rock masses into a great number of finite elements to achieve reasonable results. Consequently, mesh generation for these methods is a time-consuming task. Though the element-free Methods (EFM) have overcome the difficulty, the computation cost in the mesh-free methods is much greater than that in the finite element method; and the imposition of essential boundary conditions is quite awkward in the mesh-free methods.

The natural element method (NEM) is a new Galerkin method for the solution of partial differential equations. In NEM, the test and trial functions are constructed using natural neighbor co-ordinates. Natural neighbor co-ordinates are based on well-known geometric concepts such as the Voronoi diagram and the Delaunay tessellation. The Voronoi diagram and the Delaunay triangulation of the scattered nodes can be automatically formed with computer. NEM has advantages of both finite element method and mesh-free method, and avoids their disadvantages.

According to the characteristic structural features of jointed rock masses, a natural element method model is proposed for the mechanics analysis of jointed rock masses based on the natural neighbor interpolation. In the model, the jointed rock mass is regarded as a system of intact rock blocks connected by discontinuous joints modeled by interfaces. The displacement field in every rock block is constructed by an array of nodes through the natural neighbor interpolant, and the joint is modeled by the interface element. To deal with the discontinuities of rock masses, the displacement fields are constructed to be discontinuous between blocks. The displacement fields and their gradients are continuous in each block; hence no post processing is required for the output of strains and stresses. Numerical simulations illustrate that NEM has advantages of both high accuracy and simple data preparation. The present method which is developed for two-dimensional linear elastic analysis for jointed rock structures can be extended to three-dimensional and non-linear analysis.

Keywords: Block-interface model; natural neighbor interpolation; natural element method; jointed rock masses.

6.3. Field and Laboratory Studies

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PHYSICAL PROPERTIES OF FRACTURED ROCK MASS DETERMINED BY GEOPHYSICAL METHODS

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Fractures are most important factor which influence on physical properties of rocks. It is known that cracks cause velocity reduction, which depends on crack geometry and elastic properties of material filling cracks (water, gas or silt). Preferred orientation of crack systems involves a velocity anisotropy. The seismic wave velocity is smaller in direction perpendicular to a crack plane than in direction parallel to it. P-wave velocity anisotropy is bigger than S-wave velocity anisotropy. Saturation of cracks reduces P-wave velocity anisotropy.

Crack systems flow also on electrical conductivity of rock mass. Oriented cracks cause an anisotropy of rock resistivity. In this case, the conductivity has to be described by a tensor quantity. Electrical conductivity of rock shows extreme value in directions according to orientation of crack systems. For dry, unsaturated cracks the conductivity measured parallel to crack strike is minimum. For cracks filled with mineralized water this conductivity is maximum. It is possible to determine rock permeability knowing fracture features determined by geophysical methods what allows applying them in investigations of fractured aquifers.

The existence of relationship between crack and seismic or electric anisotropy allowed using geophysical methods for determination of fracture density and orientation of crack systems. The present study was carried out in several sites in the north - eastern border of the Upper Silesian Coal Basin, southern Poland. Seismic and geoelectrical measurements were carried out near the places where fracture parameters were measured.

Fracturing has an important meaning for the elastic and electric properties of these carbonate rocks. The present research allowed to establish the relationship between characteristic parameters of the tensors. Velocity tensors and electrical conductivity tensors were calculated and compared. The information allowed to find out mean direction of cracks sets and consider about fracture saturation. So we can say that geophysical methods, especially seismic and geoelectrical methods are useful in investigations of fractured rocks in sites where the rocks are inaccessible to direct observation.

Keywords: Fracturing; Carbonate rocks; Seismic anisotropy; Electric anisotropy; Velocity tensor; Electrical conductivity tensor.

ROCK MASS MECHANICS AT THE MINING OF LARGE ORE BODIES IN THE URANIUM DEPOSIT OF ROŽNÁ

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From the year 1945 to the half of the 90's of last century, uranium mining belonged to significant industries in the Czech Republic, and the Czech Republic was at the forefront of uranium concentrate production in the world. Nowadays, only the deposit of Rožná is exploited; the exploitation will be performed by the year 2008. The mining plan presupposes mining operations at the depth of up to 1200 metres below ground, i.e. under conditions that are, from the geomechanical point of view, markedly worse than the present-day conditions.

For deposit exploitation, high-efficient caving methods are preferred, specifically the method of "top slicing and caving under the artificial roof". This method ensures, under conditions of the deposit, a high productivity of labour together with a relatively high level of safety. To assess the stability of rock mass at the exploitation of uranium deposit of Rožná by this mining method, spatial mathematical modelling making it possible to forecast the behaviour of mass in the course of further advance in mining was used. The mathematical modelling of stress states was done to the final depth of reserves valuation, i.e. to the depth of up to 1200 metres below ground. As quantities characterising the development of stress in the rock mass, the vertical component of stress tensor, the maximum shear stress and the stability coefficient derived from the Hoek-Brown criterion were chosen and modelled. For the calculation of stress states, a programming system for solving extremely extensive spatial tasks by the finite element method was used; the system being developed in the framework of project HIPERGEOS at the Institute of Geonics of Academy of Sciences of the Czech Republic in Ostrava. As output from modelling, a graphical representation defining induced stress fields in the mass in individual observed years on cross-sections running through typical parts of the deposit (for examples of outputs see Figs. 1, 2 and 3) was chosen. Mathematical modelling together with practical experience from deposit mining provided a sufficient amount of findings to optimise the control of face front.

Keywords: Top slicing and caving; rock mass geomechanics; mathematical modelling.

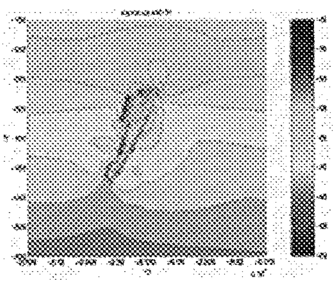

Fig. 1. Vertical component of stress.

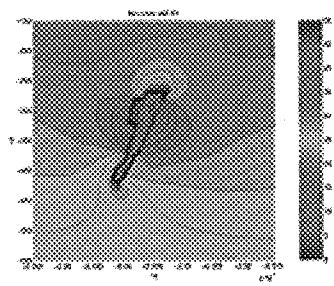

Fig. 2. Maximum shear stress.

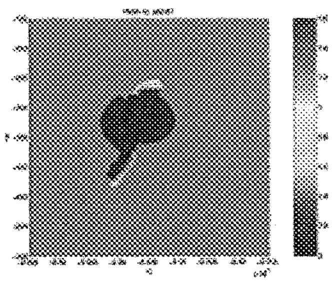

Fig. 3. Hoek-Brown criterion.

PREDICTION OF MODULUS OF ELASTISITY AND DEFORMABILITY OF ROCK MASSES FROM LABORATORY AND GEOTECHNICAL PARAMETER

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Cognition of deformability of rock mass is prime importance as it governs the behavior of the structure's surrounding rock. Usually rock masses are seldom intact and are dominated by discontinuities and means of anisotropy which affects their mechanical behavior. Based upon the degree and extent of anisotropy, scale, boundary condition and insitu structural setting the rock mass deformability values differ from the laboratory tested.

Studies on the laboratory test result with consideration of their structural nature would help the practicing engineering to save time and reduce the number of expensive field experiment. Farther more this would be helpful in estimating the field modulus at an early stage of a project.

Dilatometery method estimating deformability has been studied in this dissertation. Investigations were also made on the various parameters affecting the deformability.

Amongst the various method of estimating rock mass deformability the experimental method was selected to correlate the field and laboratory deformability values. Based on the studies made empirical equations are suggested to estimate the insitu rock mass elastic and deformability modulus considering their intact laboratory value, loading condition and rock quality designation.

Keywords: Modulus, Elastisity, Deformability, Geotechnical, Dilatometer.

SCALE EFFECT OF SHEAR STRENGTH OF CONGLOMERATE EVALUATED BY FIELD AND LABORATORY TRIAXIAL TESTS

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An *in-situ* triaxial test method was invented for accurate evaluation of stress and strain relationships of rock masses in the field. A series of proof tests with significantly large specimens of the diameter d=400mm were conducted at an abandoned quarry of Ohya stone. Furthermore, to investigate the influences of scale effect, drainage condition and heterogeneity, two series of laboratory triaxial tests, under consolidated/undrained (CU) condition and under consolidated/drained (CD) condition, were conducted on the specimens of diameters, d=20, 50, 65 and 100mm, retrieved by rotary drilling from the same site.

As shown in Figure 1, the strength characteristics evaluated by the *in-situ* triaxial tests were compared with those by the laboratory triaxial tests, and the following conclusions are drawn.

(1) The *in-situ* triaxial test is suitable to evaluate mechanical properties of heterogeneous rock masses, such as conglomerates whose maximum gravel diameter, D_{max}, is as large as several centimeters.

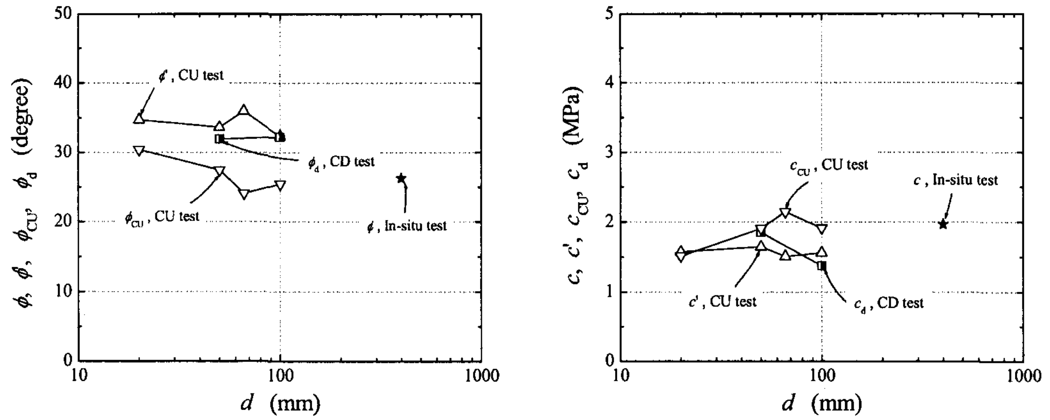
(2) The specimen of the diameter about twice the average gravel diameter, $d/D_{50} \doteqdot 2$, is too small for accurate evaluation of the strength of the conglomerate due to its heterogeneous nature.

(3) The principle of effective stresses holds good for Ohya stone.

(4) There appears to be no scale effect on strengths of Ohya stone using the representative specimens of diameter twice to tenfold of the average gravel diameter, d/D_{50}=2-10.

(5) It is estimated that the *in-situ* triaxial tests on Ohya stone were conducted under the condition close to the drained one.

Keywords: Shear strength; Triaxial test; *In-situ*; Rock; Scale effect; Conglomerate.



(a) Relationship of ϕ and d (b) Relationship of c and d

Fig. 1. Strength parameters, angle of internal friction and cohesion.

EFFECT OF ROCK STRENGTH PROPERTIES ON BREAKAGE OF ROCK MASS: AN EXPERIMENTAL ANALYSIS OF INDIAN MINES

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The principal mechanism of rock mass breaking by explosives and the interaction between rock parameters and blast induced stress wave propagation is complex and still not unanimously explained and understood. Behavior of the rock mass under blast loading determines the blast results in terms of fragmentation. Therefore, it is important to scale down the relationship between rock parameters and degree of fragmentation.

There are several controversies over which rock mass properties influence the degree of fragmentation. Many authors noted that blasting is strongly influenced by the Uniaxial Compressive strength (σ_c). On the contrary, a unanimous opinion about the influence of tensile strength on fragmentation was established in some cases. Many researchers did not find correlation between Average Fragment Size (K_{50}) and few strength properties in their blast experimentations. Hino (1909) proposed first quantitative approach of evaluating blasting results with rock properties. He defined "Blastability Index" as ratio of Uniaxial Compressive Strength (σ_c) to Tensile Strength (σ_t), which indicates ease in blasting. He said that upper limit of rock mass strength is the compressive strength and lower limit is tensile strength. Honma (1993) did the experimentations on laboratory models by categorizing rocks into soft and hard but his results were contradicting to Hino's finding.

Considering the above controversies, the objectives of the study was set to determine correlation between rock strength properties and fragmentation in various rock formations in general and Indian rock formations in particular. It was also intended to verify the diversified concepts of "Blastabilty" proposed as regard theory of fragmentation is concerned. These results may be useful to standardize the blast designing parameters for mines and construction industry. Experimental work has been carried out into two phases. In the first phase, rock properties were determined in Rock Mechanics Laboratory and Average Fragment Size (K_{50}) was obtained by quantifying fragmentation characteristics during full-scale blasts. Seven mining sites were identified for experimentations covering sandstone, limestone, granite, quartzite, haematite etc. rock formations conducting 30 experimental full-scale blasts. Sumptuous data was collected and statistically analyzed.

Results of the experimentation show that different kinds of trend of correlations of strength properties and fragmentation exist and they basically depend on the soft and hard formations. In softer rocks, compressive strength is inversely proportional to degree of fragmentation whereas it is directly proportional in hard rock formations. Tensile strength is directly proportional to fragmentation in both the formations. It shows that both the strength properties play important role in fragmentation of rocks. While considering Hino's "Blastability Index", no correlation was found with fragmentation. Therefore, authors suggest Modified Blastability Index as (σ_c- σ_t) replacing the original term ($\sigma_{c/}$ σ_t) in Hino's 'Blastability Index' and may be called as 'Modified Blastability Index'.

Keywords: Rock strength; rock mass; mine.

STUDY ON DESIGN SCHEME FOR CONTROL OF SEEPAGE OF PINGTOU UNDERGROUND HYDROPOWER PLANT

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Based on the numerical procedure of nodal virtual flux with fixed mesh for solution to problems of unconfined seepage in inhomogeneous and anisotropic media, the improved and theoretically strict drainage substructure technique for simulating the complex behavior of seepage caused by densely distributed drainage holes and the technique of drainage switch for the identification of the real state of each overflow type drainage hole are presented, the behavior of the 3D seepage in rock masses surrounding the openings of Pingtou underground hydropower station with 3 × 60 MW generator sets is numerically simulated with finite element method in detail. The behaviors of seepage relevant to all the galleries, the impervious grout curtains, and the numerous overflow or exit flow drainage holes arranged in multi-layered around the openings are strictly simulated, particularly for the densely distributed overflow and exit flow drainage holes which play a key role in water discharging and pressure relief. The geometries and distance between the drainage holes can be exactly simulated and the results are robust and precise enough for the application to the practical project. The strongly inhomogeneous and anisotropic characteristics of seepage of the rock masses, the high hydraulic pressure in the diversion tunnel, the effect of the peripheral grout curtains, and other factors are taken into account in detail. The locations of the exit lines of seepage on the slope surfaces of the calculation domain, on the internal boundary surfaces of the exit flow drainage holes and the other openings, and the location of the free surface of seepage are finely iteratively calculated and determined. Furthermore, during the iterative solution, the real states of seepage of the overflow drainage holes arranged in the bottom gallery are also strictly distinguished in theory and algorithm. After the analysis of nearly 20 calculation cases about the behavior of seepage and researching for the optimal design scheme for the control of seepage, the whole property of seepage and the respective advantages and disadvantages of the present suggested anti-seepage and drainage measures are comparatively analyzed in detail, the optimal design scheme for the control of seepage is proposed.

Keywords: Seepage; procedure of nodal virtual flux; drainage holes; drainage substructure; optimal design scheme for seepage control; Pingtou underground hydropower plant.

7. ROCK PROPERTIES

7.1. General

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ESTIMATION OF GEOMECHANICAL PARAMETERS OF RESERVOIR ROCKS, USING CONVENTIONAL POROSITY LOG

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In recent years, rock mechanic gained great interests in petroleum industry. Geomechanical properties of reservoir rocks have an important role on designing of drilling, stability of well bore and planning for production. Rock mechanical properties usually calculate based on direct laboratory tests on cores or using sonic logs. The study of cores is not always possible or even feasible. Cores of limited parts of drilled section are normally available. Coring is expensive and destructive geotechnical test on cores, which are valuable documents, is normally prohibited. This paper presents a new method for indirect measurement and estimation of the rock geomechanical properties, using sonic logs and porosity data. Based on the study of data from three wells, drilled in different formations of three oil fields, in southern Iran, some relations with good correlation coefficients, have been developed. Derived relations can be used to present a continuous picture of the mechanical behaviors of the drilled section.

Keywords: Oil and gas wells, Geomechanical properties, Porosity, Sonic logs, Iran.

PREDICTION OF MECHANICAL PARAMETERS OF ROCK, USING SHEAR WAVE TRAVEL TIME

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Different methods have been used for downhole measuring of formation's shear wave travel time. These include visual inspection of full wave train recordings (e.g. variable density micro-seismogram), monitoring the amplitude characteristic of acoustic shear wave in selected gate(s), analysis of digital sonic data and comparison of compressional and shear wave transient time at various transmitter–receiver spacing and defining some experimental correlation between them. All these techniques have limited application, especially in the case of formations with complex lithology (combination of sandstone, dolomite, limestone, and anhydrite) and fluid content. This article presents an analytical approach for lowering the level of uncertainty in using above mentioned methods. The main objectives of present research are: reducing the level of uncertainty for different parameters and formation fluid contents, overcoming some of the limitations of existing methods and predicting mechanical parameters of drilled rocks. This has been done by utilizing the standard compressional travel time in conjunction with supplementary none-acoustic log data and petrophysical full logs interpretation results. Presented method can estimate shear wave travel time and some rock mechanical parameters with high accuracy for different lithology, especially for limestone-sandstone or limestone-dolomite-shale sequences.

Keywords: Shear wave travel time, Sonic log data, Petrophysical data, Mechanical properties of rock.

ROCK STRAIN-STRENGTH CRITERION AND ITS APPLICATION

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In order to be able to easily and quickly interpret results from measured displacements in underground excavations, it is more desirable to perform strain analyses instead of stress analyses. A rock strength criterion in term of strains is one of the key issues for applications of such strain analyses to determine the extent of plasticity in the rock. This paper presents a new and simple criterion for rock strength in term of strains. This is a general criterion expressed in volumetric strain in relation to axial strain as written as follows:

$$\varepsilon_v = \kappa \cdot \varepsilon_1 - \varepsilon_c \tag{1}$$

where ε_v is the volumetric strain; ε_1 is the major principal strain; κ and ε_c are constants. The strains are defined as positive for compaction. This strain criterion is represented as a triangle in the π-plan in the principal strain space.

The compilation of the published data of triaxial tests conducted on various rocks indicates that the envelop of the maximum compaction can be represented by a linear correlation between the volumetric strain and the major principal strain. In other words, Eq. (1) has supporting evidences from the testing data.

This paper presents also a method to convert the Mohr-Coulomb strength constants, namely cohesion C and friction angle ϕ to the strain-strength constants κ and ε_c by assuming that the stress-strain relationships before failure can be expressed by Hooke's laws. By means of this method, the strain-strength constants for various rock types can be evaluated from the existing data of rock strength in term of cohesion and friction angle.

The theory of plasticity formulated in the strain space gives the following expression of flow rules:

$$d\sigma_i^p = \eta \frac{\partial \varphi}{\partial \varepsilon_i} \tag{2}$$

where $d\sigma_i^p$ the so called increment of stress relaxation; φ is the yield surface in the strain space and η is a scalar factor to be determined by the consistency condition. Formulation of constitutive models in accordance with the theory of plasticity formulated in the strain space is briefly described in the paper.

Finally discussions are given to the application of the proposed strain-strength criterion. For engineering purposes, a new approach named as "strain path analysis" is described and a concept called "initial strain ε_o" is introduced. The method can be easily applied to estimate the extent of plasticity in the rock for underground excavations where displacements are monitored.

Keywords: Rock strain-strength, strain path, criterion, rock engineering, constitutive model.

THE EFFECT OF CALCIUM CARBONATE CONTENT OF MARLSTONES ON THE STRENGTH RESPONSE

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Marlstones are one of the most problematic geotechnical materials in their response to the excavation of the underground openings. There are numerous underground opening projects which are partially or wholly dominated by the marlstones. Therefore cognition of the behavior and strength response of the marlstones is important in the design and execution of such projects. The marlstones are comprised of the indurated clay and calcium carbonate. The variation in calcium carbonate content would result in a wide variation of strength response of the marlstones. Initially several sets of laboratory tests were performed to evaluate the rate of the calcium carbonate content of the marlstones samples. Further, the strength response of the marlstones specimen was evaluated with respect to the percentage variation of calcium carbonate content. Some equations are presented based on the results of the regression analyses. Results of this research work indicate that the increase in the percentage of the calcium carbonate would increase the compressive strength and the modules of elasticity of the marlstones. It was also found that poison's ratio is not affected by the percentage of calcium carbonate. The stress-strain curve of Marlstone showed that increase in the percentage of calcium carbonate is accompanied by decrease in the initial nonlinear part of the curve (which occurs as a result of the compression of small cracks in the rock) and finally this region becomes completely linear.

Keywords: Calcium Carbonate; Marlstones; Strength Response.

ANALYSIS OF STRUCTURE PROPERTIES AND LOAD CARRYING OF DESTRUCTIVE ROCK UNDER DIFFERENT CONSTRAINTS

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Through uniaxial, triaxial and hoop constraint tests, forty-eight standard rock samples (Φ50 mm × 100 mm) are carried out to study the structure characteristics and load carrying ability of destructive rock by MTS-815 servo-controlled testing machine.

The original rock materials (siltstone, limestone and packsand) are taken from the underground in coal field. The lateral compressive pressure under triaxial tests is 5.0 MPa, 10.0 MPa and 15.0 MPa. The confining stress under hoop constraint tests is about 20 MPa, 40 MPa and 80 MPa. Under uniaxial tests, the macroscopic failure of rock samples mainly exhibits the splitting failure nearly parallel to the axial direction but there are five different failure modes for different rock samples. It is found that shear fracture and transverse crack also exist in the different rock failure processes. As for the triaxial tests, it is common that the peak strength and residual stress increase with the lateral compressive pressure. However, different kinds of rock samples show different failure characteristics. Four failure modes happen in the triaxial experiment results. Under hoop constraint tests, the axial stress exhibits obvious stress hardening characteristic with the increase of the hoop confining pressure after peak-load point. It can reach and keep at a high axial stress state ultimately. The confining pressure increases linearly with the increase of axial stress and reaches the yielding stress state.

The distribution of rock blocks is different under different constraint tests. The degree of fragmentation can be reflected by the variation of fractal dimension. The fractal dimension increases approximately linearly with the increase of confining pressure, which indicates that the rock blocks produce further failure or secondary cracks between them under the high confining pressure. To sum up, with the increase of constrained stress, not only do the tension cracks and controlling shear cracks exist, but also secondary cracks form and propagate along the controlling shear crack surface within the destructive rock. And all the crack surfaces would develop into a grid shape finally, which leads to the gradual reduction of the mechanical properties of the rock mass and the gradual lost of its integrity.

Therefore, the maintenance of the cracked rock mass lies in the effective constraint, which indicates that the basic requirement to keep the stability of surrounding rock in underground engineering is to form an effective support structure.

The study is supported by NSFC major program (No 50490273).

Keywords: Cracked rock mass; Constraint state; Failure characteristics; Structural effect.

CHARACTERISTICS OF ROUGHNESS MOBILIZATION

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To investigate the failure mode and roughness mobilization which depend on the asperity scale and the applied normal load, analytical and experimental approaches are performed with artificial triangular asperities. In this study, a single triangular asperity model is proposed and joint shear strength tests are carried out. In the shear test, the specimens are prepared representing waviness, unevenness, and total (superposition of waviness and unevenness) roughness based on the asperity scale. Artificial stone with high unconfined compression strength and gypsum with low unconfined compression strength are used. Model analysis shows that the size of the shearing asperity and the shear mode are determined by the normal stress, the strength of material, the shear displacement, the unevenness scale, and the inclination angle. Shear test results (see Fig. 1) show that roughness (JRC*: Joint Roughness Coefficient with no basic friction influence) mobilization depends on the applied normal stress (σ) and scale of roughness. The roughness value changes nonlinearly, and the shear modes are classified into 3 stages with the increase of normal stress: sliding (1st stage); asperity failure (2nd stage); and asperity base failure stage (3rd stage). It is inferred that the sliding stage mainly consists of the geometrical component, and the asperity base failure mostly consists of the mechanical component. The asperity failure stage consists of both components. However, shear mode is also dependent on the scale of asperities. For small size asperity, the transition from the geometrical component to the mechanical component is abrupt; for large size asperity, the transition is not so abrupt. Therefore, the roughness coefficient is not constant and is a nonlinearly varying property with the asperity size and the applied normal stress. These effects should be considered in rock joint roughness quantification.

Keywords: Roughness mobilization; roughness components; roughness scale; artificial triangular asperity; shear mode; joint shear test.

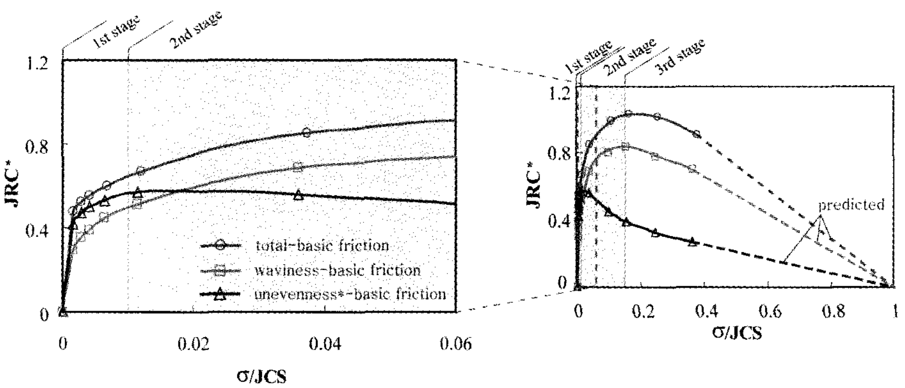

Fig. 1. JRC* versus applied normal stress relationship.

BRIEF ROCK EVALUATION BY SHOCK RESPONSE VALUE AND MRCI

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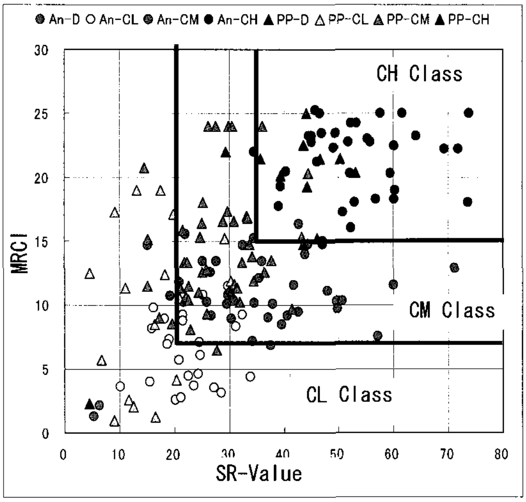
Crack frequency and strength of rock is one of the most important factors for geotechnical evaluation of rock for dam design and construction. As for rock crack frequency, RQD is very popular index and usually measured for boring core, it is seldom measured in adit or natural outcrop. As for rock strength, unconfined compressive strength and degree of repulsion of Schmidt Rock Hammer is brief and useful index. However, measurement of the former is rather expensive and that of the latter can not be performed in borehole. Therefore, the very important factor, crack frequency and strength of rock, is not systematically integrated for dam design and construction.

In this paper, MRCI and Shock Response value is found to be useful index of crack frequency and strength of rock, respectively, by comparing with Rock Class in adits and in boreholes of a dam site in Northern Hokkaido, Japan, of which geology is Neogene andesite and peperite. Result of adit is shown in Fig. 1. Each Rock Class is well separated by the two indices. When MRCI is more than 15 cm and SR-Value is more than 35 m/s^2, rock is classified into CH Class. When MRCI is less than 7 cm or SR-Value is less than 20 m/s^2, rock is classified into CL Class. When rock condition is between CH and CL Class, rock is classified into CM Class. These classification criteria are also found to be applicable for boring data without lessening the accuracy so much. Therefore, rock evaluation in both borhole and in adit is briefly and objectively integrated by measuring these two indices.

Keywords: Rock Evaluation; SR-Value; MRCI.



An: Andesite
PP: Peperite

Black line: Boundary of CH and CM
Blue line: Boundary of CM and CL

Fig. 1. Relationship between Rock Class, MRCI and SR-Value.

THREE DIMENSIONAL THERMO-HYDROMECHANICAL MODELING OF A HEATING TEST IN MUDSTONE

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In the context of feasibility study for high level and long lived radioactive waste disposal in geological formation, the French national radioactive waste management agency (Andra) has conducted a series of experimental investigations and numerical simulations in order to evaluate thermo-hydro-mechanical disturbances in engineered and geological barriers, due to excavation and heating generated by vitrified waste. For this purpose, during recent years, an extensive investigation with in situ thermal experiments has being performed by Andra in the both underground research laboratories in Bure (France) and Mont-Terri (Switzerland). The first in situ THM test, called HE-D experiment was conducted in Mont-Terri site from 2003 to 2005. Rock formation was heated through a heating packer installed inside a borehole. Variations of temperature, pore pressure and relative displacements, have been measured during the test at different positions around the borehole. This paper presents three dimensional coupled thermo-hydromechanical modeling of this in situ experiment using FLAC-3D code. In the first part, we shortly present geological situation and test conditions of the HE-D experiment. Then, the general framework for modeling coupled thermo-hydro-mechanical problems is given, together with the outlines of the elastoplastic model used for the description of rock behavior. The mechanical behavior of mudstone is simplified and described by an isotropic elastoplastic model based on the Mohr-Coulomb criterion. In the third part, we present the numerical simulations and comparisons with in situ measurements. In the present work, only the anisotropy of heat conductivity is taken into account. The effect of pore pressure on plastic flow is taken into account using the classic effective stress concept.

It seems that the numerical predictions are in agreement with measured data for main features observed. The temperature distribution is mainly controlled by thermal properties. There is a strong coupling effect of temperature change and stress change on the evolution of pore pressure. A coupled thermo-hydromechanical modelling is then necessary. The over-pressure generated by excavation and heating process may be an important factor for the evolution of damaged zone in rock formation. Using a simple isotropic elastic perfectly plastic model, there is quite large scatters between predicted and measured data in terms of deformation. More advanced constitutive model may be necessary to improve the numerical modelling.

Keywords: Thermo-hydromechanical; mudstone; radioactive waste disposal.

ELEMENT FREE ANALYSIS
FOR A MATERIAL HETEROGENEITY: 2D EXAMPLE

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Material properties of Geomechanics are naturally heterogeneous. Due to a limitation in the number of measurements as well as errors in measured data, a model constructed in terms of these heterogeneous material properties should contain an uncertainty. In this paper, the stochastic Element Free Galerkin Method (EFGM), which has been proposed as an alternative analysis tool to estimate an uncertainty in a model with heterogeneous material property, is extended and applied to a 2D elasto-static beam problem. The stochastic EFGM can be distinguishable by its ability to evaluate a material heterogeneity-induced uncertainty with comparatively less efforts, by the fact not to require a complex nodal connectivity, and by a simplicity in modifying and updating analysis conditions in comparison with the conventional analysis represented by a finite element analysis. Within the present stochastic EFGM, both a perturbation and a spectral formulation have been implemented and tested. Karhunen-Loeve expansion is used for a material parameter decomposition, and a 2nd order expansion and a Polynomial Chaos series expansion are used in the perturbation and the spectral stochastic formulation, respectively. Direct comparison between the perturbation and the spectral method under identical computational conditions (: computing machine and code) has been accomplished, and general understanding on the characteristics of each method has been discussed.

Keywords: Uncertainty; stochastic; Element Free Galerkin Method (EFGM); Karhunen-Loeve expansion, Polynomial Chaos series expansion.

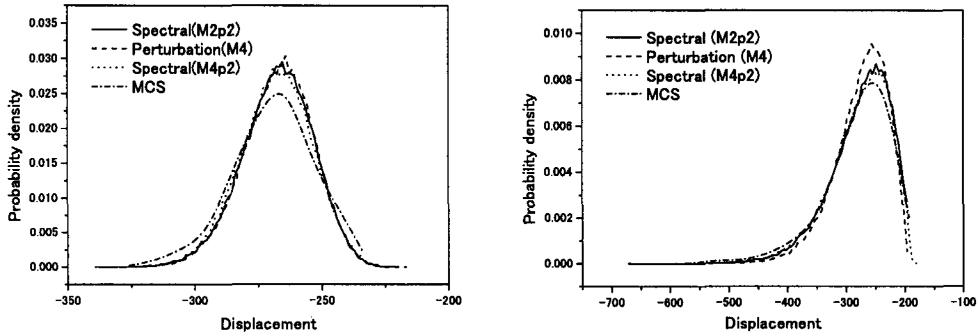

Fig. 1. Probability density distribution of computed displacement at the free end (: $x = 48$) of beam, (left: when λ_E =10 and $\sigma_E = 0.1$, right: when λ_E =50 and $\sigma_E = 0.2$).

IMPACT OF PYRITE OXIDATION ON MECHANICAL PROPERTIES OF ROCK AND ENVIRONMENT

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Pyrite is a common and abundant sulfide mineral and it is oxidized upon exposing to oxygen on earth surface. Oxidation of pyrite (FeS_2) generates acid drainage resulting in acceleration of rock weathering and discharge of heavy metals into environment. The accelerated weathering of rocks can reduce compressive strength and slope stability. We examined the changes of the physical properties of rocks and the chemical composition of drainage using a soxhlet extractor for one month. Three groups of biotite gneiss were used for the soxhlet extraction experiment (group A: < 0.1% of pyrite; group B: about 5% of disseminated pyrite; group C: about 6% of vein type pyrite). Group A and B showed no significant quick absorption ratio after one month experiment but group C had about 10% increased value. The uniaxial compressive strength of the three groups decreased about 14%, 10% and 45% for group A, B and C, respectively. The pH of drainage for group I was increased from 5.8 to 9 during the soxhlet experiment but the pH for group C decreased from 6 to 3. The pH for group B decreased to 5.0 for first one week and then increased to 9. The electrical conductivity (EC) and the concentrations of SO_4^{2-} and Fe of the drainages were continuously increased for the all groups. The EC and the concentrations of SO_4^{2-} and Fe of the drainage for group C was much higher than those of group A and B. The concentrations of heavy metals in the drainage for group A and B not exceeded the Korean water quality standard but the concentrations of Zn and Ni for group C were 1.3 mg l^{-1} and 2.4 mg l^{-1}, respectively. The iron oxide coating, signature of pyrite oxidation, was rarely observed at group A. For group B, the coating was observed only on the surface. For group C, the coating was observed on the surface as well as the inside of sample along the pyrite vein. The mechanical properties of the samples and the chemical compositions of the drainage indicate that the oxidation of pyrite contained in the samples accelerated weathering resulting in deterioration of mechanical properties of rocks and could discharged heavy metals and acid into environment with the drainage.

Keywords: Acid rock drainage; Weathering; Uniaxial compressive strength; Heavy metals.

CHARACTERISATION OF MARBLE AND EFFECT OF HIGH CONFINING PRESSURE

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A number of civil engineering projects like river valley projects and transportation projects are under construction in India. The structures like dam foundations, slopes, underground openings, powerhouse caverns are increasingly located in/on rock formations. The rocks are very complex material especially when exist in combination with joints/discontinuities. It is therefore, very important to understand the behavior of rocks for the rational design of the structures constructed on/in them. Many rocks generally show strain softening behavior under loading, mostly in tunnels and underground caverns. The strain softening is defined as progressive loss of strength when the material is compressed beyond peak/failure. Hence the use of strain softening behaviour is of utmost importance in the rational design of tunnels and underground structures. The behavior is characterised in the laboratory using stiff testing machines and closed-loop servo-controlled testing machines as it is not possible to capture the strain softening behaviour under stress controlled loading using conventional machines. The analysis of underground structures is normally carried out through numerical methods like FEM. The constitutive models which characterize the rock behvaiour are the integral parts of the numerical methods. Various constitutive models based on theory of elasticity, elasto-plasticity and elasto-viscoplasticity are used to characterize the rock behviour.

The paper presents the characterization of marble under unconfined and varying confining pressures and back prediction with constitutive model based on elasto-plasticity. The effect of high confining pressure on rock behaviour is also presented in this paper. The marble from Rajnagar, Rajsamand district of Rajasthan (India) has been used for characterisation. The rock belongs to Aravalli Supergroup from Middle Proterozoic age. The rock is fine grained and white in colour. The index properties of the rock have been found out as per ISRM. The specimens with 54 mm diameter and 108 mm height as per ISRM have been used. The experiments have been conducted under confining pressures of 0 MPa, 10 MPa, 20 MPa, 30 MPa and 46 MPa using closed-loop-servo controlled testing machine at a constant strain rate of 9.26×10^{-6}/s. The strains were measured with the help of strain gauges and specially designed extensometers. The specially designed high pressure cell with capabilities of applying 0-140MPa confining pressure has been used.

The effect of high confining pressure is very clearly noticed on the rock behaviour. With the increase in confining pressure there is marked increase in the residual strength and eventually at confining pressure of 46 MPa the rock behaviour changes from softening to hardening.

The paper also focuses on the back prediction of stress-strain-volume change response of marble under unconfined state and varying confining pressures using constitutive model based on elasto-plasticity. The predicted results are found out to be quite satisfactory and comparable with the experimental results.

Keywords: Strain softening; constitutive modeling; elasto-plasticity; FEM.

BEHAVIOR OF A SANDSTONE UNDER AXI- AND ASYMMETRIC COMPRESSIVE STRESS CONDITIONS

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Samples of a fine-grained sandstone were tested under uniaxial compression (UC), conventional triaxial compression (CTC) and true triaxial compression (TTC) conditions. The CTC tests were carried out at confining pressure equal to 12.5, 25.0, 37.5, 50.0 and 62.5 MPa. In the TTC tests the minimum principal stress (σ_3) was equal to 25.0 MPa and the intermediate principal stress (σ_2) was 1.5, 2 and 2.5 times higher than σ_3, or the intermediate principal stress was equal to 62.5 MPa and the minimum principal stress was equal to $0.4\sigma_2$, $0.6\sigma_2$ and $0.8\sigma_2$.

As a result of the experimental studies carried out, the threshold of absolute dilatancy, ultimate strength, stress at faulting, stress drop accompanying faulting, ductility, volumetric strain at the threshold of dilatancy and volumetric strain at the peak stress have been determined and dependence of these characteristic stress levels and strain quantities on confining pressure, intermediate principal stress and minimum principal stress have been established.

Under conventional triaxial compression conditions, an increase in confining pressure resulted in a strong increase in pre-dilatant compaction, in threshold of dilatancy, in pre-peak ductility and in ultimate strength. After strength failure of the rock material, samples continued to deform, at a slightly decreasing differential stress, without gross-fracturing, while at the same time undergoing a significant increase in volume. Faulting occurred in a well-advanced post-peak region. It was accompanied by a small immediate stress drop. No audible acoustic effect could be detected.

The behavior of rock samples tested under true triaxial stress conditions was totally different. Contrary to the effect observed when confining pressure was increased in conventional triaxial compression tests, increasing the intermediate principal stress led to a decrease in the ductility of the rock material. The effect of dilatancy was also strongly hampered. Rock samples behaved in a highly brittle manner and underwent faulting just at the peak differential stress or at a very early post-peak stage. Faulting was very violent, accompanied by a release of a great amount of the elastic strain energy, a very strong acoustic effect and a large stress drop.

The effect of intermediate principal stress and minimum principal stress on ultimate strength and threshold of absolute dilatancy is similar; both σ_2 and σ_3 cause some increase in these two characteristic stress levels. However, the effect of σ_2 and σ_3 on the deformational properties is different. While an increase in σ_2 causes the rock to behave in an increasingly brittle manner, an increase in σ_3 causes, just as confining pressure does, an increase in ductility. Higher ductility manifested itself also in smaller values of the faulting ratio and smaller immediate stress drops during faulting.

Mogi's empirical failure criterion, which is a generalized Mises criterion, was found the most appropriate to fit all the triaxial strength data.

Keywords: Dilatancy; ductility; failure criterion; stress drop; triaxial strength; true triaxial compression.

AN INVESTIGATION OF HYDROMECHANICAL BEHAVIOUR AND TRANSPORTABILITY OF ROCK JOINTS

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In recent years, the management of radioactive waste becomes an important environmental issue in the countries operating nuclear power plants. Geological disposal is considered to be the most promising option for a safe long-term management of radioactive waste, which requires the understanding of the physical, mechanical, hydraulic and thermal properties of the host rock masses of a radioactive waste disposal facility. The hydromechanical behavior and transportability of rock fractures can be investigated on a single fracture and also on the scale of a fractured rock mass that contains many fractures. Obviously, the behavior of single fractures must be thoroughly understood before the behavior of fractured rock masses can be understood. In this study, a new hydromechanical test apparatus for rock joints was developed, which supports the shearing process under both constant normal load (CNL) and constant normal stiffness (CNS) boundary conditions with the hydraulic tests at the same time. A number of shear-flow coupling tests were carried out by changing the surface characteristics (i.e. contact ratio and surface roughness) of two smooth parallel surfaces to evaluate the influence of morphological properties of rock fractures on their hydromechanical properties and transportability. Hydraulic simulation by using the Finite element method (FEM) was carried out to evaluate the flow in rock joints with the same surface characteristics as the experimental ones, in which the parameters such as the relationship between hydraulic aperture and mechanical aperture were obtained from the hydraulic tests. By comparing the results of the hydraulic tests, shear-flow tests and numerical analysis, several parameters and relations about hydromechanical behavior of single rock joints are proposed.

Keywords: Hydromechanical; Rock fracuture; Shear-flow coupling test; FEM; Transimissivity.

EVALUATION OF THE STATE OF STRESS IN THE VICINITY OF A MINE DRIFT THROUGH CORE LOGGING

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A mine drift was located at a depth of about 1000 m. The vertical and horizontal stresses within the cross section of the drift were estimated about 30 and 40 MPa, respectively. The ratio of the uniaxial strength of rock to the horizontal stress was in the range from 1.5 to 3.0. Exploration drilling was conducted in the drift. Core mapping was carried out to a few selected boreholes drilled in the drift wall. The mapping was similar to RQD (Rock Quality Designation) logging, but all discontinuities were accounted in calculating the index no matter whether they were original or were created afterward. The index obtained in such a way is called FRQD in this paper. The general trend for the variation of FRQD along a borehole drilled in the drift wall is as follows: FRQD was relatively low in the section from the wall to a certain depth. It became then zero due to heavy core discing in a stretch of a few metres. Beyond that the cores became less fractured but the magnitude of FRQD was still small. FRQD increased to its normal magnitude only when the distance from the wall was beyond 1-1.5 times the height of the drift.

On the basis of the results of core mapping, the country rock surrounding the drift is divided to three zones: failure zone, highly-stressed zone and normal zone, Fig. 1. In the failure zone the rock is de-stressed due to the intense fractures in the rock. The highly-stressed zone is located from 0.5 to 1 times the drift height in the wall. The stress is significantly elevated in this zone so that core discing occurs when drilled. Beyond a depth of about one drift height it is the normal zone where the rock is not much disturbed, from the engineering point of view, by the excavation. The core discing marks the position where a pressure ring is formed in the rock around the opening. The pressure ring forms a shield over the opening. The rock within the zone of the pressure ring is intact and can carry a considerable ground pressure. The failure zone located inside the pressure ring has a low load-bearing capacity, but it provides a certain confinement to the pressure ring. This leads to a philosophy for rock reinforcement, that is, the failure zone should be reinforced by bolts and/or shotcrete in order to prohibit the failed rock from disintegration. Collapse of the failure zone would result in failure of the rock within the zone of the pressure ring so that the pressure ring moves into a deeper location. It is believed that a good reinforcement effect would be achieved by anchoring rock bolts into the zone of the pressure ring. The significance of this study is that the identification of the different zones provides a good base for rock reinforcement design.

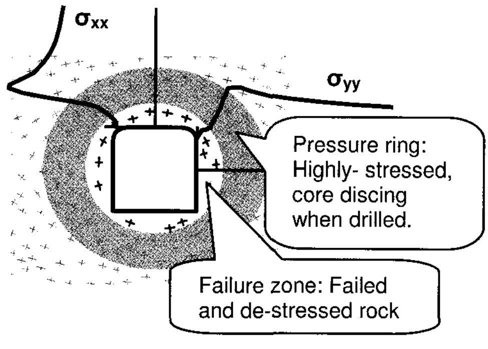

Fig. 1. Concept of the pressure ring formed in the rock surrounding an underground opening.

Keywords: Pressure arch; Rock stress; Rock stabilization, Disturbance zone.

THEORETICAL AND EXPERIMENTAL ANALYSIS ON THE MECHANISM OF KAISER EFFECT OF ACOUSTIC EMISSION IN BRITTLE ROCKS

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Kaiser effect is formally described as the absence of detectable acoustic emission (AE) events until the load imposed on the material exceeds the previous applied level. This phenomenon is usually used to estimate geostress. The common method for estimating geostress using the Kaiser effect can be depicted as follows. Take samples of six oriented cores from original rockmass, then uniaxially load them in laboratory and the AE activities are monitored. The AE onset stresses are arbitrarily considered being the previous peak stresses in the orientation. In reality, the cores are always subjected to multiaxial stress in original rockmass, but it is suffered uniaxial loading in the laboratory tests. The two stress states in the cores are different. There are arguments about the validity of the estimated-geostress extrapolated from uniaxial loading.

Rock is a typically heterogeneous material because it contains numerous microcracks and has very complex components. Therefore, rock is damaged by microcracking continuously from the early stage of low stress levels up to the final stage of macro-fault and accompanied with AE events. Based on statistic damage mechanics, the rock can be divided into many micro-elements with different strength. When rock suffered first loading (not reach the failure level), only a part of elements whose strengths are relatively lower would be destroyed. And no new failure will occur until the loading achieves the first loading level in the second loading cycle. Because AE events generate from the failure of elements, this is the real mechanism of Kaiser effect when the material under cyclic loading with the same path.

As for the application of Kaiser effect, we must distinguish the material strength and the supporting capacity of rock. Actually, material strength and supporting capacity are two different characteristics of rock specimen under multiaxial loading. Material strength is an intrinsic characteristic of a material and will not be changed by external conditions. Contrarily, supporting capacity is variable with different external conditions. Considering in conventional triaxial state, the axial supporting capacity of rock specimen increases along with the increasing of confining pressure, in this case we can say that the supporting capacity of specimen is controlled by material strength and confining pressure simultaneously. Therefore, if the rock have suffered loading with two different paths, such as uniaxially loaded in the first cycle and multiaxially loaded in the second cycle, the AE onset stress in the second cycle is predominantly variable to the previous peak stress. The results of experiments also verified this phenomenon.

In conclusion, it is more reasonable to take the AE onset stress as a measure of damage in rock rather than as a measure of the previous peak stress and further investigation on the influence of damage introduced during triaxial compression on the Kaiser effect is needed. The conventional method for estimating geostress using Kaiser effect needs modifying for determining the geostress accurately.

Keywords: Brittle rocks; acoustic emission; cyclic loading; Kaiser effect; loading path; damage.

NUMERICAL MODELLING OF SIZE EFFECT OF SINGLE-EDGE-NOTCHED BRITTLE SPECIMENS SUBJECTED TO UNIAXIAL TENSION

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A three-dimensional model is used to simulate the failure of the single-edge-notched specimens subjected to uniaxial tension. Specimens of different sizes are modeled to study the size effect.

The heterogeneities of mechanical properties are taken into consideration in the model. Five numerical specimens consist of cube elements. The material properties in each element, such as elastic modulus and Poisson ratio, are randomly distributed throughout the specimens by following Weibull distribution function. An elastic damage constitutive law is adopted to analysis the fracture process of the specimens in tensile force. Unlike other numerical models, not only can the influence of the width of the specimen's sizes be studied, but also the heterogeneous nature of material in mesoscopic scale can be taken into consideration.

The tensile tests are undertaken by applying a constant displacement increment in a step by step style. Three-dimensional distributions of stresses and displacements during the fracture process are obtained. The crack propagation traces are much more complicated than two-dimensional results due to the heterogeneities existing in the specimens. From the nominal strength and complete stress-strain curves, the specimens show more brittle failure feature and the nominal strengths decrease as sizes increase. The simulated nominal strength correlated reasonably with the formula proposed by Bazant. However, the results indicated a trend which is better described by a negative exponential function.

Keywords: Crack propagation, numerical modelling, size effect, uniaxial tension test.

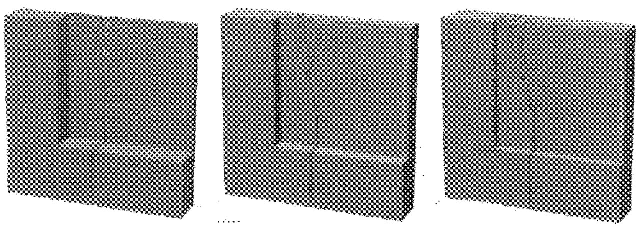
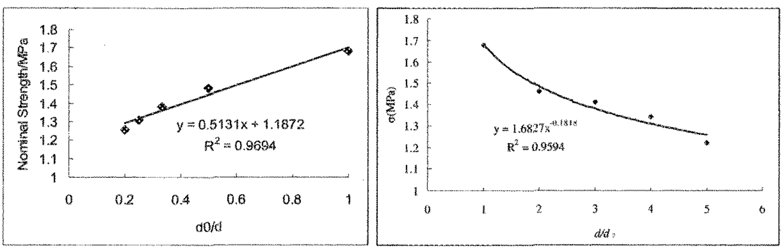


Figure 1. Propagation of three-dimensional crack at different width.



(a) (b)

Figure 2. Simulated size effect described by (a) Brazant formula and (b) Negative exponential function.

SITE INVESTIGATION FOR UNDERGROUND OIL AND GAS STORAGE ROCK CAVERNS AT JURONG ISLAND OF SINGAPORE

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A comprehensive site geological investigation program for underground oil/gas storage rock caverns is jointly planned and implemented by Tritech of Singapore and SINTEF of Norway in 2004. The project site was chosen based on a previous preliminary geological investigation performed by Nanyang Technical University of Singapore. The site is a sub-sea area underlaid with Jurong sedimentary rock formation close to reclaimed lands. A storage volume some 3-4 millions cubic meters was estimated feasible before the site investigation. The goal of the site investigation work is to provide geological and hydrogeologic information and rock mechanics data for the further feasibility study and basic design.

The program includes the following:
(1) Marine seismic reflection survey of 30 km
(2) Land seismic reflection survey of 10 km
(3) Horizontal directional drilling (HDD) of 6 boreholes with 400-450 m length each borehole
(4) Deep vertical drilling of 7 boreholes with 100-150 m depth each borehole
(5) Drilling core orientation and Rock Joint Statistics
(6) Borehole geophysical logging
(7) In-situ stress measurement of 8 points by hydraulic fracturing
(8) Laboratory rock mechanics tests and drillability tests
(9) In situ large-scale hydrogeologic testing and monitoring

This paper describes the methodology and major aspects of the investigation program including main outcome, with highlight of the special requirements of geological investigation for the purpose of design & construction of underground oil/gas storage rock caverns

Keywords: Site investigation, rock cavern, oil storage, HDD.

LITHOPHYSAL POROSITY EFFECT ON MECHANICAL PROPERTIES OF WELDED TOPOPAH SPRING TUFF

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About 81% of the proposed U.S. repository for the permanent disposal of high-level radioactive nuclear waste will be situated in the lower lithophysal unit of the welded Topopah Spring Tuff formation (Tptpll), Yucca Mountain, Nevada, and 4% will be in the upper lithophysal unit (Tptpul). Lithophysae or lithophysal cavities are a major feature in these units. The influence of the lithophysal cavities on mechanical properties of the welded tuffs was investigated. Seven cylindrical tuff specimens from Tptpll and seventeen from Tptpul were tested in uniaxial compression. Uniaxial compressive strength (σ_c), Young's modulus (E) and peak axial strain (ε_{peak}) decrease with increasing lithophysal cavity content, or porosity (n) following exponential functions:

$$\sigma_c = 368e^{-10.1n} \tag{1}$$

$$E = 54.5e^{-5.6n} \tag{2}$$

$$\varepsilon_{peak} = 0.008e^{-3.94n} \tag{3}$$

Compared to nonlithophysal tuffs, failure of the lithophysal tuff specimens exhibited less brittle behavior. With an increase of porosity, the failure mode tended to be more ductile. When conducting multiple loading-unloading cycles, the maximum stress for the second and later cycles exhibited "memory" effect, i.e. the specimens remember the strength at the unloading point in the previous cycles before the specimens lost elasticity (Figure 1).

Keywords: Welded Tuff; Lithophysae; Lithophysal Cavities; Lithophysal Porosity; Yucca Mountain.

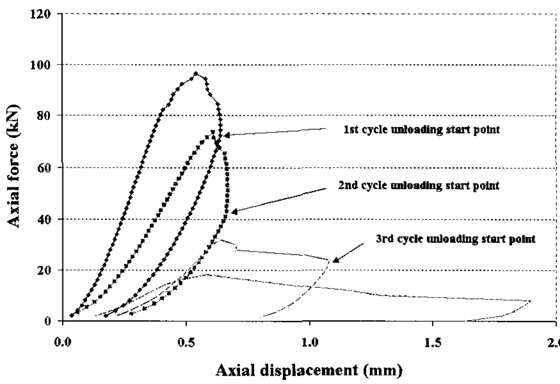

Fig. 1. Plots of stress versus total displacement under multiple loading-unloading cycles.

A STUDY ON ESTIMATING HYDRAULIC CHARACTERISTICS OF ROCK SPECIMENS USING ELASTIC WAVE

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Recently, underground space has been utilized for the purposes to dispose such as radioactive waste and CO2. In order to evaluate safety of geological disposal, it is important to know how ground water flows through rock masses. To measure underground hydrological characteristics, we develop the method to estimate permeability distribution in rock mass from elastic wave velocity. This method is based on dynamic poroelastic theory such as Biot and BISQ theory. These theories indicate that the elastic wave velocity through elastic solid saturated with pore water depends on its frequency and formulated with permeability with the characteristic frequency. Using these theories, we can estimate hydraulic characteristics from the characteristic frequency. However, there has been no study that tried to apply poroelastic theories such as Biot and BISQ theory to rock mass.

In order to verify the applicability of these theories to rock mass, we performed laboratory tests. We measured P-wave velocity of rock specimens for various frequencies from 30kHz to 1100kHz. The rock specimens used in this study were sandstone, tuff, shale, a kind of sedimentary rock and Inada-granite, a kind of crystalline rock. Especially, to create thermal crack, we put specimens of Inada granite under temperatures of 300°C and 600°C. From these results, for all rock specimens, the velocity-frequency dispersion was observed as the velocity is greater at higher frequency.

Then we compared measured results with theoretical solution. It was found that elastic wave dispersion for sedimentary rocks can be described by Biot theory and that for crystalline rocks can be described by BISQ theory. Applicable theory was different by rock structure.

Furthermore, we found that for sedimentary rocks, it is possible to estimate the permeability coefficients by Biot theory. On the other hand, for crystalline rocks, deciding the value of Squirt flow length (unique parameter of BISQ theory) in consideration of void structure, we found that it is possible to estimate the permeability coefficients by BISQ theory.

Keywords: Elastic wave; rock specimens; permeability; Biot theory; BISQ theory.

USING ARTIFICIAL NEURAL NETWORKS TO PREDICT PRESSURE-DEFORMATION OF SOLIDS WITH FLAT JACKS

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Artificial Neural Network Method is used as a tool to help decreasing the amount of in field reading during a "Compensation test" in which flat jacks are put in slots made in concrete lining of a tunnel. The potential of Creep and swell in soft rocks is a difficult engineering task which has been studied in many research projects. The pressure from swelling rocks to the support system and lining of underground openings has been the focus in recent years. One method to determine such pressures is known as compensation method. This is to study the strain behavior of a stressed solid mass (rock or concrete) after making a flat slot. The flat jack recovers the created strain by applying a counter acting pressure to the walls of the slot. This is a time consuming and potentially erroneous operation which requires measurement of displacement to 0.001mm accuracy. To eliminate repetitive closure measurements, the pressure-deformation behavior of several slots are used to train a neural network and the result has enabled reducing the number of recordings to very few and to predict the solid strain behavior over time with just measuring the slot closure without any needs to pressurize the slot again.

The paper illustrates the implications of flat jack method and explains the used neural network method and discusses the applicability and validity of the proposed approach. The performed compensation test at Masjed-Soleiman underground power house has been used as an example to illustrate the method. Further research is being undertaken exploring the potential use of other ANN techniques in predicting time dependent properties (creep and shrinkage) of concrete structures.

Keywords: Flat jack; Swelling rock; Swelling pressure; Artificial neural network.

INFLUENCE OF WATER VAPOR PRESSURE OF SURROUNDING ENVIRONMENT ON FRACTURE TOUGHNESS OF ROCK

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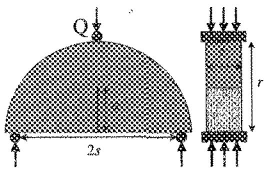
The influence of water vapor pressure of surrounding environment on fracture toughness of rock was clarified, based on the results of a series of semi-circular bend (SCB) test [1] under various water vapor pressures. The water vapor is most effective agent which promotes stress corrosion of rock [2]. The rock used in the test was Kumamoto andesite, and the region of water vapor pressure was 10^{-2} to10^{3} Pa.

Figure 1 shows schematic and photographic views of the experimental set-up of the SCB test. This set-up is installed in the special vacuum chamber as shown in Figure 2, in which the water vapor pressure can be controlled. An example of load-displacement curve at loading point Q in Figure 1 is shown in Figure 3. Fracture toughness is evaluated by maximum load. The relationship between estimated fracture toughness and water vapor pressure is shown in Figure 4. The fracture toughness K_{IC} linearly decreases with increasing water vapor pressure p in a logarithmic graph. This relation is approximated as

$$K_{IC} = \beta p^{-m} \quad , \quad \beta=1.62,\ m=0.0125 \tag{1}$$

Keywords: Rock; Semi-circular bend test; Fracture toughness; Water vapor pressure; Stress corrosion.



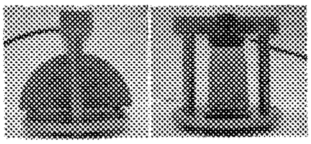


Figure 1. Experimental set-up for the SCB test.

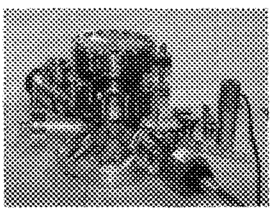


Figure 2. Vacuum chamber.

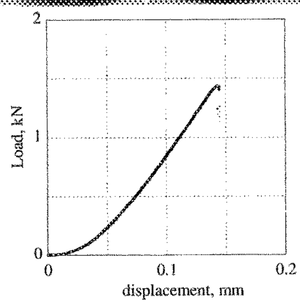


Figure 3. Load-displacement curve at loading point Q.

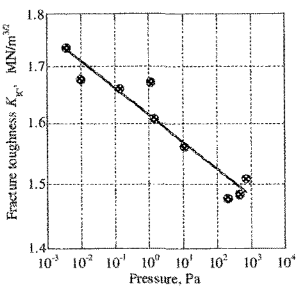


Figure 4. Relationship between fracture toughness and water vapor pressure.

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RECENT EXPERIENCES IN SINGAPORE LIMESTONE ROCKS

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The existence of Limestone rock has not been detected or recorded during the publication of the "Geology of the Republic of Singapore" by PWD in 1976. This document however continues to be used widely by the engineering community in Singapore. Nowadays the presence of limestone rocks in Singapore is acknowledged and greater attention is being paid during site investigations to detect its presence.

The properties of Singapore limestone rocks and the information required for design and construction in this formation are to date not very well established. The paper by Jeyatharan *et al.* 2003, summarised the information available to that time on Singapore limestone rocks and made an attempt to define its extent. Since the publication of that paper underground construction works have been carried out in some areas where limestone rock is present. One such area is located along the coast line between the eastern end of West Coast Highway and Jalan Buroh bridge encompassing West Coast Park and Pasir Panjang Container Terminals, Figure 1. Within the stated area substantial site investigation has been undertaken and underground construction works have been executed. The objective of this paper is to summarise the information available from the study area with a view to reaching generic conclusions regarding engineering aspects and construction issues related to Singapore limestone rocks.

Keywords: Limestone; geology; Singapore; properties.

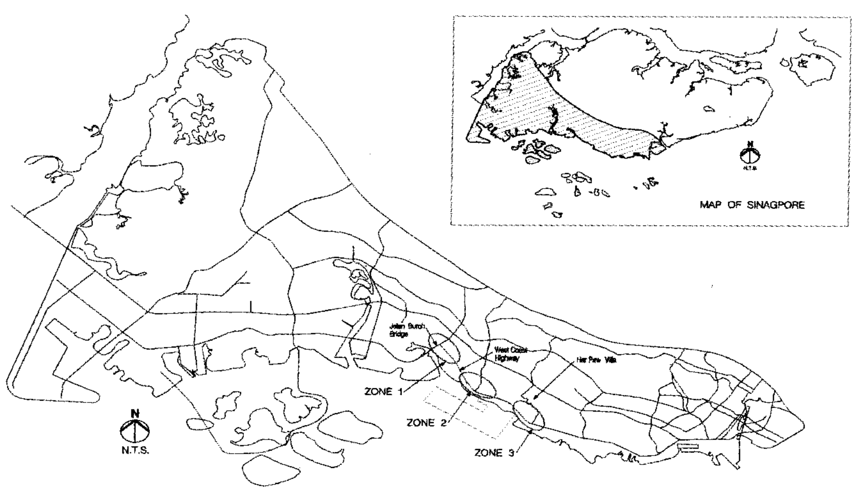

Figure 1. Extent of area where limestone rocks are likely to be met in Singapore and locations of zones covered by this study.

SUBSURFACE ASSESSMENT IN THE KARST AREA USING 3-D RESISTIVITY TECHNIQUE

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As the social needs for the infrastructures including railways, bridges, highways, and tunnels rapidly grows in the modern countries, the constructions of large-scale facilities are not avoidable even in the areas with weak rock mass such as karst terrains. There are a lot of underground cavities formed due to the dissolution of the limestone by the groundwater in the karst area. Therefore, there seems to be more chances that the ground subsidence could occur in the karst areas where the large-scale structures are under construction. The ground subsidence may cause the interruption of construction works and the changes of the original engineering designs of the constructions, and more importantly safety issues, resulting in large economic losses. Therefore, it is required to obtain detailed information regarding the locations and distribution of the underground cavities prior to the construction works in the karst areas.

In this study, we examined the applicability of 3-D electrical resistivity technique to probe underground cavities at the field test site in the karst area, located in Yongweol-ri, Muan-gun in the Southwestern part of Korean Peninsula. The underground cavities are widely present within the limestone bedrock overlain by the alluvial deposits in the test site where the ground subsidences have occurred in the past. The limestone cavities are mostly filled with groundwater and clayey soils in the test site. Therefore, underground cavities show anomalously low electrical resistivities compared to the surrounding host bedrock. The results of the study have demonstrated that the zones of low resistivities correspond to the zones of the cavities identified in the boreholes in the site, and that the 3-D electrical resistivity survey used is a very effective tool to detect and map underground cavities in the karst area.

We examined the applicability of 2- and 3-D electrical resistivity survey to detect limestone cavities of ground subsidence area. The major findings of this study can be summarized as follows:
1) The limestone cavities have low resistivity because cavities are mostly filled with groundwater and clayey soils. 2) It was proven that the zone of low resistivity 50 ohm-m or less was exists limestone cavities identified in the boreholes at the site. 3) And also 3-D electrical resistivity survey used is very effective tool to detect and map underground cavities in the karst area.

Keywords: 3-D electrical resistivity survey; limestone cavities; ground subsidence.

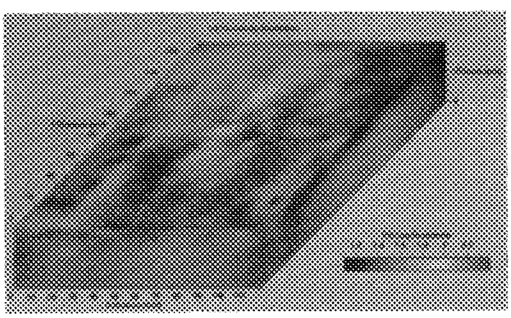

Fig. 1. 3-D resistivity distribution.

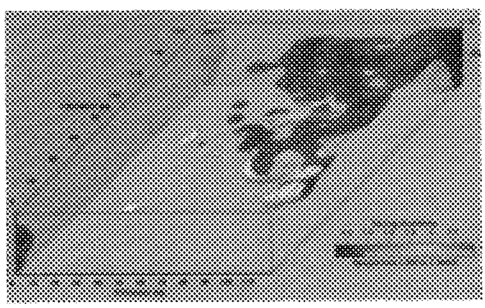

Fig. 2. Underground cavities imaging from 3-D electrical resistivity survey.

CONSIDERATIONS IN DEVELOPING AN EMPIRICAL STRENGTH CRITERION FOR BIMROCKS

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The strength of geological materials is one of the fundamental input parameters used in the design of civil engineering works; including those projects to be constructed in complex geological mixtures or fragmented rocks such as mélanges, fault rocks, coarse pyroclastic rocks, breccias and sheared serpentines. These often chaotic, mechanically and/or spatially heterogeneous rock masses are composed of relatively rock inclusions surrounded by weaker matrix, and may be considered bimrocks (block-in-matrix-rocks).

It is almost always impossible to prepare standard core samples from bimrocks in order to perform laboratory studies. Therefore, for these rocks, determination of the mechanical parameters such as cohesion, friction angle and uniaxial compressive strength is extraordinarily challenging. Although there is sparse literature describing empirical and laboratory studies on bimrocks, there is no widely accepted empirical approach in the rock mechanics community due to the limitations of the existing empirical equations, which were developed largely for more tractable, relatively homogenous rock masses.

In this study, an exhaustive database was developed composed of literature overviews and laboratory studies. Artificial bimrocks were prepared in the laboratory for uniaxial and triaxial compression testing. Plaster of Paris, bentonite, cement and water were mixed in different ratios to fabricate matrix types with various strengths. In addition, real tuff and andesite blocks, fragmented to centimeter sizes to create blocks, were mixed with the matrix to create artificial bimrocks. Uniaxial and triaxial compression tests were conducted on specimens of pure matrix and artificial bimrock mixes having different block proportions.

Finally, a series of statistical regression analyses were applied to the results of the laboratory strength tests to develop an empirical approach for estimating the overall strength of bimrock mass, by incorporating the Mohr-Coulomb strength model and the Hoek-Brown empirical criterion, both of which are widely used in rock engineering. The empirical equations were summarized in Eqs. (1 and 2). The empirical approach based on the Hoek-Brown was found to yield a slightly better predictive performance than an empirical approach based on the Mohr-Coulomb.

$$c_N = 1.25 - \exp\left(\frac{VBP-100}{75}\right) \qquad c_{bimrock} = c_N x c_{matrix} \tag{1a}$$

$$\phi_N = \exp\left(\frac{8xVBP}{1000}\right) \qquad \phi_{bimrock} = \phi_N x \phi_{matrix} \tag{1b}$$

$$UCS_N = 1 - \exp\left(\frac{VBP-100}{25}\right) \qquad UCS_{bimrock} = UCS_N x UCS_{matrix} \tag{1c}$$

$$UCS_{bimrock} = \frac{2c\cos\phi_{bimrock}}{1-\sin\phi_{bimrock}} \qquad \sigma_1 = UCS_{bimrock} + \left(\frac{1+\sin\phi}{1-\sin\phi}\right)\sigma_3 \tag{1d}$$

$$m_{i\_N} = \exp(0.015xVBP) \qquad m_{i\_bimrock} = m_{i\_N} x m_{i\_matrix} \tag{2a}$$

$$\sigma_1 = \sigma_3 + UCS_{bimrock}\sqrt{\left(m_{bimrock}\frac{\sigma_3}{UCS_{bimrock}} + 1\right)} \tag{2b}$$

where, c and ϕ are cohesion and internal friction angle, UCS is uniaxial compressive strength, m_i is m parameter of intact sample. σ_1 and σ_3 are maximum and minor principal stresses.

Keywords: Bimrock; Hoek and Brown; Mélange; Mohr-Coulomb.

GROUND STABILITY AT LIMESTONE REGION WITH UBIQUITOUS CAVITIES BY FLUCTUATION OF GROUNDWATER

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For the residential region where limestone bedrock exists near the surface and the groundwater has been developed for the agriculture during a few decades, the instability of surface ground is diagnosed and predicted. The geophysical and geotechnical investigations are carried out for understanding the distribution of the cavities. PFC(Particle Flow Code) modelling is performed for the simulation and prediction of the surface subsidence according to the conditions of cavity geometry and groundwater level.

According to the field investigation, limestone bedrock extends to the near surface up to about 10m below the ground surface and small cavities are scattered especially at shallow depth of 10~40m. Soil overburden is mostly clay and limestone bedrock mostly has narrow and steep discontinuities of network type. The groundwater level shows fluctuation between the depth of 5~20m and this trend is estimated as the main factor of washing away the soil overburden above the cavity and the sinkhole type subsidence of surface ground.

PFC(Particle Flow Code), which can visualize the dynamic behaviors of ground in time sequence, is adapted as the methodology of numerical modelling for subsidence process simulation. The key factors in this stability analysis are geometry of cavity(shape, size, depth), characteristics of soil overburden and variation of groundwater level. The variation of groundwater level in numerical modelling is simulated by supplementing the reaction force corresponding to hydraulic head of groundwater level to the inner wall of cavity model. Numerical simulations for various combinations of the these key factors are carried out and the criteria on the stability of ground subsidence are suggested as the relations between the cavity pattern(width, height, depth) and the groundwater level.

The cavity pattern which causes the subsidence of surface ground, when associated with the draw down of groundwater level, is estimated as that at 10m depth just below the soil overburden. In this case, the draw down of groundwater level below the lower boundary of soil overburden disturbs the mechanical equilibrium of ground. This drives washing away the overburden soil through the cavity channel and consequently causes the subsidence of surface ground. It is shown from the numerical modelling analysis that it is very important for the stability of surface and near surface ground in limestone region to maintain the groundwater level above limestone bedrock namely, within the soil overburden depth.

Keywords: Limestone cavity; ground subsidence; groundwater fluctuation; particle flow modelling.

STRENGTH DEGRADATION OF GRANITE UNDER CONSTANT LOADING

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Power shortage has become obstacle of rapid economic development. Nuclear power could provide an unlimited source of cheap and clean power. The only disadvantage is that it produces high-level radioactive waste (HLW). Geological disposal has been regarded as the best option for disposing the HLW. Some radioactive isotopes should be isolated in the repository until they have no significant risk to the biosphere. The technical challenge of repository design is the unprecedented long designed lifetime and the potential effect of temperature due to the heat generated by radioaction of the HLW.

This paper summarizes a series of constant loading tests under various temperatures and confining pressures. Some typical results are presented in term of deformation and deformation rate, strain path, dilatancy index and strength degradation with time. Acoustic emission moment tensor analysis is also carried out to study the failure mechanism. The results provide fundamental understanding of the long-term behavior of brittle rock and will be very useful for model calibration and numerical analysis in the repository design.

Keywords: Constant loading test; Strength degradation; Dilatancy; Acoustic emission; Granite.

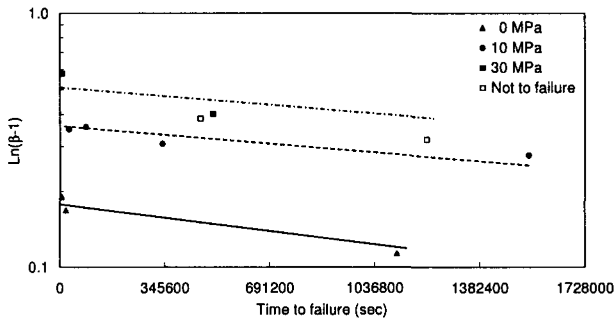

Fig. 1. Strength degradation with time (Room temperature, various confining pressures).

APPLICATION OF DESIGN OF EXPERIMENTS TO PROCESS IMPROVEMENT OF PFC MODEL CALIBRATION IN UNIAXIAL COMPRESSION SIMULATION

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Calibration of bonded particle model has been debated ever since Particle Flow Code was introduced to simulate rock engineering and geomechanics problems. In this study, a new approach is devised for calibrating contact bonded particle models using Design of Experiment and optimization methods in uniaxial compression simulation. Sensitivities of input microparameters on uniaxial compressive strength, Young's modulus, and Poisson's ratio (i.e. macroscopic responses of model) were tested by Plackett-Burman design method. Then, for each macroscopic response two microparameters having the largest impacts were sorted out and their non-linear relations were estimated by statistical Central Composite Design method. Using the results from Plackett-Burman design and Central Composite Design method, the problem of finding a set of input microparameters was solved using optimization method by which the optimum set that gives the best agreement between the results both from the bonded particle model simulations and laboratory test is obtained. Adequacy of the optimum solution is checked by comparing stress-strain curves from laboratory test and simulation. Also the overall procedure was applied to calculate optimum sets of input microparameters for different rock types. Results from both simulations and laboratory tests gave good to fair agreements for hard rocks. The method is currently applied to simulation of rock material with its physical properties falling within the following range: uniaxial compressive strength (40–134 MPa), Young's modulus (22–55 GPa), and Poisson's ratio (0.19–0.25). The presented approach is useful at the onset of a bonded particle modeling especially in choosing a proper set of input microparameter for model generation.

Keywords: PFC; Design of Experiment; Plackett-Burman design; Central Composite Design; Optimization.

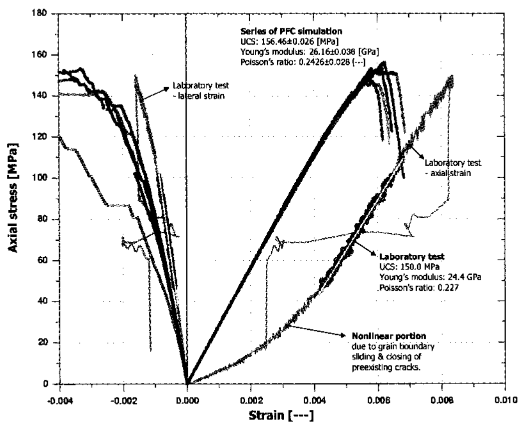

Fig. 1. Comparison of stress – strain curves from uniaxial compression laboratory test on Korean granite and PFC simulation. The contact bonded particle model is generated using the optimum set of microparameters obtained through optimization.

ACOUSTIC EMISSION BEHAVIOR IN THE PROGRESSIVE FAILURE OF ROCK SAMPLE CONTAINING WEAK ZONES

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Numerical simulation of the progressive failure leading to collapse is performed under uniaxial compressive condition by a finite element method, Rock Failure Progress Analysis (RPPA^{2D}).The simulated rock sample with three weak inclusions is drove by an increasing displacement with an increment of 0.001 mm/sep. The mesoscopic scale heterogeneity for the elements by assuming the random strength and elastic modulus to follow a Weibull distribution is introduced in the inclusions. The macroscopic heterogeneity represents the variation of the inclusion size, shape, mechanical properties and the volume percentage. One sample with the same distribution of strength and elastic modulus but no inclusions is performed a uniaxial compressive test for the comparison with the failure pattern of the sample including three inclusions to specify the influence of material heterogeneity in both meso and macro scales. The acoustic emissions or microfracturing events display self-organized criticality—from disorder to order. The rock failure is of great structural effect, which is greatly dependent on the material heterogeneity (especially on the macroscopic scale). The localization (or nucleation) in rock failure process contributes much to the failure pattern. Through the analysis of acoustic emission (AE) time series by rescaled range analysis method, it is found that the AE time series exhibits the similar scale-invariant property. It is inferred that in some cases, earthquakes are not predicted so successfully because we don't know much about the earth's crust although we understand earthquake mechanism and control related predictable methods well.

Keywords: Acoustic emission; Rescaled range analysis; Long rang correlation; Numerical simulation.

Table 1. Material mechanical property inputted in the numerical model.

	Rock mass	Inclusion I	Inclusion II
Homogeneity index	4	2.5	1.5
Elastic modulus (MPa)	60000	30000	10000
Compressive strength (MPa)	150	100	50

*The study is supported by NSFC major program (No 50490273).

STUDY ON THE DAMAGE EVOLUTION EQUATION OF THE FRACTURED ROCKS BASED ON THE TRIAXIAL COMPRESSION TESTS

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Based on the analysis of the data of triaxial tests, and the process of elasticity and plasticity damage evolution of rock, the elastic and plastic damage evolution equation (EPDEE) of the two typical at the LuZhong Metallurgy Cooperation is proposed.

$$D = 1 - \exp(a\varepsilon_p^b)$$

The EPDEE has few parameters and clear physical concepts. By the tests, the a and b can be observed. We can get the next table.

Damage evolution equation of fractured rocks and the maximum value of damage variables

Courses		Damage evolution equation	The most damage value
ε_1—Direction	Altered diorite-porphyrite	$D_1 = 1 - \exp(-0.585\varepsilon_{p1}^{0.331})$	0.57
	Skarn	$D_1 = 1 - \exp(-0.546\varepsilon_{p1}^{0.162})$	0.55
ε_3—Direction	Altered diorite-porphyrite	$D_3 = 1 - \exp(-0.615\varepsilon_{p3}^{0.332})$	0.59
	Skarn	$D_3 = 1 - \exp(-0.478\varepsilon_{p3}^{0.262})$	0.58

The expression is simple which has only two coefficient factors a and b, which can be determined from the tests results.The damage evolution equation describes the damage state and its relation to the plastic strain of the fractured rock material. This shows that the damage of fractured rock results from the plastic strain (residual strain) and has nothing to do with reversible elastic strain. Accordingly the established equation reflects the real reason that leads to damage and more importantly it also has clear physical concepts.

Keywords: Triaxial test, elastic and plastic damage evolution equation (EPDEE), fracture rock.

GROWTH AND COALESCENCE OF INTERNAL FLAWS IN BRITTLE MATERIALS

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Under the action of compressive load, the internal flaws in brittle materials (rock and rocklike materials e.g.) will result in rock and rock mass fractures. The mechanisms on growth and coalescence on internal flaws are not clear. So a series of experiments on growth and coalescence pattern of two internal flaws were finished with frozen casting resin material inside which imbedded with two or three parallel polyester films. The results show that: Although the initial process of the internal flaws are similar with a single flaw, their growth and coalescence process are different. The fracture process of internal flaws may be divided into three stages. (1) The initial wing-wrapping cracks (primary cracks) turn up on the flaw tip and gradually rotate to the loading direction. (2) Anti-wing wrapping cracks (secondary cracks) grow toward the opposite direction of initial cracks. The growth of cracks will form a big stress kinking zone near flaws. (3) The anti-wing wrapping crack and wing wrapping crack continuously grow inside the specimen and result in burst-like failure of the whole specimen. The wing cracks (Mode I) primarily result from the tensile stress at flaw tip. Petal cracks (Mode III) emerged along flaw boundary. Anti-wing cracks belong to Mixed Mode fractures in middle of flaws. But, the fracture pattern takes on some new performances. The sample failure comes from the growth and coalescence between the anti-wing wrapping crack induced from one flaw and the wing wrapping crack initiated from another flaw. Some parameters of cracks growth are listed in the Table 1. The mechanisms lead to growth and coalescence of internal cracks in brittle materials are described in the paper. Although some fundamental results are obtained in this study, the fracture mechanisms of 3-D internal flaws cannot be still fully understood, especially the stress distribution near flaws is not computed during propagation till now. Further studies will be carried out on the topics.

Keywords: Internal flaw; Growth; Coalescence; Wing crack; Anti-wing crack; Mechanisms.

Table 1. Parameters of growth of the wrapping cracks

Flaw No.	Initial wing angle (°)	Initial wrapping angle (°)	Maximum Length of crack propagation (mm) Wing crack Upper tip	Wing crack Lower tip	Anti-wing crack Upper tip	Anti-wing crack Lower tip
①	79	150	32	12	12	10
②	75	155	8	15	3	11
③	80	152	9	40	2	6
④	69	147	44	20	13	19
⑤	66	123	18	53	7	5

7.2. In-Situ and Laboratory Tests

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MODELING BRITTLE FAILURE OF ROCK USING DAMAGE-CONTROLLED TEST

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At low in-situ stress magnitudes, the continuity and distribution of natural fractures in a rock mass control failure processes. However at highly stressed levels, the failure process are affected and eventually dominated by stress-induced fractures growing preferentially parallel to the excavation boundary. This fracturing is generally referred to as brittle failure by slabbing or spalling. Continuum models with traditional failure criteria such as Hoek-Brown or Mohr-Coulomb based on simultaneous mobilization of cohesive and frictional strength components have not been successful in prediction the extent and depth of brittle failure. Instead cohesion weakening and frictional strengthening (CWFS) model based on non-simultaneous mobilization of cohesive and frictional strength components is known as a model to predict well brittle failure.

In this paper to evaluate the brittle failure around deep underground openings, numerical simulations using CWFS model were performed. For obtaining the input parameters in CWFS model damaged-controlled tests were carried out. Damaged-controlled test is loading and unloading repetition test for obtaining complete a stress-strain curve. Crack damage stress, cohesion strength and frictional strength with changes of volumetric strain were determined from the tests. The test results show that as friction is mobilized in the sample, cohesion is reduced. And they can be expressed as a function of rock damage or plastic strain.

In numerical simulation using the CWFS model with the parameters from the damage-controlled test, the predicted depth and extent of failed zone (shape of failed zone) is in agreement with the measured failed zone, contrary to numerical simulation with traditional criteria.

Keywords: Failure; rock test; modeling.

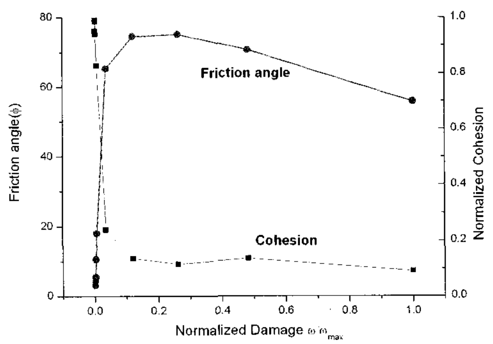
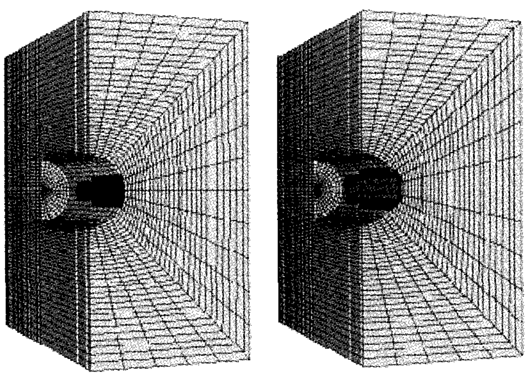


Fig. 1. Mobilization of friction and cohesion.



(a) MC model (b) CWFS model

Fig. 2. Comparison of the MC model and CWFS model.

DETERMINATION OF ELASTIC CONSTANTS FOR TRANSVERSELY ISOTROPIC ROCK SPECIMENS BY A SINGLE UNIAXIAL COMPRESSION TEST

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The rotation transformation used by Lekhnitskii (1963) and others thereafter was modified. The subsequent calculations to obtain the elastic moduli in the past researches were also reviewed and corrected. A new uniaxial compression testing method was suggested where four independent elastic moduli can be obtained by a single uniaxial compression test with the attachment of stain rosette gauges on the specimen. The newly suggested method has the advantage in that the elastic moduli are directly obtained from a single test. In order to examine the validity of the new method, cement mortar specimens were used. The increased accuracy and precision of the determination of elastic moduli was observed.

Keywords: Uniaxial compression test, anisotropy, transversely isotropy, rotation transformation, elastic muduli, Poisson's ratio, cement mortar specimen.

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MECHANICAL RESPONSE OF VINDHYAN SANDSTONES UNDER DRAINED AND CONFINED CONDITIONS

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The mechanical responses like compressive strength, tensile strength, shear strength and cohesive strength, of rocks are some of the basic input parameters for planning and design of civil and mining engineering structures. Rocks exhibit variable strength in space and time due to their inherent fabrics and associated environments. Among various factors affecting the mechanical properties of rocks, the water is one of them significantly influencing the mechanical parameters of rocks; which would be relevant before planning and design of any engineering structure due to higher possibilities of water interaction with engineering structures like tunnels, dams, bridges. Hence, the study related to effect of water on deformation and mechanical properties of rocks are of utmost relevant to geo-mechanics and engineering geological investigations for development of engineering structures like tunnels, damns, roads, bridges, mines, etc.

Therefore, the present paper illustrates the effect of water on deformation and mechanical response of sandstone under various stress conditions. For the purpose sandstones belonging to Upper Kaimur Group, Vindhyan Supergroup were collected form Vindhum Fall locality 15 km east of Mirzapur City ($23^0 25' – 23^0 50'$ N latitude and $76^0 - 76^0 52'$ E longitude), District Mirzapur, Uttar Pradesh, India. The sandstones of the area are homogeneous and isotropic in nature (mechanical anisotropy 0.98), thus useful for geomechanical and engineering geological studies.

Both dry samples and samples saturated with ground water were deformed under uniaxial compression and tension (by Brazilian method) under drained condition. In addition, the some of the dry samples and samples saturated with water were deformed under triaxial stress condition at confining pressure of 50, 100, 150 Kgf/cm^2.

The experimental results reveal pronounced variation in compressive, tensile, shear and cohesive strengths may be attributed to the differences in mineralogical constituents, matrix/cement, textures and microstructures. The sandstone of the upper level of vertical structural column exhibits highest compressive strength (102.5 MPa), tensile strength (6.93 MPa), shear strength (17.64 MPa) and cohesive strength (33.06 MPa) and lower angle of internal friction (37^0). However, the sandstones of the bottom level show lower compressive strength (55.57 MPa), tensile strength (3.37 MPa), shear strength (8.29 MPa), cohesive strength (12.92 MPa) and higher angle of internal friction (42^0). The sandstones saturated with ground water shows pronounced reduction in mechanical properties in comparison to the mechanical properties of sandstones in dry condition. The reduction in mechanical properties induced by saturation of ground water is minimum in sandstones of upper level of structural litho-column, while it is maximum in sandstone of bottom level may be due to quantitative variation in pore pressure developed in pore spaces, resulted through the distinct differences in textures and microstructures of the sandstones. On the other hand, the saturation of sandstones with ground water shows highest reductions in strength under shear stress environment in drained condition may be due to enhanced sensitivity of dissolution of matrix/cement (binding martial) and corrosion of grains by water under shear stress and minimum reduction in strength under tri-axial stress environment in confined condition, which may be imparted by low dissolution and corrosion potential of ground water to internal structures of the sandstone in addition to developed pore pressure.

Finally, the study suggests that the mechanical response of the rocks are the function of mineral constituents, matrix/cement, textures and microstructures and the sensitivity of water to reduce the values mechanical properties of rocks depends on the pore pressures in intergranular spaces in the frameworks of the rocks, dissolution of matrix/cement and corrosion of grain boundaries. Special cares are recommended for the natural rock slopes and under ground spaced engineering structures in watershed region particularly in zone subjected to shear stress environment. The confinement (lining and support system, retaining walls) or release of water from slopes and such engineering structures would be a control measures for long term safety of engineering structures and to minimize the rock slide hazards.

Keywords: Rock properties; rock test; sandstone.

ROOF GEOSTRUCTURE LOGGING SYSTEM USING PORTABLE PNEUMATIC DRILLING MACHINE

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We developed a new logging system for roof rock structure to visualize changes of strata and crack distribution. Information on roof rock along a roadway of underground coal mine is important to maintain mine safety, that is, rock fall prevention and effective support by roof bolting and cable bolting.

This system consists of hardware and computer software. To detect the mechanical data (torque, thrust, revolution and stroke), four sensors are attached on the portable pneumatic drilling machine, Trussmaster 1 (Rambor Ltd.). All data from sensors are recorded on a memory card of data logger with the sampling time 10 msec, after passing through the amplifier and barrier circuit.

Figure 1 shows a typical result of laboratory drilling tests. Ratio of torque and thrust shows the difference of drilling materials as well as rotational energy. In this test, drill bit penetrates hard concrete to soft one. Black dots on the horizontal axis of the torque/thrust log in Fig.1 represents the boundaries of discontinuity sach as the aggregates of the concrete test block and the boundary of layers. After compiling these logs about a test block or a roof rock region of roadway, the computer software can visualize their structure in 3D space.

Keywords: Geostructure; Roof Rock; Mechanical Data; Drilling Machine; Visualization.

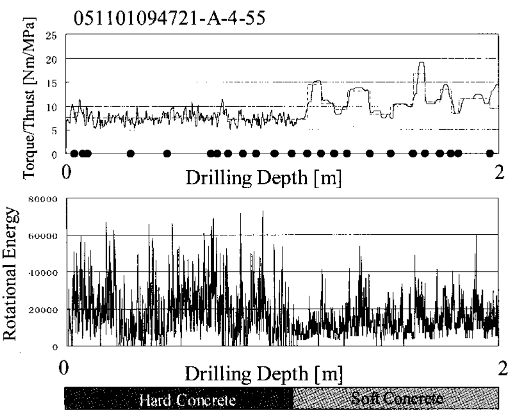

Fig. 1. Torque/thrust log and rotational energy log for concrete block test.

EFFECT OF POROSITY BETWEEN SPIRAL BAR AND CRUSHED ROCK IN BOREHOLE

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The spiral bar is the new support system for the purpose of the stability of slope, tunnel, ground, and so on. To evaluate an effect of porosity between spiral bar and crushed rocks in a borehole, pullout test in laboratory and numerical analysis using FLAC3D based on a three-dimensional finite difference method (3-D FDM) were performed. The results show that the porosity not only varies according to the dimension of spiral bar but also has an effect on the pullout load. Pullout load initially increases with the increasing displacement, and then gradually decreases. The results of the relationship between pullout resistance and porosity of crushed rock in borehole suggest that smaller the degree of porosity, larger the pullout load, and also porosity is related to the size of the spiral bar (Fig. 1). The relationship between pullout load and displacement from the experimental and numerical analysis revealed that the result of numerical analysis in low porosity nearly coincided with one of the experiments. On the other hand, the one in high porosity showed large deviations from experiment result. It is considered that the former is due to the state of residual resistance after the maximum pullout load, and the latter is due to decreasing pullout load by means of slip phenomena.

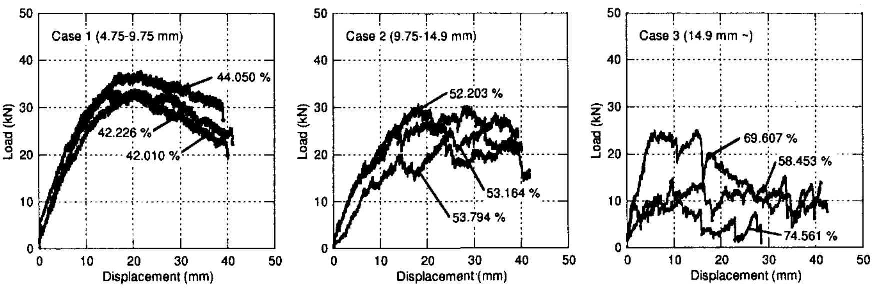

Fig. 1. Relationship between pullout load and displacement. The size of crushed rocks of (a) ϕ4.75-9.75 mm (Case 1), (b) ϕ9.75-14.9 mm (Case 2) and more than ϕ14.9 mm (Case 3).

Keywords: Spiral bar; Pullout; Crushed rock; Porosity; Borehole; Numerical analysis; 3-D FDM (finite difference method).

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DETERMINATION OF MODE II STRESS INTENSITY FACTOR USING SHORT BEAM COMPRESSION TEST

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The stress intensity factor is the most important parameter in LEFM since it is able to correlate the crack propagation and fracture behavior. Mode II, shear failure is one of major problems in the stability of materials and structures in rock engineering. One of difficulties in the shear failure studies is to produce pure shear failure in laboratory tests.

In this research, the short beam compression test has been used to estimate the shear strength and stress intensity factor for Flagstaff sandstone. The specimen has a length of W, a width of L and a thickness of B. Two equal and opposite pre-existing slots, which have a depth of $a = L/2$, and slot separation of c, are cut perpendicular at the central part of specimen. The test specimen and schematic illustration of testing apparatus are illustrated in Figure 1.

Results of experimental and finite element analyses show the validity of the short beam compression test as a shear test. A slot separation ratio of 0.2 was the most favorable geometry for the shear test.

The mode II stress intensity factor is determined by finite element analysis using the displacement extrapolation method. Since shear cracks developed not in the co-planar direction but perpendicular to the pre-existing slot, small notches are introduced perpendicular to the pre-existing slots. Using these small notches, the stress intensity factors are evaluated at a given c/W ratio. The average fracture toughness of Flagstaff sandstone is 1.31 $MPa\sqrt{m}$. Applied biaxial stress did not affect the geometry factor but an increasing biaxial stress results in an increase in the fracture toughness.

Keywords: Short beam compression; Mode II stress intensity factor; Fracture toughness, Shear failure.

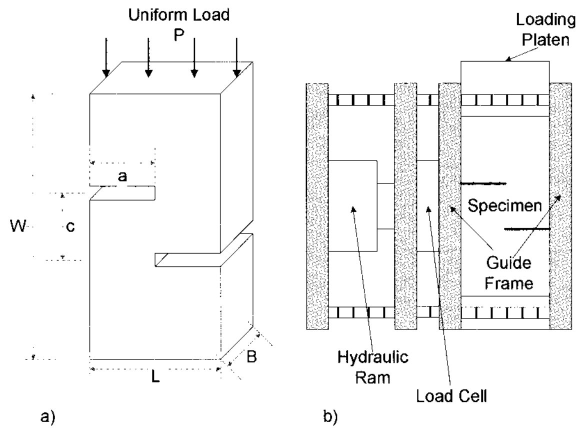

Fig. 1. Short beam compression test: a) specimen geometry; b) testing apparatus.

EXPERIMENTAL STUDY ON STRENGTH AND DEFORMATION CHARACTERISTICS OF PHYLLITE

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Most often the metamorphic and sedimentary rocks possess inherent or structural anisotropy. Since many underground constructions are coming up on such rock formations, the research on the influence of intrinsic anisotropy on the strength and deformation characteristics has attracted a great deal of interest in recent years. The engineering behaviour of naturally occurring schistose or layered rocks is often transversely isotropic, *i.e.* they display a polar symmetry with respect to axis normal to the plane of foliations or bedding.

Besides anisotropic nature of rocks, the moisture content is another important factor which influences the strength and failure mechanism of rock and needs utmost attention. In underground openings and coal mines, the degree of saturation may vary from very low to sometimes even 100% depending upon several factors.

With this in view, an extensive experimental investigation has been carried out on one such anisotropic rock, namely phyllite purposely selected from the lower Himalayan region in northern India. The rock samples were procured from an important project site with the aim to study its engineering behaviour in different test modes under tensile and compressive stresses. Tests were conducted on oven-dried and water-saturated specimens. The experimental results pertaining to different tests under dry and saturated conditions are analysed and discussed in the paper in order to develop an improved understanding regarding the effect of parameters like mineralogy, orientation of schistosity with respect to loading direction and saturation on the strength and deformation behaviour with special reference to the degree of anisotropy in different characteristics. The results have shown a pronounced effect of fabric anisotropy on various mechanical properties of strength and deformation. Saturation of phyllite is found to decrease the anisotropy ratio by 8.49% in wave velocity test whereas to increase it by 12.0% in axial point load test, 3.6% in diametral point load test, 16.0% in Brazilian test and 4.4% in uniaxial compression test. Furthermore lower values of tangent modulus of elasticity and axial and diametral strains were obtained in saturated specimens. The rock exhibited maximum anisotropy ratio in Brazilian test, and hence it is suggested to be used for preliminary estimation of the degree of anisotropic nature of foliated rocks.

Keywords: Strength; deformation; phyllite; anisotropy; saturated.

STUDY OF ANISOTROPY OF ROCK ELASTIC PROPERTIES OF FAIRBANKS SCHIST UTILIZING ULTRASONIC WAVES

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The elastic properties (Young's modulus, Shear modulus and Poisson's ratio) of rock can be obtained based on the dynamic method, which is conducted by measuring and analyzing the P- and S-wave velocities through the rock specimen. Elastic wave velocity anisotropy exists due to the presence of discontinuities such as grain boundaries, foliations, and microcracks within the rock. The anisotropy of elastic properties then can be determined based on the anisotropy of wave velocities. The anisotropic elastic properties of rock can be valuable data for the design of various engineering structures in foliated rock mass.

In this study, ultrasonic measurements of P- and S- wave velocities are carried out on cuboidal and cylindrical specimens prepared across and along the foliation planes. Both P- and S-wave velocities are recorded along different directions. The analysis of sonic velocities of P- and S-wave determines the directional elastic properties of the rock specimens. A series of compression experiments are also conducted to investigate the anisotropy of elastic properties along different directions in rocks. The results from the two testing methods are compared with each other and the correlation is analyzed. Our measurements reveal that the foliation of the rocks induces a clear elastic properties anisotropy between three orthogonal directions. The dynamic approaches utilizing ultrasonic velocity measurement provide quick and reasonably accurate estimates of the anisotropic elastic properties of rock.

Keywords: Anisotropy; Elastic Properties; Ultrasonic; Schist.

EXPERIMENTAL INVESTIGATION OF CREEP IN A SALTY MUDSTONE

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Salty mudstone is often found in bedded salt deposits as interlayers; these could have a great effect on the long-term stability of fluid storage facilities constructed in these deposits. One may deduce that the creep properties of non-salt interlayer materials such as the salty mudstone are very important, as significantly differences in deformation rate over time would undoubtedly lead to stress concentrations or strain incompatibilities, leading to slip and interface weakening. In this paper, some stepwise creep experiments of salty mudstone samples are described. The evaporitic mineral ingredients of salty mudstone have some effects on its creep properties, and the results demonstrated that the steady creep rate changed not only with load, but also with the mineral constitutive contents of the specimen.

The comparison between the steady creep rate of mudstone and rock salt samples demonstrated that they are within an order of magnitude: the steady creep ratio of the rock salt specimens is about 5 times that of the mudstone samples. Given the slow creep rates, small strains, and typical geomaterial heterogeneity (in salt as well as in the mudstone), we may assume that this is not a large difference. Although debonding of the interfaces will eventually occur, the fact that the claystone creeps means that stress redistribution will greatly delay problems (as compared, for example, with anhydrite or sandstone beds which may likely do not creep at the temperatures and stresses of interest).

Keywords: Salty mudstone; creep; mechanical property; rock mechanics.

Table 2. List of transient creep of salty mudstone and salt.

Lithology	Transient creep/%	Deviatoric stress/MPa
Salty mudstone	0.1~0.2	15.0
Silty rock salt	0.03~0.06	15.0
Rock salt	0.02~0.04	15.0

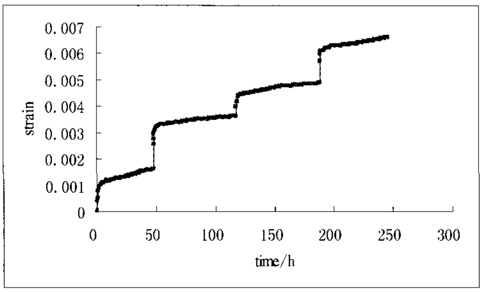

Fig. 2. Stepwise loaded creep curves of claystone sample 2#.

COMPARISON OF DIRECT SHEAR TEST RESULTS USING A PORTABLE DEVELOPED AND CONVENTIONAL DIRECT SHEAR TEST APPARATUS

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One of the methods for investigating the mechanical properties of rocks is direct shear test. It is considered as the simplest and the most commonly used methods for the shear testing of rock specimens containing a weakness plane or a discontinuity. This kind of test is normally carried out in the laboratory, but it may also be performed in the field, using a portable shear box to test discontinuities contained in a piece of drill core. As the normal and shear stresses are applied by hand pumps in the portable shear tests, there are some doubts on the accuracy of results. These ambiguities may be eliminated when using electrical hydraulic pumps instead of mechanically controlled hand pumps for applying shear stresses on the specimen. The shear stress increasing rate would be regular and may be controlled and changed by means of electrical pumps. As the power supply will be a 12 volts battery, the developed shear apparatus will also be portable. A series of shear tests have been conducted using both conventional and developed direct shear test apparatus. To be able to have a close comparison, artificially made specimens containing a discontinuity have been tested. The results obtained from these apparatus show some discrepancies which makes one think about the results of the conventional portable shear box.

Keywords: Direct Shear Test; Portable Shear Box; Artificially Made Specimens.

MEASURING ELECTRIC RESISTIVITY OF ROCK CORES FOR THE UNDERGROUND SEQUESTRATION OF CARBON DIOXIDE

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The technique sequestering CO2 into the ground is a high degree of expectation for reducing CO2 emission. The realization of the CO2 sequestration into underground aquifer must require the confirmation of the stability in long term after the sequestration of CO2. It is considered that measuring electric resistivity is a one of useful method to monitor CO2 migration. If we know resistivity variations of rock cores taken from wells, we can estimate CO2 saturation with high reliability. This study shows the experimental result of CO2 injection into rock specimens in a pressure vessel, which generates the same pressure and temperature environment in deep ground. We measured the temporal change of electric resistivity of rock specimens under the injection of CO2, which is composed of gas, liquid or supercritical phase. The results indicate the increasing of electric resistivity with CO2 injection. Therefore, the method of electric exploration can observe the distribution and variation of CO2 flow in the ground. Also, we can estimate CO2 saturation in pores using Archie's equation and the values are quite equivalent to the value calculated from the output volume of the experimental apparatus (Fig. 1). The saturation value estimated from electric resistivity is considered to have a high reliability.

Keywords: CO2; resistivity; supercritical.

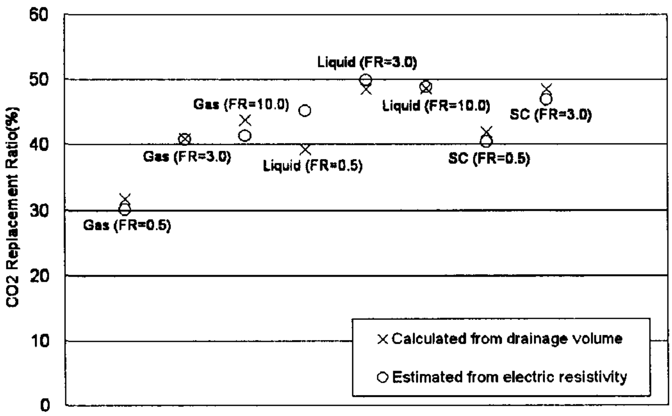

Fig. 1. Saturations estimated from the resistivity data and calculated from actual volumes evacuated from the rock core. FR means 'Flow Rate' and the unite are 'ml/min'. SC means 'Super Critical' phase. The saturations estimated from resistivity change using Archie's equation suit to the saturations calculated from actual evacuated volumes after the experiments.The saturation or displacement ratio of liquid or super-critical CO2 is higher than that of gas CO2. This indicates that CO2 condition should be the liquid and super-critical phase when we inject CO2 into underground. The difference is considered to be influenced on the ratio of viscosity between formation water and CO2.

RESEARCH OF MECHANICAL ENERGY AND TEMPERATURE DISTRIBUTION DURING DYNAMIC LOADING OF ROCKS

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Research on energy characteristics of rocks under dynamic loading is initiated by the need to observe these characteristics for better understanding processes that take place at anomalous geomechanical events in the rock mass. Up to now, the characteristics of rocks under dynamic loading have merely been studied from the stress-strain point of view. At this study anomalies have been found that may be explained by the conversion of strain energy to the other energy. To understand these events better, it is necessary to introduce the observation of the other energy of the test specimen in the course of testing. The test device MTS 816 Rock Test System is used for this research. The need to examine these events follows from the solving of the most serious problems from the area of mining geomechanics, i.e. anomalous geomechanical events, mainly rock bursts. These events go on dynamically at damaging rocks by the influence of elastic wave propagation. In the laboratory, rock loading may be simulated on the given device similarly to elastic wave propagation in the rock mass.

In the course of cyclic loading the following items were recorded: time, deformation and force. The record of these parameters was performed in the peak-valley mode, which enabled the calculation of mechanical energy A_{ci} supplied to the test specimen:

$$A_{ci} = \frac{(\Delta z_i - \Delta z_{i-1})(F_i - F_{i-1})}{2} \qquad \text{(J)} \qquad (1)$$

where Δz is the deformation of the test specimen (mm),

F - acting force (kN).

The total deformation energy applied to the test specimen on the certain state characterised by the test time t – A_{ct} was determined from the relation:

$$A_{ct} = \sum_{t=o}^{t} A_{oi} \qquad \text{(J)} \qquad (2)$$

The thermal energy A_T due to heating the test specimen was determined from the relation as follows:

$$A_T = a_T \cdot V \cdot \Delta T \qquad \text{(J)} \qquad (3)$$

where a_T is the specific heat of rock (for Carboniferous sandstone $a_T = 1.8$ MJ.m^{-3}.K^{-1}),

V - the volume of test specimen (m^3),

ΔT – temperature change (K).

The share of thermal energy in the total deformation energy supplied was expressed by the relation:

$$P_{AT} = \frac{100.A_T}{A_{ct}} \qquad (\%) \qquad (4)$$

Results of those measurements for Carboniferous sandstone are in the paper.

Keywords: Dynamic loading; Cyclic loading; Mechanical energy; Thermal energy.

EXPERIMENTAL STUDY ON DEFORMATION BEHAVIOR OF ROCK UNDER UNIAXIAL COMPRESSION AND DIRECT TENSION

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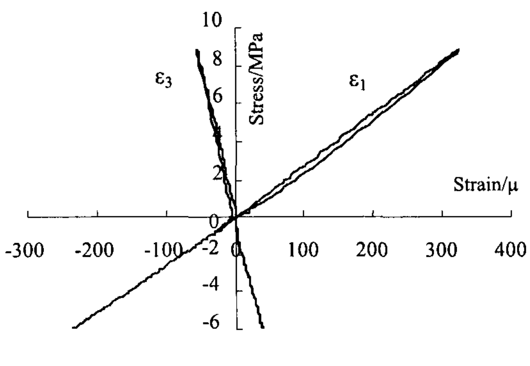
A new testing apparatus for rock's uniaxial compression and direct tension is introduced in this paper, which can carry out both tests on a same rock sample. This apparatus changes the situation of analyzing the test results by different rock samples under uniaxial compression and direct tension respectively, which will lead to a wrong conclusion. It also provides the possibility for studying the transitional characteristic of rock from being compressed to being stretched or vice-versa. Furthermore, the deformation characteristic of a kind of purple sandstone in Dayao copper mine is studied. The studies show that the compressive elastic modulus of sandstone is quite the same as the tensile elastic modulus, but the relationship between Poisson's ratios in compression and in tension is quite different. The tests also prove that rock compression and tension loading characteristics are not similar and the deformation curve of unloading in compression joins continuously together with that of loading in tension.

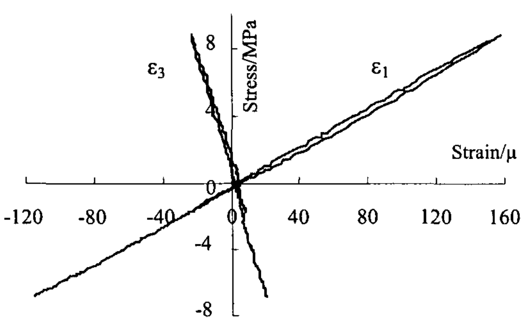
Keywords: Rock; Direct tension; Uniaxial compression; Deformation behavior.

Table 1. Comparisons of compressive elastic modulus and tensile elastic modulus of sandstone.

The parameters	E_A^y/GPa	E_A^t/GPa	E_{50}^t/GPa	E_A^y/E_A^t
Average	49.36	51.31	51.52	0.966
Variation Coefficient (%)	11	9	15	11



(a) Sample 1



(b) Sample 2

Fig. 5. The stress-strain curves of purple sandstone of Dayao copper mine both under uniaxial compression and direct tension.

FIELD TEST AND ANALYSIS OF ROCKS OF THE SOUTH-TO-NORTH WATER DIVERSION PROJECT

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The western Route Project for China's South-to-North Water Transfer is to divert about 20 billion m^3 water annually from the upper of the Yangtze River to the Yellow River, which will bring relief to the drought-ridden northwestern and northern parts of China. Better understanding of the site characteristics is very important for engineering project including successful geotechnical design and geoenvironmental issues. Engineering geological investigation and field and laboratory tests were carried out to obtain the physico-mechanical properties of the rocks. Engineering geological investigation discovered the site complicated geological conditions, such as polytype rocks, overgrown folding, numerous fractures and even some active fractures, etc. which will have great effect on the slope stability of the high dam and the excavation of tunnel and cave. Laboratory test cannot be used to accurately find out the mechanical parameters of these rocks because of difficult sampling and tester limitation, so several representative sections were chosen to carry out field tests, such as large-scale shear test (500mm × 500mm × 350mm) and wave-velocity test, etc. The investigation results were presented and evaluated critically in this paper. The means and procedure of field shear test were expounded and the relation curves of shear stress to displacement and normal stress to shear stress have been obtained. And then, the shearing strength and shear strength of rocks obtained from field shear test and the dynamic parameters of wave-velocity test were provided. The effects of parameters acquired from different methods were compared through a slope stability analysis, which is of utmost importance to the slope stability analysis and the earthquake-resistant design.

Keywords: Case study; water way; rock properties.

EXPERIMENTAL STUDY ON MECHANICAL PROPERTIES AND LONGITUDINAL WAVE CHARACTERISTICS OF TUFF, GRANITE AND BRECCIA AFTER HIGH TEMPERATURE

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To evaluate safety status of rock engineering (e.g. rock cavern, rock tunnel and rock slope) after disaster, it is necessary to establish elementary knowledge on mechanical and physical characteristics of different rocks after high temperature. This paper presents the results of an experimental study on mechanical properties and longitudinal wave characteristics of tuff, granite and breccia after different high temperatures. The scopes of the research involve: a) variation of peak stress, peak strain and elastic modulus with suffered temperature, b) variation of longitudinal wave velocity with suffered temperature, c) relationship among longitudinal wave velocity, peak stress and elastic modulus. A box-type resistance furnace titled SX30/13Q-YC with silicon-carbon rod as the heating element and high performance fiber material for heat insulation was used to heat specimens. The temperature varied in the range of 20°C-800°C, and the heating velocity was 30°C/min in the testing furnace. Furthermore, the mechanical experiments are conducted in the uniaxial compression testing system.

The results show that the peak stress, elastic modulus and longitudinal wave velocity of these three types of rocks decreases greatly with suffered temperature. And the higher is the suffered temperature, the larger is the decrease of these parameters after temperature. However, because of difference in mineral compositions and micro-structures different rocks present different decrease rate of peak stress, elastic modulus and longitudinal wave velocity after high temperature. For tuff and granite, the peak strain increases significantly with suffered temperature, however, for breccia, the peak strain decreases slightly with suffered temperature. This indicates that the impacting mechanism of high temperature on different rocks is different. Furthermore, according to the type of rock the relationship among longitudinal wave velocity, peak stress and elastic modulus present different law. These relationships help to determine peak stress and elastic modulus through longitudinal wave velocity for different rocks. It is noted that comparing to tuff and breccia the decrease of longitudinal wave velocity of granite is greater than the decrease of the peak stress. It results from that the affection of high temperature on the physical properties of granite is stronger than the mechanical characteristics.

The results can be used to quickly determine mechanical and physical characteristics of rocks in field for safety evaluation and reinforcement of rock engineering after high temperature.

Keywords: Rock; High temperature; Peak stress; Peak strain; Elastic modulus; Longitudinal wave velocity.

EXPERIMENTAL STUDY ON THE PERMEABILITY OF SOFT ROCK

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The laboratory tests to investigate the permeability of brown mudstone and clayey siltstone in the complete stress-strain progress are introduced in this paper, including how the permeability characteristics are influenced by stress level and anisotropy of materials. In the test the instantaneous pressure pulse method was employed and the MTS 815.02 Rock Mechanics Testing System was used. The test results show that:

(i) For clayey siltstone, the permeability coefficient decrease with the increase of axial strain at the first stage of compression, then increase till the peak strain. The permeability coefficient of specimen vertical to bedding plane is larger than parallel to bedding plane, and the former maximal permeability coefficient is approximately 1.5 times of the latter.

(ii) For brown mudstone, the permeability coefficient decreases as the strain increases; after the stage of peak strain, the permeability coefficient decreases slightly and even fluctuates within a narrow range. There are not obvious difference in peak stress, peak strain and permeability coefficient on the orientation between vertical to bedding plane and parallel to bedding plane.

By the analyze of the relationship between the permeability and the strain of sever typical curves, it is concluded that:

(iii) The alternation of the permeability of clayey siltstone is correlative to the alternation of cubic strain. The relationship between the permeability coefficient and the cubic strain can be simulated as Eq. (1):

$$K = \begin{cases} K_0 \dfrac{1}{(1+\varepsilon_v)}(1+\dfrac{\varepsilon_v}{\phi_0})^3 & \textit{before peak stress} \\ \eta K_0 \dfrac{1}{(1+\varepsilon_v)}(1+\dfrac{\varepsilon_v}{\phi_0})^3 & \textit{after peak stress} \end{cases} \tag{1}$$

(iv) The alternation of permeability of brown mudstone is correlative to the alternation of the pore structure and the connectivity. The relationship between the permeability coefficient and the cubic strain can be simulated as Eq. (2):

$$K = K_0(e^{-218\varepsilon_a+2.015}+0.124) \tag{2}$$

The results of the tests can be a reference for further research in the field.

Keywords: Permeability; Soft rock; Anisotropy of stratum; Complete stress-strain progress.

REQUIREMENTS FOR ROCK STRESS MEASUREMENTS IN PRESSURE TUNNELS OF SEYMAREH DAM PROJECT

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One of the key factors for designing power tunnels is the prevention of hydraulic jacking in the surrounding rock. This problem is occurred when the water pressure within tunnel exceeds the in-situ compressive stress in rock. This may lead to the jacking of a large mass of rock away from the tunnel towards ground surface or adjacent underground space resulting in excessive leakage or large scale landslides. Due to the catastrophic and very expensive consequences of hydraulic jacking it is necessary to be ensured by adequate in-situ compressive stresses. For in-situ stress measurements in power tunnels, the method of hydraulic fracturing tests has been suggested as the most relevant tool. Seymareh dam and hydropower plant project is located in the western province of Ilam in Iran. Because of disastrous probable consequences of hydraulic jacking in the power tunnel of project and the high cost of steel lining on the other hand, it is intended to perform hydraulic fracturing tests in this project in order to measure in-situ stresses and determine the required length of steel liners. The power system contains three penstock tunnels each with internal diameter varying from 6.4m to 4.9m. Figure 1 illustrates the plan and longitudinal section of power system.

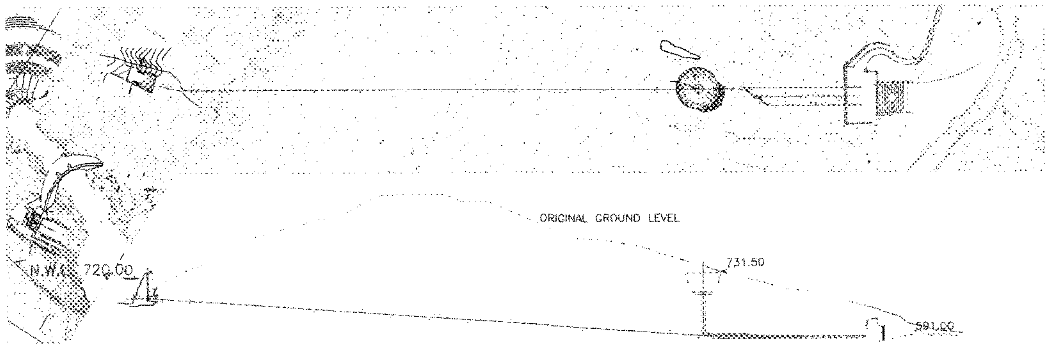

Fig. 1. Horizontal plan and longitudinal section of pressure tunnel in Seymareh dam project.

The conventional hydraulic fracturing tests are usually carried out in vertical boreholes. However, because of the available access tunnel in the downstream of power tunnel of project, it is planed to perform these tests on horizontal boreholes along penstock tunnels. In this paper the situation of power tunnel, the geological characteristics and the reasons for in-situ stress measurements by hydraulic fracturing tests in horizontal boreholes are explained.

Keywords: Hydraulic fracturing; hydraulic jacking; pressure tunnel; power tunnel; Seymareh dam.

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8. DISCONTINUITIES

8.1. General

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EFFECT OF EXCAVATION SEQUENCE AND FAULT ORIENTATION ON STRESSES AND DEFORMATION AROUND A CAVERN

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The presence of discontinuity in the form of joints and faults may have a significant influence on the deformation and stability of an underground opening. In addition, the stability of an underground opening may also be governed by the excavation sequence adopted for the construction. An investigation into these factors is therefore crucial to ensure that excessive stresses and deformation do not develop around the cavern, both during and after cavern construction.

A series of numerical parametric studies has been performed to investigate the effect of excavation sequence and fault orientation on the stresses and deformation induced around an underground cavern. The two-dimensional Universal Distinct Element Code (UDEC) was adopted in the investigation. The jointed rock mass was modelled as an equivalent rock material and the fault was modelled as a joint. The joint normal stiffness and shear stiffness in the numerical model was calibrated using data from published literature, and the general validity of the proposed numerical model has been affirmed.

The numerical results for various excavation sequences demonstrate that excavation sequence S3 is most favourable considering cavern roof and wall displacements only, whereas excavation sequence S6 produced the least number of plastic points around the cavern. A shear zone was formed near the right cavern wall at an angle of 43° to the horizontal, equal to the friction angle of the rock mass. As the fault orientation changes, the displacement changes with the fault orientation, being most severe at $\theta = 60^{\circ}$ and $\theta = 70^{\circ}$. The inclination of the shear zone near the right cavern wall changes with θ and appears to increase as θ increases. More shear zones appear between the fault and the left cavern wall as θ changes, with $\theta = 45^{\circ}$ showing two shear zones. In summary, excavation sequences have a marginal effect on the displacement and stresses around the cavern but fault orientation have a great influence on displacement and stresses around the cavern.

It has been shown from the study that the displacements and stresses induced around an underground cavern are highly affected by excavation sequence and fault orientation. The results obtained from this study are believed to be useful for the interpretation of field measurements and preliminary design considerations.

Keywords: Cavern; fault; UDEC.

IMPORTANCE OF INFILLED JOINTS IN SHEAR STRENGTH ASSESSMENT OF ROCK MASS

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Rock masses are typically characterized by joints, fractures and other planes of weakness. Infill material found in natural rock joints may cause a reduction in joint shear strength, influencing entire rock mass stability. In general, the shear response of infilled rock joints is controlled by the type and thickness of infill, the joint roughness, and the drainage conditions. An increasing degree of complexity is introduced into the problem, when the thickness of infill is less than the roughness amplitude of the wall rock.

This paper reports an experimental study to investigate the complex behaviour of infilled rock joints. A series of tests were conducted at the University of Wollongong (UoW) on idealised rock joints with different types of infill materials such as bentonite clay, natural silty clay, and graphite. Direct shear tests were carried out under Constant Normal Stiffness (CNS) in drained conditions for varying infill thickness to asperity height (*t/a*) ratios and initial normal loads. However, undrained conditions were employed in triaxial tests to study the effect of pore water pressure development under different confining pressures for varying *t/a* ratios. Test results show that the effect of asperities on shear response is significant up to a critical *t/a* ratio, $(t/a)_{cr}$, which falls between 1 and 2, whereas the shear behaviour is totally controlled by the infill alone beyond this critical *t/a* limit. However, the critical *t/a* ratio may vary with infill materials and test conditions. In addition, the triaxial test results illustrate the importance of pore pressure development in the shear strength assessment of rock mass. The normalized shear strength model developed at UoW successfully describes the observed shear strength of infilled idealised rock joints regardless of test conditions and joint-infill combinations. This model can be applied to various rock-engineering problems as long as the empirical constants can be evaluated by laboratory tests.

Keywords: Laboratory test, pore pressure, rock joints, shear strength.

UNSTABLE PHENOMENA AT THE FACE BASED ON THE QUANTIFICATION OF DISCONTINUITY IN ROCK MASSES FOR TBM EXCAVATION

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Hida tunnel is a part of the Tokai Hokuriku Expressway in Japan. The tunnel is composed of a main tunnel and an evacuation tunnel for use in an emergency. The length of the tunnel is approximately 10.7kilometers, Hida tunnel is the second longest road tunnel in Japan (eighth in the world). The main tunnel is excavated by means of the large section TBM with a diameter of 12.84m. The overburden is more than 1,000m, and in-situ earth pressure around the tunnel is assumed to be very high. Host rock is Nouhi-rhyolite, which was formed due to igneous activities about 64 million years ago. Due to the structural features of discontinuities and high earth pressure, the mass of rock falls out in the shape of a dome ahead of the face. In this paper, we closely observed the rock mass quality at the face and quantified discontinuitor by using the crack tensor theory to form the equivalent continuum for discontinuous rock mass. Then the three-dimensional FEM analyses have been performed so as to analyze the face stability. Results are compared with the real observed phenomena during the tunnel driving. They were in good accordance on the whole. Finally, we obtain a quantitive index for face stability in discontinuous rock mass.

Keywords: Discontinuity; rock mass; excavation.

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INFLUENCE OF THE ELASTICITY OF ROCK WALLS AT LARGE SCALE ON THE MECHANICAL BEHAVIOR OF ROCK JOINTS

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Rock joints are the weak points of rock masses, for this reason an understanding of them is needed in the field of civil engineering for the construction of underground structures, underground waste repositories and many other fields. The rock joints can be defined at three levels of scale: the "micro scale", which is the scale of the asperities (about a millimeter order), the "meso scale" (about a decimeter order) and the "macro scale" (more than one meter). In each case, their mechanical behavior appears to be different. This phenomenon is defined as scale effect. The "micro" scale is not included in this study.

At the "meso scale", the mechanical behavior of a rock joint is now experimentally investigated thoroughly by means of advanced shear apparatus. Numerical modeling of "meso" joint behavior is also progressing but still requires many improvements. At the "macro scale", which is the current scale in the natural environment, due to handling difficulties related to the sample size, getting results is more difficult. A possible lighting, not excluding experiments, on macro scale behaviour is the use of numerical modeling.

In this paper, the non linear finite element code Plaxis is used to model the mechanical behaviour of rock joints. The aperture distribution is modeled by the standard PLAXIS interface law. Plaxis is used to model rock joints of various dimensions, and consider the effect of rock wall elasticity at large scale. Numerical shear tests are performed at "meso" and macro scale. The mechanical behavior of rock joints is studied along a classical shear paths at constant normal stress path. The simulation process is detailed. Numerical results at large scale are investigated and important results are obtained, especially concerning the elasticity of rock walls at large scale.

The numerical modeling of shear at constant normal stress test for different scales showed that for very long macro joints, some "spring" effect (compression of rock walls at extremities) appears, even at high Young's modulus. This "spring" effect has not been mentioned so far in the literature. The initial shear stiffness displays an hyperbolic behavior, and a linear behavior in log log scale. Stiffness decreases sharply for rock joints of less than one or two meters and then decreases much more slowly and constantly for rock joints of greater length until finally almost vanishing for samples over 50 meters in length. The shear stiffness, not only decreases with an increase in scale, but also increases as the Young's modulus increases.

Keywords: Rock joint, scale effect, finite element modeling, shear stiffness, dilatancy.

SURROUNDING ROCK REINFORCEMENT OF UNDERGROUND POWERHOUSE BY JOINT MAPPING

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Random wedge stability is the most important problem in rock excavation engineering, but it is not easy to find the block which built by little joints in a surface of underground rooms. However, based on the field investigation and sampling on joint dip diction, dip angle, spacing and trace length, the probabilistic distribution function and associated mean and standard deviation may be interpreted, then, it is possible to simulate the joint distribution in engineering rock mass by the following two steps: *Step 1: Interpreting statistical parameters for each joint set.* The geometry of a joint is defined by its dip direction, dip angle, aperture and trace length. The frequency of occurrence of this set of discontinuity can be defined by another parameter, the spacing, and *Step 2: Simulating the jointed rock mass.* The second step is to create a 'map' of the rock mass cross section, which contains the joint web that bears the same statistic characteristics to those of the prototype. Via the joint mapping and block theory, the wedge distribution can be understood. When find the wedge, the stability can be analyzed by using Wittke's method. Furthermore, the optimizing anchorage scheme can be developed. The method of determining 3D joint persistence ratio determining and random wedge stability analyzing was discussed in this paper.

The block self-searching method based on 3D joint mapping was proposed by the authors, the steps which are required to deal with this problem are:

(i) From field joint geometry data collection, determine the average dip, dip direction, trace length and spacing of every discontinuity sets.

(ii) Mapping three-dimensional joint map by using Monte-Carlo simulation method.

(iii) If the rock mass was excavated, the trace will be appeared on the excavated surface, the trace can be determined.

When the wedges exposed on the excavated surface were searched, it is easy to calculate the stability factor by using the equilibrium method and blocky theory proposed by Goodman and Genhua Shi, but, in author's method, the persistence ratio was taken to calculate the wedge boundary shear strength.

According to the analysis results of the wedges, all the wedges' volume, the situation exposed on the excavated surface and the stability factors will be obtained, then, the stability factors' distribution will be gained, furthermore, the anchorage model can be determined by using optimism method. In fact, the wedges which the stability factor are more than an acceptable level, we can neglect these wedges, but for those stability factors are less than the acceptable level wedges, it is necessary to reinforce by anchorage.

As a practical example, Xilongchi pumping storage power-station underground surrounding blocks stability analysis and reinforce scheme are studied.

Keywords: 3D joint mapping; random wedge; stability analysis; simulated annealing method.

THREE DIMENSIONAL JOINT MAPPING AND ITS APPLICATION ON ROCK MASS SIMULATION

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It is well known that a rock mass is intersected by several sets of discontinuities whose geometry exhibits a random behavior. So, to determine the rock property is difficult by classic geology methods. When research on the discontinuities distributed in rock mass, it is impossible to describe every joint. However, for random behavior joints, it can be well understood in a stochastic way. Various mechanical properties exhibited by a rock mass, such as the shear strength, permeability, rock joint persistence, etc. can then be interpreted in a probabilistic approach which enables us to understand these mechanical properties by finding their *expectations* and the associated *degree of confidence.* The method is generally referred to as the *Monte Carlo Simulation* method and joint mapping method. The Monte Carlo method has been widely used in various disciplines of sciences in simulating random phenomenon. When used for simulating jointed rock mass, it includes some limitations. However, experience has shown that this technique did provide valuable information for understanding the fundamental behavior of rock mass, which otherwise can hardly be processed on a rational theoretical basis. When the simulated geometry data of joints were obtained by Monte-Carlo method, it is possible to map a simulated rock mass. Therefore, the shear strength, permeability can be easily determined by the map, furthermore, the stability of rock slope, the stability of surrounding rock of excavated room can be analyzed.

Based on the field joint collecting, the distributed model of joints can be founded by statistical method, furthermore, by using Monte-Carlo method, the joint map can be mapped. According to Monte-Carlo simulation method, three-dimension joint mapping procedure is put forward, a program called 3DPERC by using Visual C++ combined with Open GL technique was developed by the authors, it can display the joint network in three dimensions. Experience has shown that this technique did provide valuable information for understanding the fundamental behavior of rock mass, which otherwise can hardly be processed on a rational theoretical basis. The simulation steps can be divided into the followings:

(1) Determining the isotropic area of joints.

(2) Field sampling method for joints.

(3) Setting joints based on statistical methods.

(4) Establish a histogram for the samples.

(5) Fit the histogram by several probabilistic distribution functions and find one that best fit the samples.

(6) Give the statistic parameters, i.e., the mean and standard deviation, of the samples associated with the specified probabilistic distribution function.

(7) Joint mapping. In 3D joint simulation, volume density is defined as joint number in unit volume. Because it is impossible to get joint volume density by modern exploration method, the best way to obtain volume density is calculated based on line density or area density. The authors applied a complete random point method to simulate the location of one joint center point. To every joint set, the number of joint center points in the simulated volume can be calculated by random sample number of Poisson distribution.

Keywords: Jointed rock mass; random wedge; stability analysis; degree of confidence; joint mapping.

8.2. Theoretical and Numerical Analyses

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IMPROVEMENT ON SPACING SIMULATION IN 3-D NETWORK MODELING OF DISCONTINUITIES IN ROCKMASS

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Network modeling for discontinuities in rockmass is a new technique developed recently, which is a very important work to evaluate the stability and obtain geometric parameters for engineering design in jointed rockmass. The technique is based on the in-situ investigation of discontinuity traces in sampling window or line. With probabilistic statistics, stochastic, Mont-Carlo simulation and computer programming, 2-D or 3-D numerical network model for discontinuities can be formed.

According to a large amount of survey, discontinuities are shown to have the character of alternation between sparseness and denseness but all not uniformity distribution. In the past, the coordinate values of the discontinuity center points were modeled according to uniformity distribution, which led to the simulate results of the spacing with uniformity distribution as well. It bought on the distortion of simulation results.

In this paper, a new method of spacing modeling with Monte-Carlo principle is introduced. By the method, the discontinuity spacing is simulated according to its statistical model coming from rockmass survey and all are not uniformity distribution. Obviously, this modeling results are closer to the fact.

In the new modeling method, the coordinate system is rotated to make the new z-axis parallel the normal line of discontinuities. In new coordinate system, the z coordinate value of the discontinuity center point is modeled according to its statistical model, while the coordinate values of x and y according to uniformity distribution. After the uniformization of z coordinate value, the new coordinate values of x, y and z are calculated from these coordinates under the new frame. In order to ensure that the spacing of the simulation network is consistent with its preconcerted value, it is necessary to check the spacing through adjusting the number of discontinuities. It is proved that the discontinuity spacing of the simulate results can fit its statistical model by this new method.

The study shows that the modeling results obtained from the new method are closer to the fact. The defect of traditional spacing modeling is remedied with the improvement method. It is of good value in the study the engineering geologic characters and the settlement the engineering geologic problems of rock mass.

Keywords: Spacing; network modeling; discontinuity; uniformization; rotating of coordinates frame.

NUMERICAL MODELING OF SHEAR BEHAVIOUR OF INCLINED SAW-TOOTH MUDSTONE-CONCRETE JOINT USING FLAC

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Rock socketed piles are commonly used in Melbourne mudstone to support large loads from the superstructure. The shear behaviour of concrete-mudstone joints has been investigated extensively by the Monash Geomechanics Group since 1980s. This research has led to the development of the commercial computer program, ROCKET, for predicting the side resistance of rock socketed piles. However, due to the need for an efficient and economic foundation system, an alternative foundation is always on the agenda. Tapered piles, which are good in frictional ground, are considered to be a solution for this problem due to its superior load carrying capacity. A continuous taper along the entire length of the pile forces the ground to expand radially as the pile moves downward, thereby increasing the side resistance. In this paper, the performance of an inclined concrete-mudstone joints resulting from a tapered section will be evaluated numerically using FLAC program. A saw-tooth asperity was modeled for the simplicity of analysis.

The shear stress of a joint (τ) with regular triangular asperities (i=asperity angle) can be estimated reasonably well by $\tau = \sigma_n \tan(\phi_b+i)$, where σ_n= normal stress and ϕ_b= basic friction angle of the joint. However, for an inclined joint, as found in a bored tapered rock socketed pile, this equation is no longer applicable. Considering a joint inclination angle of ω, the side shear resistance of a regular saw-tooth joint can be estimated by $\tau = \sigma_n \tan(\phi_b+i+\omega)$. Prior to investigate the inclined joint behaviour, a bench mark model is established to simulate the laboratory test results of concrete-Johnstone joints. Following this, the numerical model is extended to simulate the inclined joint behaviour using different joint inclinations (ω = 1, 3 and 5 degrees) under the constant normal stiffness condition. Modeling results indicate that the effect of inclined joints in mudstone could be modeled reasonably well using the proposed equation. The shear strength of inclined joints is observed to increase with the increase in joint inclinations (Figure 1) by as much as 40% for a joint inclination of 5°.

Keywords: Numerical model; shear strength; inclined rock joints.

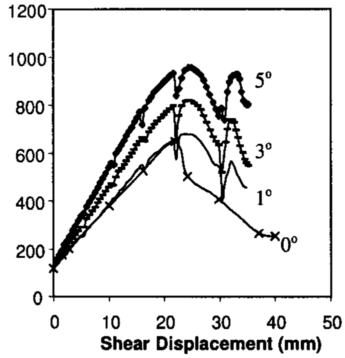

Fig. 1. Shear behaviour of inclined joints with various joint inclinations.

UNIFIED SHEAR MODEL FOR ROCK JOINTS

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The shear strength of rock joints is assessed by means of direct shear tests. The common procedure of these tests usually comprises the sliding of a joint sample under 4 or 5 different normal stresses. Between each joint sliding, the joint is cleaned of the small rubble that the wear during shear has caused and positioned in its original natural (generally mated) position. The increasing order by which the different normal stresses are applied is intended to minimize the wear of the joint surfaces and the resulting decrease in shear strength. The results of each rock joint test consist of the 4 or 5 slidings of the same joint and can be plotted as a shear stress shear displacement graph.

The analysis of a large amount of this type of graphs showed that all curves referring to the same rock joint tested under different normal stresses are similar. Moreover, this fact could be found in sets of tests different rock lithologies performed under quite different ranges of normal stresses: tests under low normal stresses 20 to 160 kPa and higher normal stresses 0.5 to 4 MPa.

The result of each one of the slidings of a rock joint test is the shear strength τ_{res} under the given normal stress. Defining the normalized shear stress as τ/τ_{res} a unified shear model for rock joints based on the normalized shear strength vs shear displacement graphs (Fig. 1), which presents the major advantage of being independent of the normal stress.

The paper also discusses the test procedures and standard methods and describes the results that can be drawn from the tests. It is noted that the actual ISRM and ASTM procedures require some updating, namely, in using the same joint to determine the shear strength under different normal loads, and in the application of the normal prior to each sliding.

Keywords: Rock joint; shear strength; direct shear test.

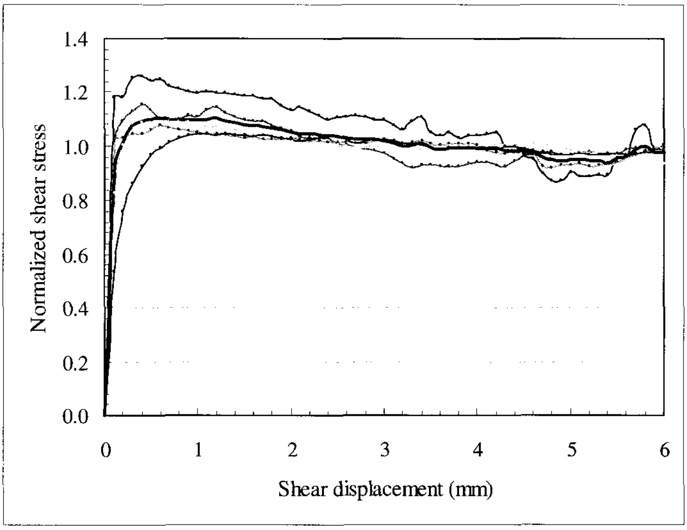

Fig. 1. Normalized shear stresses.

NEW CONSIDERATIONS ON ROCK LOADS FOR MINED TUNNELS

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A mined (sometimes called a "bored") tunnel is an underground opening that is excavated underground, as opposed to, for example, a cut-and-cover tunnel that is built in an open (surface) excavation and afterwards backfilled. One of the most important aspects in mined tunnel design is the design of the support system. Hoek and Brown (1980) state the following. "The design of underground excavations is, to large extent, the design of underground support systems. These can range from no support in the case of a temporary mining excavation in good rock to the use of fully grouted and tensioned bolts or cables with mesh and sprayed concrete for the support of a large permanent civil engineering excavation. These two extremes may be said to represent the lower and upper bounds of underground support design."

A support system usually needs to be designed to carry three primary loads: (1) dead load (self-weight) of the support itself; (2) ground load; and (3) groundwater load/pressure, should this be present. In addition, there may be secondary loads such as live loads, construction loads, handling loads, etc. These loads, with the exception of the ground load, can, in general, be readily and reliably quantified. (Earthquake loading is also complex, but this paper does not consider this special situation.) It is imperative when making decisions on support requirements that all the pertinent aspects be considered and assessed before initial support is specified. These aspects would include at least the following: geology, in particular the jointing, in-situ stress (or depth of the tunnel), required support pressures, displacements of the tunnel walls and depths of overstressed rock (depths of "plasticity"). Detailed geological mapping followed with calculations, using numerical models, is required to quantify all these aspects. Once quantified, final support recommendations that are consistent in all respects can be made.

The advent of more powerful numerical models and faster computers, particularly over the last ten years, has allowed support design engineers to investigate novel ways of determining, indirectly or directly, ground loads. In this paper, the authors present some new considerations that have arisen from these studies with which they have been involved during recent practical design.

The estimation of the rock load in mined tunnels in rock masses is a fundamental requirement for the design of the support systems. Traditionally, empirical methods such as rock mass classification systems are used to provide the required support pressure, the required support system or both. Two well-known methods are the NGI (Barton/Q) and Geomechanics (Bieniawski/RMR) systems. This paper examines some aspects of these systems in the light of recent experiences by the authors. The approach to support estimation using numerical models in comparison with the Barton system is also presented. The paper concludes that empirical approaches should be used with caution and by experienced persons and the support recommendations given by empirical methods should be checked with simple two-dimensional numerical models.

Keywords: Mined Tunnel; Numerical; Empirical; Rock Load; RMR; Barton/Q.

RESEARCH ON COUPLED PENETRATING-DISSOLVING MODEL AND EXPERIMENT FOR ROCK SALT CRACK

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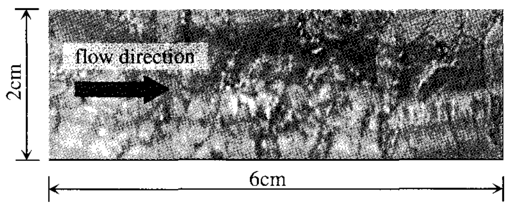
Efforts of this paper mainly focus on the coupled action between dissolution and permeability variation of single rock salt crack. Firstly, dissolving process of rock salt crack is theoretically simplified based on Fick's law and dissolving mechanism of rock salt for the sake of convenience of theoretical analysis. Then, a coupled penetrating-dissolving model for rock salt crack is set up with dissolution and permeability variation of rock salt crack considered simultaneously.

A device for coupled penetrating-dissolving test for rock salt crack is developed by the authors, and the test for coupled penetrating-dissolving process of rock salt crack is performed under certain conditions. The dissolved surface of rock salt crack is shown in Figure 1. The proposed coupled penetrating-dissolving model is then used to simulate the coupled penetrating-dissolving process under the same conditions, and comparison of the average dissolved thickness of rock salt crack surface between test and calculated results is shown in Figure 2. It can be found that the calculated result perfectly agrees with that obtained by test.

The coupled penetrating-dissolving model proposed in this paper can provide an important reference to the further study on coupled mechanical-penetrating-dissolving behavior of rock salt and analysis on the stability of underground salt caverns.

Keywords: Rock salt crack; coupled penetrating-dissolving model; experiment; simulation analysis.



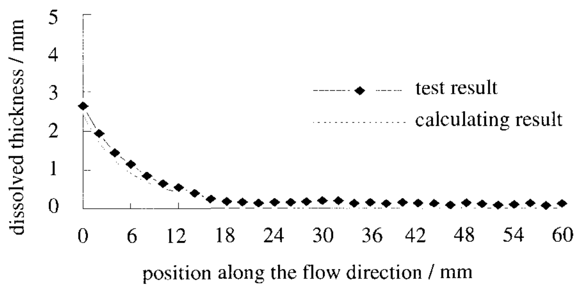


Fig. 2. Comparison of the average dissolved thickness of rock salt crack between test and calculated results.

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8.3. Field and Laboratory Studies

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HYDROMECHANICAL BEHAVIOR OF ROCK JOINTS BY ROTARY SHEAR-FLOW TEST

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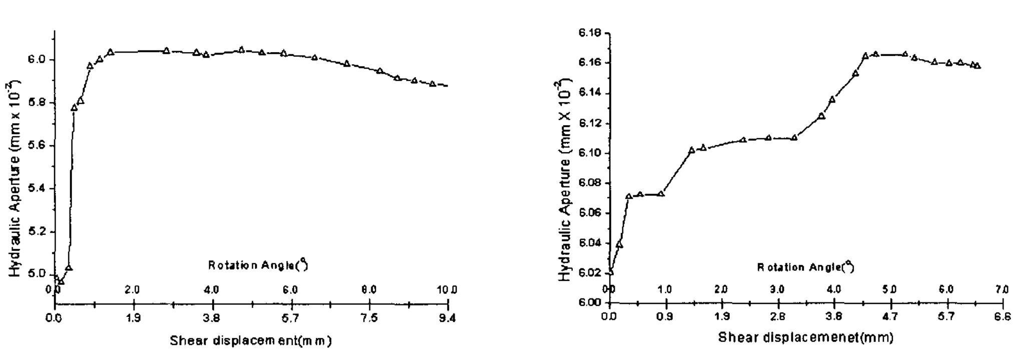
The rotary shear-flow test has the merits to allow the radial water flow from inner to outer of hollow cylinder specimen with a joint and to keep the constant area of shear surface for the large shear movement. In this paper, the improved rotary shear-flow test was carried out to investigate the coupling between the mechanical behavior and hydraulic conductivity of rock joints under the low normal stress of 0.1 MPa. The water flow rate, normal and shear load, normal displacement and rotated shear angle were measured every half second simultaneously for the hollow cylinder type replica of natural joints under the constant water head without any stopping until the end of the test by using a flow meter of turbine type (18.9~454.2 cc/min.) and data logging system. It was found that mechanical aperture continuously increased with the formation of voids immediately after the peak of shear stress, hydraulic aperture seemed to be affected by the geometric factor related to the formation and propagation of voids. These were analyzed by the occurrence rates, mean size and connection of voids on the shear surface as introduced in this paper. The key point was that the peak of hydraulic conductivity happened when the occurrence rates and mean size of voids were not the maximum. It is concluded that the hydraulic conductivity is controlled by the formation of flow channel related to the distribution and connection of voids. Figure 1 shows the typical behavior of hydraulic aperture.

Keywords: Rotary shear test; Joint roughness; Single joint model; Mechanical aperture; Hydraulic aperture.



(i) Replica M1 (JRC = 12.13)

(ii) Replica M7 (JRC = 5.66)

Fig. 1. Hydraulic aperture in the rotary shear displacement.

STUDY OF THE INTERACTION BETWEEN HYDRAULIC FRACTURES AND GEOLOGICAL DISCONTINUITIES

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Hydraulic fracturing is applied in a wide range of areas that cover petroleum and water well stimulation, disposal of waste underground and recently in modifying rockmass strength for mining. Regardless of the area of application, the hydraulic fractures often grow through and interact with pre-existing geological discontinuites.

Fracture initiation and propagation in the presence of pre-existing geological discontinuities is a topic that has been widely studied, both theoretically and experimentally. Criteria to predict fracture crossing or arresting at discontinuities have been developed. For designing purposes, a better understanding of how an induced fracture interacts with a pre-existing geological discontinuity is fundamental for predicting the ultimate size and shape of the hydraulic fractures formed by a treatment.

We have carried out laboratory experiments to further study the effect of pre-existing geological discontinuities on the hydraulic fracture growth. Our experimental results are compared to two published crossing criteria (Blanton, 1986; Renshaw and Pollard (R&P), 1995) and are being analysed using a numerical model (Zhang et al., 2005; Zhang et al., 2004).

The experiments consist of growing hydraulic fractures through sets of orthogonal frictional interfaces subject to controlled normal stresses. Sets of small rock plates were tested to establish stress and fluid viscosity conditions that produce crossing interactions. These initial tests were then followed by larger scale tests using biaxially loaded plate systems. The main purpose of the experiments was to measure stress, strength and fluid viscosity conditions that result in hydraulic fracture crossing, offsetting, or arresting at pre-existing discontinuities. Experiments were run for conditions where the energy to propagate the hydraulic fracture was dominated either by fracture toughness or by viscous dissipation. The final shape of the fracture was determined for each test. An additional issue being addressed by our study was the mechanism for reinitiation of a hydraulic fracture in an unfractured material or at existing flaws.

The sample preparation step highlighted the fact that curvature and local surface imperfections in the rock slabs tested, result in stress concentrations that can dominate crossing behavior. From our results and from mapping and measuring fracture growth in field experiments, we believe that R&P's criterion provides a lower limit on interface strength or stress conditions that result in hydraulic fractures crossing geological discontinuities. For hydraulic fracture growth, R&P predict crossing to occur for cases where we observe the hydraulic fracture arresting.

Additionally, we postulate that a hydraulic fracture in a naturally fractured rock will tend to grow more rapidly in the direction of the maximum than in the minimum in-plane stress, resulting in an elliptical shape to the fracture from this effect alone. We have demonstrated by laboratory experiments, a retarded but stress-dependent growth in the direction associated with fracture propagation through interfaces. As normal stress acting on the interfaces was increased between tests, the ultimate fracture extent in that direction becomes larger; which supports this hypothesis.

Keywords: Discontinuity; interaction; hydraulic fracture.

EXPERIMENTAL STUDY AND NUMERICAL MODELING OF DIRECT SHEAR TESTS OF ROCK JOINTS UNDER CONSTANT NORMAL STIFFNESS

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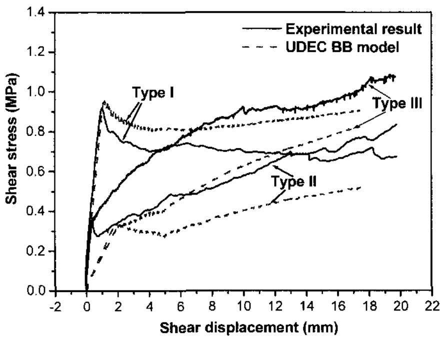
In this study, direct shear tests were performed on replicas of artificial tension joint surfaces casted in cement mortar under constant normal stiffness (CNS) condition. The servo-controlled shear apparatus is used to apply varying normal loads in response to dilation. From the experimental results, three types of typical shear behaviors under the CNS condition were defined. Experimental behaviors of rock joints were modeled by using Universal Distinct Element Code (UDEC) and the mobilized dilation routine of the JRC-JCS model. Comparisons between the predicted behavior and experimental results showed a good agreement.

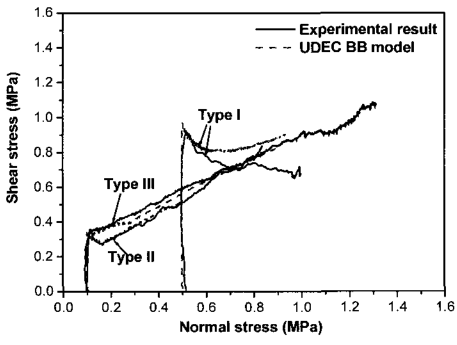
Figure 1 shows graphs of the simulated shear stress vs shear displacement and the shear stress vs normal stress using the UDEC BB model together with the experimental data for all types of shear behavior. Post-peak shear stress in Type I is overestimated in the UDEC model because of the values used for calculating mobilized roughness in the UDEC BB model. In figure 2 (a), numerical results are not in good agreement with the experimental results because numerical peak shear stress is observed to occur at a larger shear displacement than obtained from laboratory tests. In figure 2 (b), however, the trend of the numerical shear stress vs normal stress is similar to the experimental results in Type II and Type III. These differences occur because peak shear displacement depends only the joint length in the UDEC BB model

Because shear stress of Type I after the peak stress in BB model was overestimate, mobilized JRC may be reassessed under the CNS condition. Mobilized JRC was modified by using the routines of JRC-JCS model under CNS condition for Type I. JRC-JCS model shows good agreement with laboratory test better than UDEC model for the Type I case.

Keywords: Direct shear test; rock joint; stiffness.



(a) shear stress – shear displacement



(b) shear stress – normal stress

Fig. 1. Numerical UDEC modeling and experimental direct shear test under the CNS condition.

CROSSING OF FAULT ZONES IN THE MFS FAIDO BY USING THE OBSERVATIONAL METHOD

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The presentation is dealing with the crossing of severe fault zones in the multi-functional section Faido, part of the Gotthard Base Tunnel. The excavation procedure and the support system are specified over the combination of geotechnical computations and the observational method.

During the excavation work on the section Faido of the Gotthard Base Tunnel large, not prognosticated fault zones in the range of the multi-functional section (MFS) Faido were found. Due to the very unfavourable situation of the fault zones the entire tunnel system of the MFS had to be re-arranged during the running excavation works. The underground structures with very large cross sections for the two tunnel changes were shifted to the south.

Concurrent to the new layout arrangement of the MFS the support system was reviewed under the criteria of the changed ground conditions. For the north-west tube a rigid construction with HEM steel girders was selected. In spite of this measure, drastic clearance problems arose within larger ranges. Also, the deformations could not be controlled. For this reason the excavation was finally stopped and it was decided to re-excavate the HEM section with an enlarged profile. The support system was changed to a flexible steel construction in combination with shotcrete with convergence slots. This method is now being used successfully.

With the occurrence of the unexpectedly squeezing ground behaviour the original computations and dimensioning based on rock parameters and the characteristic line curve were carefully analyzed and verified with the help of the Geological Strength Index and by back analysis. The results of all considerations and computations show that different methods can lead to completely different results. None of the methods has proved reliable to seize the various influences from the fault zones close-to-reality. Under these conditions, geotechnical computations can give assistance but may not be regarded as sole source for the dimensioning of the support. For the excavation of the further tunnel sections in the MFS Faido, geotechnical computations are complemented with input from the observation method. Important thereby is the current observation of geology on site together with the experiences (observations, measurements) from the last excavation steps.

Keywords: Gotthard Base Tunnel, MFS Faido, fault zone, geotechnical computation, observational method.

ESTIMATION OF PERMEABILITY STRUCTURE OF THE MEDIAN TECTONIC LINE FAULT ZONE IN OHSHIKA-MURA, NAGANO, JAPAN, BY USING LABORATORY TESTS UNDER HIGH PRESSURE

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Regions of which tectonic activity is relatively high generally contains faults in depth, and these faults possibly affect the regional groundwater flow. Fault zones exhibit a wide range of fluid transport properties in comparison to their host rocks because of the complicated and variable microstructure patterns, and each fault zone has largely heterogeneous distribution of hydraulic properties. Therefore, in order to examine a fluid circulation through a domain including faults, it is often important to understand the hydraulic properties of a single fault zone. This paper reports the results of laboratory permeability measurements of fault rocks and their host rocks obtained from the Median Tectonic Line, the largest strike-slip fault in Japan.

The measurements are made by using a gas-medium apparatus under confining pressure up to 180MPa, which simulates in situ conditions. Samples of fault gouge, cataclastic mylonite, and protoliths (Tonalite and pelitic shist) were collected from the Kitagawa and Ankoh outcrops of the MTL and adjacent areas in Ohshika-mura, Nagano Prefecture, central Japan. Permeabilities of these samples were measured under hydraulic pressure at dry conditions, with nitrogen as the pore fluid. Most samples from the incohesive zone have a permeability ranging between 10^{-13} to 10^{-17} m^2. These permeabilities are greater than those of cemented cataclasites and mylonites by more than 2 orders of magnitude at all effective pressures up to 180 MPa. Clayey fault gouge material has permeability as low as 10^{-19} m^2 at high effective pressures, but such impermeable fault gouge does not constitute a continuous zone on the two outcrops we studied. The resulting permeability architecture of the fault zone is very heterogeneous. We also carried out permeability measurements of fault gouge and cemented cataclacite specimens during tri-axial compression tests. The experiments revealed marked effects of deformation on the permeability of the specimens, and the effects of fault gouge and cataclacite were very different from each other. Thus, overall permeability structure of a fault may change abruptly prior to or during fault activities and during the inactive period.

Keywords: Permeability; Fault Zone; the Median Tectonic Line; Laboratory Permeability Measurement; Deformation Test.

EFFECT OF DILATION ANGLE ON FAILURE MODE AND ENTIRE DEFORMATIONAL CHARACTERISTICS OF ROCK SPECIMEN

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FISH functions were written to calculate the axial, lateral and volumetric strains as well as the ratio of negative lateral strain to axial strain (called the calculated Poisson's ratio in plane strain compression, which is different from Poisson's ratio in elastic stage) of rock specimen with a material imperfection closer to the base of the specimen in plane strain compression.

The influence of dilation angle on the pattern of shear band and the entire deformational characteristics was numerically investigated by use of FLAC.

In elastic stage, the adopted constitutive relation of rock was linearly elastic. Beyond the peak stress, a composite Mohr-Coulomb criterion with tension cut-off was used and the post-peak constitutive relation was linear softening.

Numerical results show that the increasing dilation angle leads to an increase of the number of yielded elements initially occurring in the vicinity of the material imperfection. The transition of failure mode from single shear fracture to conjugate shear fracture is observed as dilation angle increases.

The measured shear band's inclination angles at different dilation angles are even closer to the theoretical predictions by Arthur's theory. Higher dilation angle results in wider shear bands, as is in agreement with the previous formula proposed by Wang et al. (2004) considering shear dilatancy based on gradient-dependent plasticity.

As dilation angle increases, the peak stress, the axial and volumetric strains corresponding to the peak stress as well as the absolute value of the lateral strain corresponding to the peak stress increase. At post-peak, higher dilation angle leads to less steep stress-axial strain curve and stress-lateral strain curve. According to the previously analytical solutions by Wang & Pan (2003), the increasing number and thickness of shear band are responsible for the ductile post-peak behaviors. For higher dilation angles, rock specimen can reach larger lateral expansion at the same axial strain so that higher Poisson's ratio can be expected and the deformed volume of the specimen can exceed the initial volume.

Beyond the peak stress, the slopes of the lateral strain-axial strain curve, the Poisson's ratio-axial strain curve and the volumetric strain-axial strain curve are not influenced by dilation angle. Rock specimen with the highest dilation angle can reach the minimum volume in compression.

More apparent precursors to the unstable failure of rock specimen can be observed for higher dilation angles.

Keywords: Shear dilatancy; shear band; compressive stress; lateral strain; volumetric strain; Poisson's ratio.

9. BLOCK THEORY AND DDA

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VIBRATION ANALYSIS OF LAMINATED BLOCKS BY DISCONTINUOUS DEFORMATION ANALYSIS

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An earthquake is one of the triggers of rock slope failure and rockfall which cause heavy damage to human beings. In order to conduct a safety assessment for the sake of protecting human beings against these disasters, simulation of the behavior of rock masses during earthquakes is necessary. The purpose of this study is investigation of the applicability of discontinuous deformation analysis (DDA), which can analyze accurately static and dynamic problems and is expected to come into practical use, to vibration analysis and its characteristics. By comparing vibration response of the laminated blocks in the shaking table tests with results of simulation by DDA, the applicability was proved and knowledge about applying DDA to vibration analysis was acquired.

First, the shaking table tests were performed to study vibration response of simple laminated blocks. Two types of laminated blocks were used, which had different shapes and physical property values. Some sinusoidal input motions which had different frequency and amplitude were used. From these tests, it was proved that the blocks had various behaviors, stability or toppling, depending on input motions and the modes of failure were not only sliding but rotation.

Next, the behaviors of blocks in the tests were simulated by using DDA. In the shaking table tests, contact force between blocks was mainly frictional force and it is very important to represent frictional force accurately in vibration analysis. Thus, we performed basic analysis before simulation of the shaking table tests so as to set appropriate contact spring stiffness. In this analysis, we compared results of DDA to an analytical solution which was developed here for the response of a single block resting on a block which vibrated horizontally. In consequence, if contact spring stiffness is lower than thousandth part of the Young's modulus, the error between DDA results and analytical solution of horizontal displacement of the block is small. But if contact spring stiffness is larger than that value, the error is very large and frictional force doesn't work appropriately. Next, in consideration of contact spring stiffness from this analysis, we simulated the behaviors of laminated blocks in the shaking table tests. In the result, it is proved that with soft contact springs DDA can simulate the behaviors of blocks, stability or toppling, depending on input motions of the tests. And horizontal displacement and acceleration spectrum of blocks can be expressed well by DDA.

This study proved the applicability of DDA to vibration analysis and its characteristics by comparing simulation by using DDA with the results of the shaking table tests. Applying DDA to vibration analysis, soft springs is needed to be applied so as to make contact force (mainly frictional force) work accurately. By using soft springs whose stiffness was lower than thousandth part of the Young's Modulus of blocks in this study, DDA could simulate well vibration response of the blocks in the experiments. Therefore knowledge about applying DDA to vibration analysis was acquired and DDA proved to be a powerful tool for it. The authors plan to collect and assemble analytical data in order to make it possible to set quantitatively analysis parameters.

Keywords: DDA; vibration analysis; dynamic problem; shaking table tests.

BLOCK REMOVABILITY ANALYSIS OF A ROCK SLOPE USING STATISTICAL JOINT MODELING

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Removable blocks of a rock slope are made by intersection of joints and a slope face. Sliding and falling down of the removable blocks can endanger human lives and properties. Block theory has been used to discriminate the removable blocks and analyze their stability. This theory, however, uses simplified joint information and therefore, only provides rough analysis result such as the maximum block occurrence area.

In this study, rock joints were statistically modeled: probability distribution functions of the joint orientation and size, and volumetric frequency were estimated from a window sampling data. Rock blocks whose faces consisted of the modeled joints and a slope face were simulated and their removability was tested. All of the analysis process was coded into a computer program and each algorithm constituting the program was verified by comparing its result with theoretic solutions.

In many cases, slope collapse is triggered by an excessive hydraulic pressure due to an intensive and/or continuous rain fall in a monsoon season. This shows that the water saturation of the slope and its discontinuities lowers the rock block safety of the slope. Therefore the groundwater effect on the slope stability was also tested in this study.

The developed analysis tool was applied to a rock slope located in Kwangyang, Chonam, Korea. Through the removability analysis of rock blocks, volume, maximum depth and occurrence location of each block were obtained.

Keywords: Block theory, rock slope, stability analysis, statistical joint modeling, groundwater.

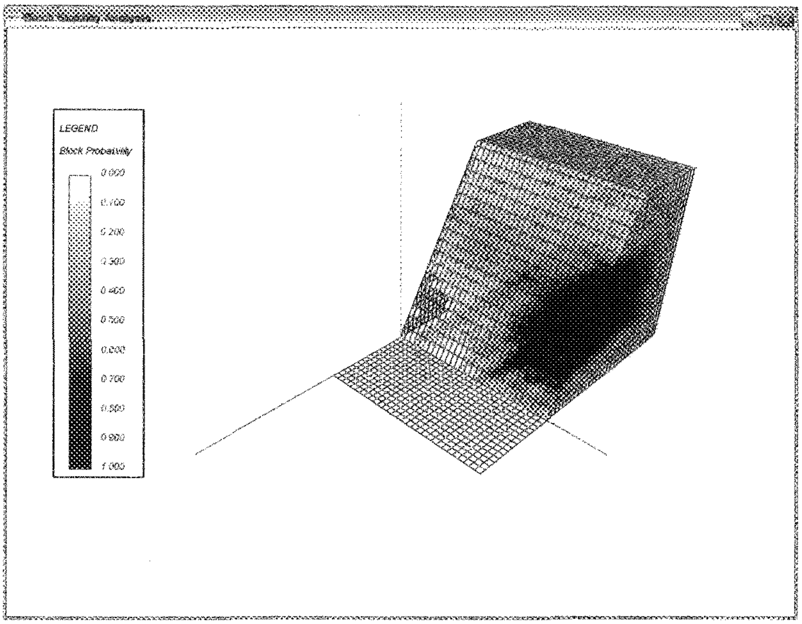

Fig. 1. Occurrence probability of removable blocks in a rock slope.

SEISMIC RISK DETERMINATION USING NUMERICAL ANALYSIS OF BLOCK DISPLACEMENTS IN HISTORICAL MONUMENTS WITH DDA

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The numerical Discontinuous Deformation Analysis (DDA) method was used for back analysis of structural failures in archaeological sites along the active Dead Sea rift system in Israel and preliminary constraints on historical seismic ground motions were obtained.

Two case studies are presented in the paper, in which historic masonry structures were modeled and both synthetic and real earthquake records were applied as loading functions. The response of the structures was studied up to the point of incipient failure in a mechanism similar to the one observed in the field. In both case studies the dynamic analysis was found to provide more complete and accurate results than the pseudo-static solution. Therefore, we believe that such an approach can be employed, where relevant, to provide constraints on paleo-seismic ground motions and consequently on expected PGA values in seismically active regions.

Keywords: Seismic Hazard, PGA, DDA, Masonry Structures, Paleo-seismicity.

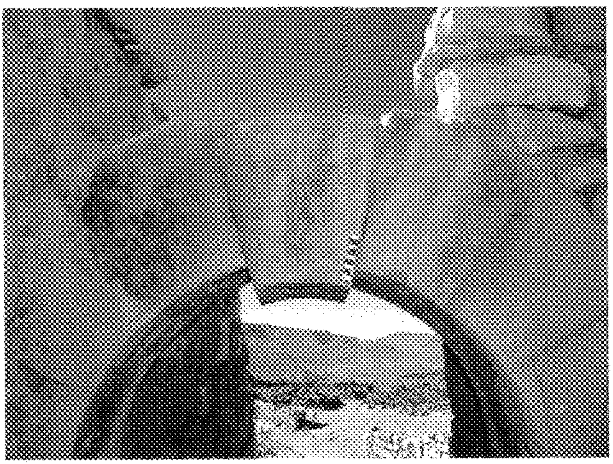

Fig. 1. An example of a 2000 year old masonry arch which displays a paleo-seismically induced, measurable, block displacement suitable for back analysis with DDA.

DETERMINATION OF BLOCK SIZES CONSIDERING JOINT PERSISTENCE

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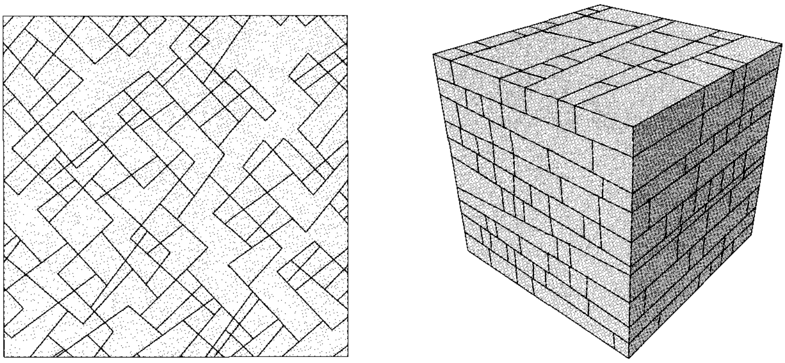
Although the importance of joint persistence on the overall rock mass strength has long been identified, the impact of persistence on rock strength is in most current rock mass classification systems underrepresented. If joints are assumed to be persistent, as is the case in most designs, the sizes of the rock blocks tend to be underestimated.

In this study, UDEC and 3DEC are used to generate block systems delineated by joint sets with different persistence. The objective is to statistically analyze how the joint geometric parameters, such as joint orientation, spacing, and persistence, will affect the size of each individual block and average size of the block system, with the focus on the study of the influence of joint persistence on the determination of the block sizes. Fig. 1(a) presents an example showing the blocks generated by discontinuous joint sets in two-dimensions. Fig. 1(b) shows an example of the blocks delineated by one continuous joint set and two discontinuous joint sets. Based on the experimental design, blocks systems with various joint persistence factors, spacing, orientation, etc., are generated. It is found that the probability density functions of the block area and volume obey the lognormal distributions, which agree with the conclusions reached by other investigators. According to the result of the ANOVA analysis, the persistence factor's contribution to the statistical significance of the block size, is higher than that of the angle between joint sets, indicating that it is important to consider joint persistence in a rock mass classification program. Using equation (1), the effect of joint persistence on the block volume delineated by discontinuous joint sets can be quantified, and approximate equivalent block sizes of jointed rock masses that contain discontinuous joints can be obtained. In this fashion, the interlocking due to the presence of discontinuous joints can be indirectly considered by increased block size in the GSI system.

Keywords: GSI, block size, persistence, UDEC, 3DEC.

$$V_b = 0.998\frac{s_1 s_2 s_3}{\sqrt[3]{p_1 p_2 p_3}} \approx \frac{s_1 s_2 s_3}{\sqrt[3]{p_1 p_2 p_3}} \qquad (r^2 = 0.911, \quad p_i < 1) \qquad (1)$$



(a) (b)

Fig. 1. Examples of blocks delineated by discontinuous joint sets: (a) 2D; (b) 3D.

NUMERICAL MANIFOLD METHOD OF THE POTENTIAL PROBLEM FOR THE GROUNDWATER FLOW

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The numerical manifold method is a newly developed computational approach for blocky systems that is applicable to general continuous and dis-continuous media. The method is derived from the finite cover approximation theory and gains her name after the mathematical notion of manifold. It demonstrates a good consistency with both the conventional finite element method and the discontinuous deformation analysis. Because of this discrete element root and the unified mathematical framework, the manifold method can be used both in integrating the continuum and in discrete analysis. The manifold method is more suitable than other numerical methods for problems with discontinuous and moving boundaries such a crack development and free surface flow. Usually, the governing equations of the numerical manifold method are derived from the minimum potential energy principle. For many applied problems, e.g. heat conduction and potential flow, it is very difficult to derive in general outset the functional forms of the governing equations. This obviously strongly restricts the implementation of the minimum potential energy principle or other variational principles in NMM. In fact, the governing equations of the NMM can be derived from a more general method of weighted residual (MWR). And then the minimum potential energy principle or the variational principle can't be used to derive the governing equations of the numerical manifold method. For the potential problem of the groundwater flow, the governing equations of the numerical manifold method are derived by the method of weighted residuals (MWR). Laplace's equation is common field governing equations to describe various physical natures. These differential equations can represent heat conduction and potential flow, etc. Therefore, the manifold element formulation of the Laplace equation paves the way for the application of the NMM to a wider range of problems. The numerical manifold method of Laplace equation is presented in the paper. The numerical manifold methods of the potential problems are derived by the method of weighted residuals. This is demonstrated by the derivation of the governing equations of the NMM for one typical case in the paper: the Laplace's equation for which the NMM cannot be derived from the minimum potential energy principle. The method developed in this paper is more general than that of the minimum potential energy principle. In the method of weighted residuals, the choice of weight function is abundant and convenient. Using different weight function, different governing equations of numerical manifold method can be formed. The method developed in this paper enriches the mathematical foundation of the NMM and extends its field of applications. At last, the working of the method is illustrated with a numerical example of the seepage. The method developed in this paper is more general than that of the minimum potential energy principle. The work developed in this paper enriches the mathematical foundation of the NMM and extends its field of application.

Keywords: Numerical manifold method; method of weighted residuals; finite cover technology; potential problem; seepage.

COUPLING OF CERTAIN AND STOCHASTIC DISCONTINUITIES IN 3-D DISCONTINUITY NETWORK MODELING

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The conventional modeling method of discontinuities network has very important function in studying the engineering geology characters and settling the engineering geology problems of rock mass, but it is based on the probability and the uncertain is its outstanding character. So, it leads to that the discontinuity's network from random modeling isn't the same as actual distributing of discontinuity. So the application of the conventional modeling method is limited.

The study shows that the statistical rules of modeling results are the same as those of actual discontinuity, but the position, size, orientation and etc of every discontinuity are inaccurate. In this paper, the discontinuity that lies on the outcrop surface and can be measured is defined as "certain discontinuity", and the discontinuity coming from random modeling of discontinuity's network and can't be measured is defined as "stochastic discontinuity". The coupling modeling method between certain discontinuity and stochastic discontinuity is introduced emphatically.

In this paper the principles of the coupling simulation are discussed. These principles include:

- the coupling principle of same team discontinuities;
- the coupling principle of equation amount discontinuities;
- the priority coupling principle of nearest distance;
- the coupling principle of the outcrop surface;
- the size and the coordinate of centre point of discontinuity are random.

According to these principles, the coupling steps are introduced in this paper. Several steps are very important and different with the conventional modeling method.

- The trace ends of certain discontinuity divide two kinds. One is named as "cutting end" which is located in the exterior of the sampling window, and the other kind is named as "nature end" which is located in the inner of the sampling window. In order to confirm the size and the centre coordinates of a measured discontinuity, cutting end must be restituted to "nature end".
- The radius of certain discontinuity may be confirmed with Monte-Carlo random sampling method, and its centre coordinates (x, y, z) can be gotten.
- The stochastic discontinuity can gain with the method of the conventional modeling method. Through coupling between certain discontinuity and stochastic discontinuity one team by one team, we can gain the coupling results.

The correctness of the method of the coupling modeling is confirmed and it can play the advantages of the discontinuity's network simulate.

Keywords: Coupling modeling; stochastic discontinuity; certain discontinuity; discontinuity network.

ENGINEERING GEOLOGY CHARACTERISTIC AND THE LOW-LOOSE METHOD OF CAVING MINING SYSTEM IN XIADIAN GOLD MINE

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Sublevel caving is a mass mining method based upon gravity flow of blasted ore and caved waste rock. Its major advantage is safety, since all mining activities are conducted from relatively stable openings. Sublevel caving is the lowest cost and most productive underground mining method, providing that all aspects are working well. One of the key aspects of a successful sublevel cave is the control of dilution entry during the drawing of broken ore. Analysis of gravity flow characteristics of broken ore in each type of rock is essential. The most popular stochastic theories are still based on closed-form solutions that apply, strictly speaking, only for statistically uniform flows; and caving mining systems have been widely used in underground mining, where the design and production management are based on the theory of ore drawing.

This paper presents an efficient, convenient approach for modeling complex stochastic flows in moderately heterogeneous loose ore-drawbody. The first stage in the mining process is the characterization of the rock mass or ore-bodies using the in situ and laboratory data. The effects of rock and ore complexity on the rock mass behavior are discussed with reference to laboratory and field observations. The second stage involves the use of a new mining techniques or mode to investigate rock response in the near-field rock mass surrounding the mining excavations. Specifically, we reformulate the governing stochastic equations and introduce a new transfer function to characterize the propagation of caving mining system. A new sublevel caving parameters is optimized using the ore-drawing theory in Xiadian Gold mine. The primary purpose of this research is to investigate the above-stated shortcomings and find ways to overcome difficulties related mainly to mining method in Xiadian Gold Mine. The ore behavior during caving and dilution with surrounding rock was investigated in situ also.

This study shows that the new mathematical model of gravity flow of loose material is proposed in the study on the basis of the improved stochastic theory which can be used to investigate movement field of blasted ore and waste rock in caving mining systems. It has been proved by practice in Xiadian Gold mine, China, that the new simulation process can satisfy the requirement of optimizing the structure parameters of caving systems. By this way the sublevel caving stope is safety. These results suggest that model of low-loose in caving mining system can be applied to model the mining behavior of complicated discontinuous rock masses.

Keywords: Ore-drawing control; parameters of stopes; the dilution and ore loss; caving mining systems; the theory of ore drawing; mathematical model.

CUTTING JOINT BLOCKS AND FINDING KEY BLOCKS FOR GENERAL FREE SURFACES

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This paper describes a process of cutting blocks from statistically generated finite joint polygons in 3-d space. If the ratio of joint length divided by joint spacing is less than 10, the rock mass is likely to be connected. If this joint length ratio is greater than 10, the rock is likely to be blocky. The paper also presents an algorithm for finding all removable blocks along any given moving direction. The rock mass boundary can be any excavated and natural free surfaces. The algorithm works for both joint sets and for any joint system where each joint has its own direction. This is an application of polygon cutting code DC of 3-d discontinuous deformation analysis (3-d DDA).

Keywords: Rock block; discontinuous deformation analysis; excavation.

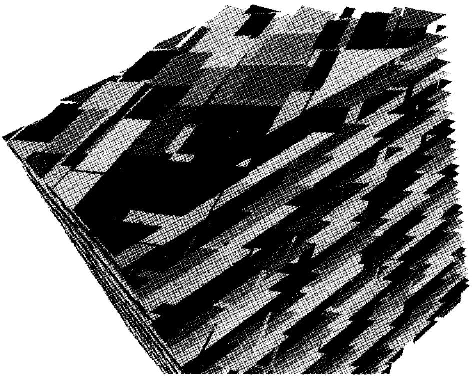

Fig. 1. Statistically produced joint polygons of a joint set.

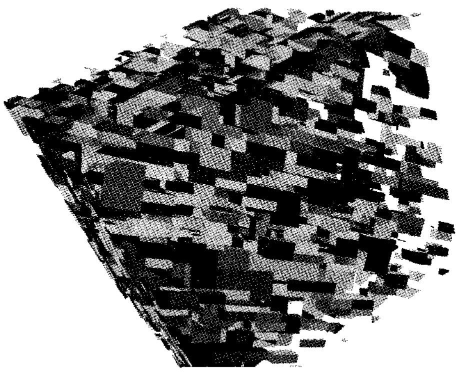

Fig. 2. One fifth of produced block.

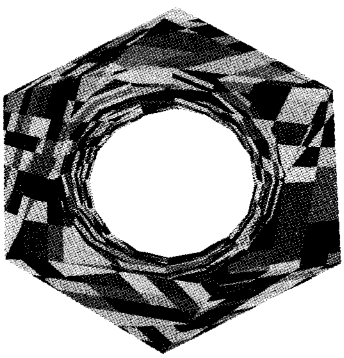

Fig. 3. Produced blocks after excavation.

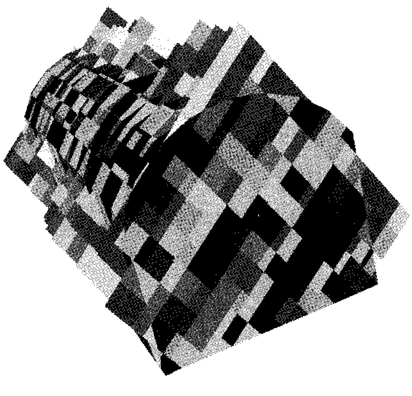

Fig. 4. Maximum removable block groups of tunnel.

AN EXPERIMENTAL-COMPUTATIONAL APPROACH TO THE INVESTIGATION OF DAMAGE EVOLUTION OF EDZ IN ANISOTROPIC ROCK MASS

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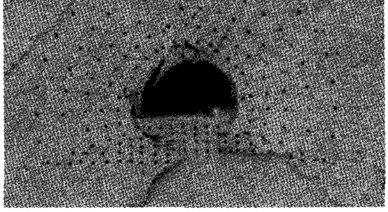
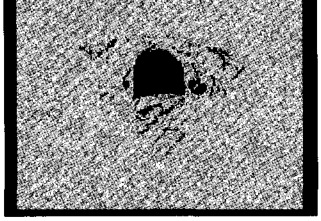
A combined experimental-computational approach to study the evolution of microscope damage to cause failure in anisotropic rock tunnel. During the construction of underground excavations, anisotropic failure has been detected in rock EDZ (Excavation Damaged or Disturbed Zone). As part of the investigation, Scaled model tests considering the heterogeneous and anisotropic material property was performed to investigate the damage and failure process of a tunnel, three loading modes are applied in the scale model, and some experimental results were obtained. Numerical analyses were performed using the computer program Realistic Failure Process Analysis (RFPA) from a mechanics point of view. The observed failure zone around the excavation was simulated using both the maximum tensile strain criterion and Mohr-Coulomb criterion respectively (as the damage threshold). The simulated results reproduce the damage process. On the basis of the simulated results, it is found that this experimental-computational approach is able to simulate the complete mechanical responses of EDZ under static loading, which include the crack patterns associated with different loading stages and loading conditions, localization of deformation, stress redistribution and failure process. Computational model predictions that include crack propagation and failure modes of rock show a good agreement with those of the scaled model tests. It is pointed out that the damage evolution of EDZ strongly depends on the inhomogeneity, the excavation mode, anisotropic property, and the various loading conditions.

Keywords: Scaled model test, Numerical modeling, Microstructure, Anisotropy; excavation.



(a) Photograph of the scale model result



(b) Displacement vector plots of model result (k=1)

Fig. 1. The biaxial testing machine ($\beta = 45^o$).

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10. FAILURE, FRACTURE AND BURST

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NUMERICAL STUDY OF FRACTURE CONTROL TECHNIQUE FOR SMOOTH BLASTING

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Fracture control technique has many practical applications in rock blasting engineering, especially in underground excavation where smoother and undamaged walls are of concern. There are mainly two kinds of fracture control techniques, namely 1) notching along blasthole or drilling satellite holes beside the blasthole; 2) using pre-split charge holder. For the former one, the stress concentration in the tip of the notch will lead to the extension of the notch and the fracture can be controlled. For the latter one, the fracture initiates and propagates along the direction of the pre-split of the charge holder and the rock in other directions is protected by the charge holder. In the present study, the Johnson-Holmquist (JH) material model, which includes a strength model, a polynomial equation of state and a damage model, is implemented into LSDYNA to analyze the effectiveness of the fracture control techniques. Parametric studies are carried out. It is found that these two fracture control techniques are effective in controlling the initiation and propagation of fractures only when the blasthole load meets certain conditions. Stem can improve the efficiency of the notch. For 2-slits charge holder, fracture initiates and propagates exactly at the position of the slits. The 2-slits charge holder may be effectively adopted in pre-splitting operation. For smooth blasting, multiple slit charge holder may be used to produce a smooth plane with fragmentation on one side of the plane.

Keywords: Numerical simulation; Fracture control technique; Blasting; Notch; Pre-split charge holder.

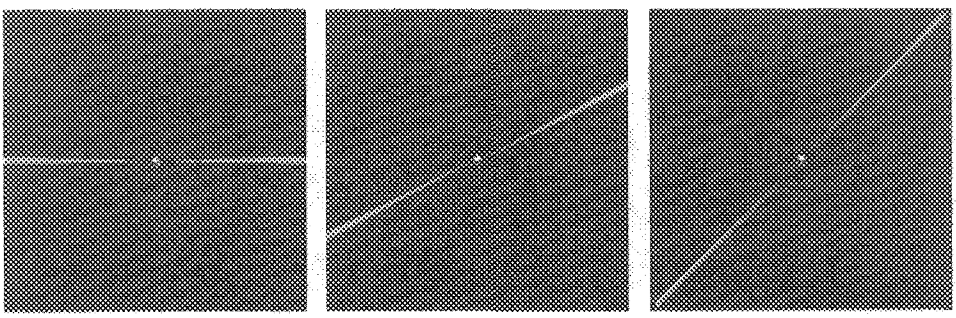

Fig. 1. Fracture pattern with 2-slit charge holder.

NUMERICAL ANALYSIS OF ROCK FRACTURING PROCESS BY DEM USING BONDED PARTICLES MODEL

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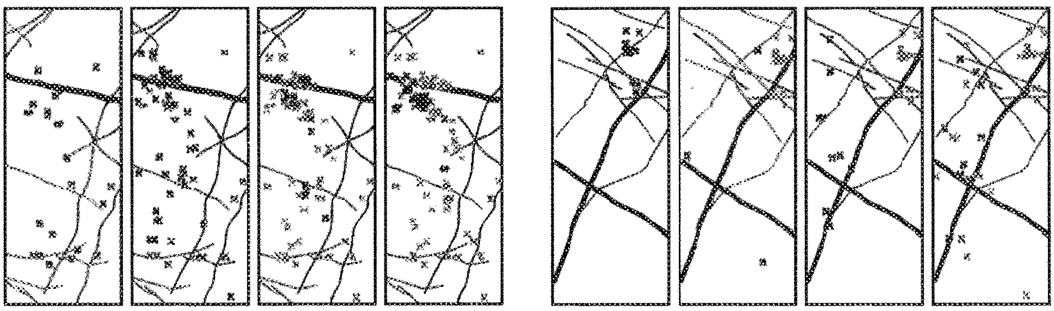
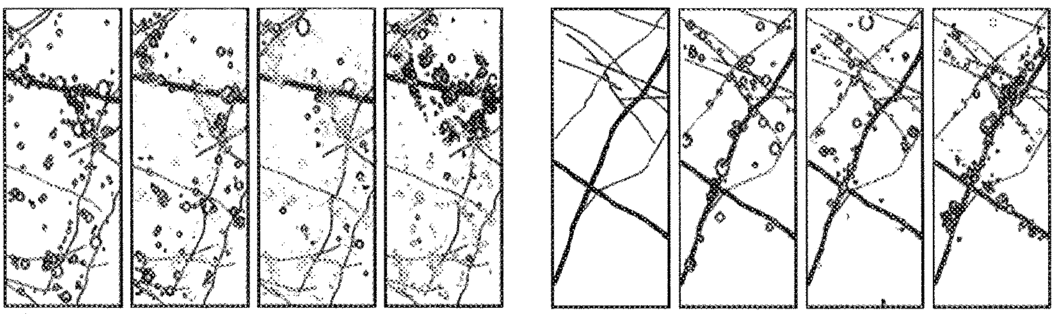
In order to examine the relationship between AE behavior and rock failure process, the fracturing processes of the rock specimens in the laboratory uniaxial compressive tests are simulated by Distinct Element Method (DEM) using the Bonded Particles Model (BPM), and the relationship between the AE sources in the experiment and the locations of the initiated cracks in the DEM simulation is examined. Furthermore, the rock fracturing process around a highly stressed rock cavern is simulated, and the simulation result is examined using the field measurement data. From the DEM simulation of the laboratory tests, it is confirmed that the crack initiation and propagation process computed by DEM shows good agreement with the actual rock fracturing process and the spatiotemporal distribution of AE sources. From the DEM simulation of a highly stressed rock cavern, it is found that the crack initiation and propagation process computed by DEM shows good agreement with the field measurement data around the large underground cavern. Thus, it is confirmed that the DEM simulation is an efficient tool to predict the heterogeneous fracturing process not only for laboratory rock specimen but also for in-situ rock mass.



ε=0% ε=0.34% ε=0.38% ε=0.39% ε=0.41% ε=0% ε=0.20% ε=0.51% ε=0.56% =0.59%

Fig. 1. Spatial distribution of AE sources (black x) of AE sources at the specific phase of strain (I, II, III, & IV). Gray x shows previously initiated AE source location. (left: specimen 1, right: specimen 2).



ε=0% ε=0.34% ε=0.38% ε=0.39% ε=0.41% ε=0% ε=0.20% ε=0.51% ε=0.56% =0.59%

Fig. 2. Spatial distribution of crack (black circle) at the specific phase of strain (I, II, III, & IV). Gray circle shows previously initiated crack. Size of circle shows magnitude of released strain energy. (left: specimen 1, right: specimen 2).

DEVELOPMENT OF FLUID FLOW ANALYSIS PROGRAM IN 3-D DISCRETE FRACTURE NETWORK INCLUDING CONSIDERATION OF ITS INPUT PARAMETERS AND HYDRAULIC BEHAVIOUR

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Fractures in a rock mass play a dominant role in the mechanical and hydraulic properties of rock mass. The dominant factors that influence on the fluid flow are hydraulic and geometric characteristics of fractures and connectivity among them. So the discrete fracture network (DFN) approach has widely been used in order to investigate the hydraulic behaviour of fractured rock mass. Generally, fracture networks are generated by assuming that fractures are planar, circular discs oriented by Fisher distribution function and their centers are randomly distributed. The fluid flow in a fracture is assumed as a parallel plate fracture model. On the assumptions mentioned previously, we have developed the program that can generate the 3-D discrete fracture network and analyze the connectivity of the generated fractures and then estimate the equivalent hydraulic conductivity.

In general, DFN model is used in order to analyze the hydraulic conductivity in relatively small area in comparison to the continuum model that can analyze it in wide area. It is because DFN approach has a limit that analysis cannot be performed on over a certain number of fractures. A tool to overcome the shortcoming of DFN model is elimination of fractures having a small aperture or a short length. By applying the elimination method, the elimination of fractures has little influence on the fluid flow. Figure 1 shows that the fractures with aperture smaller than 30% of mean aperture have little influence on a analysis.

Keywords: Discrete fracture network, fluid flow, equivalent hydraulic conductivity.

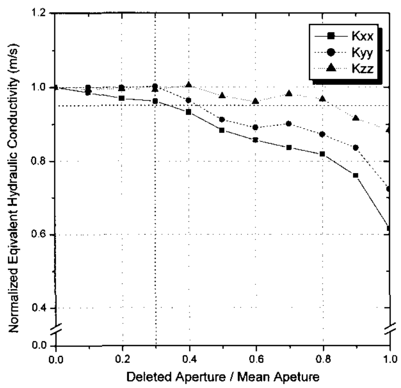

Fig. 1. Influence of eliminating fractures with small aperture on the normalized equivalent hydraulic conductivity tensors, K_{xx}, K_{yy} and K_{zz}.

THE CONDUCTIVITY VARIATIONS OF SINGLE ROCK FRACTURE DURING NORMAL LOADING

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Coupling behavior of rock joints is of prime importance in the field of environmental safety evaluation, especially for high polluted waste depository in rock masses. According to results acquired from relevant researches over the past three decades, the aperture distribution, corresponding to specific stress level during normal deformation, is not only the key factor controlling conductivity variations of single joint, but also the most difficult part to obtain, either from the model analyses or from the experimental simulation.

In this research, the hydro-mechanical coupling behaviors under normal loading are investigated by both theoretical model analysis and experimental studies. In the model analysis, a theoretical coupling model of rock joint is developed on the basis of the micro-mechanical concept. Hence, the aperture distribution for each loading level corresponding to normal closure behavior is obtained from the proposed model. In addition, the conductivity variations of a joint and its flow field under closure process could be calculated with FEM seepage program based on the obtained aperture distribution. Especially, a series of measurements on aperture distribution of artificial model joints under specific normal loading is carried out with a water drop method proposed by Hakami. Meanwhile, the flow tests of the model joints are also conducted to verify the applicability of the proposed coupling model.

According to results from this investigation shows that the rougher the joint surface is, the larger the initial aperture of tensile joint will be. Also, variations of conductivity of the rougher joint could be decreased about 2 orders as normal stress up to $0.25\sigma_c$ of model material. On the other hand, the influence of roughness on the conductivity is analyzed by two parameters, tortuosity of flow path and contact areas of asperities, respectively. Finally, deviations between cubical flow rule and that estimated flow rate induced by roughness are investigated in details by the above mentioned two approaches and it shows that the former has better effect than the latter one.

Keywords: Conductivity; rock fracture; load.

INFLUENCE OF CELESTIAL BODY ACTIVITY ON THE ROCK BURST OCCURRENCE IN COAL MINE

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The factors influencing rock burst occurrence are mainly focused on two aspects, geological conditions and mining conditions. Influence of celestial body activity on the rock burst occurrence is mainly researched by theoretical analysis, and explained with the statistical data of rock burst events in Chinese coal mines.

The stress difference induced by the variety of earth rotation velocity in the thin earth's crust is calculated as Eq. (1).

$$\Delta = \Delta_1 \rho \alpha^2 D(\alpha) \gamma \tag{1}$$

The results are showed in Table 1.

Table 1. Stress-difference among the different latitudes.

Latitude, °	0	15	30	45	60	75	90
Stress Value, MPa	74	69	55	37	41.5	49	51.4
Stress-difference, MPa		5	14	18	4.5	7.5	2.4

The biggest stress-difference (18MPa) lies between the latitude 30° and latitude 45°, in which is the most dangerous region of the rock burst occurrence and earth quake. The geographical distribution of Chinese rock burst mines has confirmed this.

Compared the rock burst seasonal occurrence with the nonuniform velocity of earth rotation, the maximum number months of rock burst occurrence are relative to the minimum velocity months (from Feb. to Mar., November) and the maximum velocity months (from Jun. to Sep.) of earth rotation. So, the months in which rock burst is easily occurred in a year are from Feb. to Mar., from Jun. to Aug., and from Nov. to Dec., which is proved by the rock burst occurrence number of the Mentougou mine of China.

The solar activity changes from day to day, and circulates once at average intervals of about 11 years. Some geophysics processes are induced by the solar ultraviolet radiation, X radial and particle radial. The alternating electromagnetic field acts on the rock in several years, which greatly changes the rock strength and deformation, then the stress in the earth's crust also greatly changes. So the solar activity can induce the rock burst occurrence.

In the first day and the fifteenth day of lunar calendar, the sun, moon and earth lie at the same line, so the gravitation of sun, moon to earth intensifies to maximum value, which can induce the rock burst occurrence.

The activity of sun, earth and moon has obvious influence on rock burst occurred in the coal and rock mass which is in the state of critical instability.

Keywords: Rock burst; Celestial body activity; Earth rotation; Sun-moon tide.

EXPERIMENTAL ASSESSMENT OF HEALING OF FRACTURES IN ROCK SALT

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Cracks or fractures induced around salt caverns by solution mining or fluctuation of operating pressures can become preferential leakage path of the stored brine, gas or compressed-air to nearby rock formations or to adjacent caverns. It has long been recognized that damage in rock salt may be healed under hydrostatic and non-hydrostatic compression. The governing mechanisms and parameters for the healing of salt fractures however have not been truly understood. Even though extensive mathematical models and constitutive equations have been derived to describe the mechanical behavior of the damaged and healed salt, experimental studies on salt fracture healing and its influencing factors have been rare.

The objective of this research is to assess experimentally the healing effectiveness of rock salt fractures as affected by the applied stresses, fracture characteristics, moisture contents and time. The effort involves (1) fracture pressurization tests under uniaxial and radial loading, (2) gas flow permeability tests to monitor the time-dependent behavior of the salt fractures, and (3) point loading and diameter loading tests to assess the mechanical performance of the fractures after healing.

Tension-induced fractures and fractures formed by saw-cut surfaces and by polished surfaces have been prepared in salt specimens. Roughness and mineralogical features of the salt fractures are recorded. Series of gas flow testing have been performed to monitor the changes of the fracture permeability under pseudo-static loading ranging from 0.7 to 20 MPa for up to 120 hours. Healing tests under static loading have been carried out under both dry and saturated conditions. Brazilian tensile strengths and point load strengths of the fractures after healing under variable test parameters are measured and compared with those of the intact salt to assess the healing effectiveness.

The results suggest that the primary factors governing the healing of salt fractures are the origin and purity of the fractures, and the magnitudes and duration of the fracture pressurization. Inclusions or impurity significantly reduce the healing effectiveness. The hydraulic conductivity of the fractures in pure salt can be permanently reduced by more than 4 orders of magnitude under the applied stress of 20 MPa for a relatively short period. For most cases the reduction of salt fracture permeability is due to the fracture closure. This fracture closure does not always lead to fracture healing. Both processes are time-dependent. The closure involves visco-plastic deformation of the asperities on both sides of the salt fractures, while the healing is the covalent bonding between the two surfaces. Fracture roughness and moisture contents apparently have insignificant impact on the healing process.

Keywords: Healing; rock salt; fracture; permeability; creep; closure.

ELASTIC-PLASTIC FRACTURE DAMAGE ANALYSES ON THE ROCK COVER OF NINGBO XIANGSHAN HARBOR SUBSEA TUNNEL

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The paper applies the 3-D finite element program of fracture damage model to the project of Ningbo Xiangshan Harbour Subsea Tunnel to model the ground movements that occur when the subsea tunnel is installed in discontinuous jointed rockmass. According to the geological section of Ningbo Xiangshan Harbour Subsea Tunnel, the tunnel axes of nine sections selected are researched and analysed. The appropriate tunnel axis of each section is fixed on through lifting and shifting the tunnel axis according to the thickness of bedrock. The paper applies the finite element procedure to analyze different cases of the sections. Analyzing their damage zones, plastic zones, stress state and displacement variation in all cases, the stability of the tunnel is studied. Based on the above analysis, the most appropriate tunnel axis position of every section is proposed. The rock cover of the subsea tunnel showed in Table 1 is obtained.

Keywords: Fracture; rock cover; tunnel.

Table 1. Propositional soleplate thickness according to numerical simulation.

Location of section	Soleplate attitude of design	Soleplate attitude of computation	Thickness of designing above rock	Thickness of computing above rock
25000	-99	-99		
26100	-143.2	-151.2	17.95	25.95
27100	-150	-156	25.75	29.75
27600	-149.95	-153.95	18.57	22.57
27900	-150	-152	24.95	26.95
29253	-140	-142	27.45	29.45
29644	-138	-138	28.95	28.95
30100	-128.4	-130.4	29.1	31.1
30760	-111.4	-113.4	24.1	26.1
31340	-95	-99	11.1	15.1

A COUPLED APPROACH FOR GAS OUTBURST SIMULATION

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Outbursts occur as a result of mutual interaction of a number of factors: rock pressure, gas presented in coal, and physical and mechanical properties of coal, etc. Although the importance of influential factors and their interaction has been treated in various ways, the following description of outburst mechanism is generally accepted: changes in stress and reservoir pressure during mining cause deformation and failure of the coal and the gas/coal interaction in the seam manifest itself to a dynamic type of failure (outbursting). More specifically, there are two conditions which must be met for an outburst to occur: the coal must be failed under an effective stress and gas must be able to desorb rapidly from the coal and the gas pressure must be large enough to push the failed coal into the mining opening instantaneously. In other words, the occurrence of an outburst is the result of combined effects of stress redistribution, gas desorption, coal property (e.g., coal strength) and time.

Attempts have been made to find the analytical solution for initiation of an outburst. The approaches have concentrated on the deviation of solutions for the evolution of the fluid pressure for some mathematically tractable problems, and based on the assumption that outbursts are likely to be related to the fluid pressure gradient and certain mechanical properties, such as tensile strength of the coal. Although the analytical approach can provide some very useful insight into the significance of some important parameters, its application is limited to problems with simple geometry and boundary conditions.

Although a great effort have been made to develop a generalised numerical model to simulate the process of outburst initiation, it has proven to be difficult to do so because the process covers two separate entities of geomechanical stress analysis and reservoir simulation. To date, except for a few specialized simplified approaches, there is no single computer code that handles such a coupling. An alternative method commonly employed is to hybrid two well developed commercial codes in which one is for geomechanical stress analysis and the other for reservoir simulation.

This paper describes a three-dimensional coupled approach for gas outburst simulation aiming at determining threshold gas concentration for outburst control in coal mining. The coupled simulator is developed by linking two well developed commercial codes FLAC3D, a geomechanical stress analysis code and COMET3, a reservoir simulator. The built-in language FISH in FLAC3D is used for data processing and transferring between the two commercial codes. The processes in terms of their concepts and models, as well as their integration are described in the paper.

The simulator is applied to investigate the effect of coal strength on critical gas concentration. The modelling result indicates that coal strength plays a very important role in a potential outburst occurrence. The greater the coal strength the bigger the critical gas concentration. The result can be used to help predict threshold value of gas concentration for gas outburst occurrence.

Keywords: Gas outburst; simulation; analytical solution.

NUMERICAL STUDY OF THE SHEARING OF LARGE FRACTURES HAVING PROPAGATING BOUNDARIES

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An important condition that Swedish Nuclear Fuel and Waste Management Co (SKB) imposes on the safety of a repository is the adoption of a size of 10 cm as the ultimate deformation the system "canister hole – canister" may be exposed to. Having said that, the shear displacement of distinct fractures, hitting or running through a rock horizon, where a repository would be placed, turns out to play an important role in the safety analysis. Site investigations of candidate sites have revealed competent rock masses at depths of around 500 m, where the existence of outstanding fractures justifies their treatment as independent bodies.

In a series of analyses performed, using the distinct element code 3DEC, single fractures were modelled in such a way that propagation past the outline of a fracture – through a number of simplifying assumptions and numerical manipulations – was made possible. Fractures studied, being circular in shape, had a diameter of 50, 100 and 200 m. To prepare a basis for comparison, in a next series of analyses, no enlargement of a target fracture was allowed past its outline. This condition in 3D resembles the more common case in 2 D, where a target fracture is said to have "fixed" ends. All other conditions were identical to those used in analyses pertaining to fractures with propagating boundaries.

Inferred in-situ stress fields were borrowed from investigations SKB is currently undertaking. As an example, the major principal stress at a depth of 500 m – a candidate depth for the placement of the repository and where the centre of a target fracture in the numerical simulations was assumed to lie – was equal to 45 MPa. Rock types entered into the numerical analyses were tentatively designated as competent and less competent with deformation modulus equal to 70 and 40 GPa, respectively. These values are typical values the recent site descriptions by SKB convey.

As expected, the numerical analyses showed that fractures with propagating ends shear in larger displacements compared with fractures with fixed ends. The discrepancy in the maximum shear displacements for comparable cases could be significant.

While learning much from the investigations of rock propagation at very small scale – where laboratory experiments supported by adequately sound theoretical studies form a main frame – up scaling brings about new parametrical unknowns, the evaluation of which would just be hypothetical. Appropriately designed numerical models fed with plausible input parameters help, however, come closer to how natural fractures exposed to shearing forces may behave.

Keywords: Rock fractures; Shear displacement; 3DEC; Nuclear waste.

STUDY OF OCCURRENCE CONDITIONS AND CRITERIA OF ROCK BURST IN COAL MINE

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The rock burst is an "explosion-like" fracture which usually occurs at the edge of a seam or in a pillar, its occurrence is always influenced by the tectonics stress in many minefields. In this study, the simulation of rock burst under the action of tectonics stress is implemented by a new numeric calculation method, the RFPA tool.

Through calculating, recording and analyzing the typical models, the elastic module map of the roadway transformation is shown by the results of the calculation. At the same time, the stress variation and destructive process in rock and coal mass, and the rock burst features during the process of numerical simulation are acquired.

The simulation of the fracture process is considered as the actual rock burst occurrence, it is because:

(1) The criteria of deep fracture: the fracture of rock burst occurs in the deep floor of the roadway, and then expanding to the free surface of the roadway abruptly.

(2) The criteria of abrupt fracture: most of the fracture appeared on the same step of the calculation at the same time, the rock burst occurrence is an abruptly fracture in a large scope of the coal and rock mass.

(3) The criteria of the energy release rate: the released energy in the current step is more than 60% of the total released energy until the current step.

Different critical horizontal stress is put forward under the condition of different lateral loading coefficient through researching the influence of different lateral loading coefficient on the conditions of rock burst. It is the objective of this paper to describe the occurrence conditions and the criteria of the hard and soft rock layers lying in roadway roof or floor, this is one of necessary condition of occurring the rock burst in roof or floor through researching the influence of different combination of coal seam and rock layer on the conditions of rock burst.

The compatible criteria of rock burst occurrence are firstly put forward:

Criterion 1: The angle between the direction of the roadway and the maximum horizontal principal stress is more than 60°. This is a necessary condition of the rock burst occurrence.

Criterion 2: The maximum horizontal principal stress exceeds the critical horizontal principal stress. This is sufficient condition.

Criterion 3: The hard and soft rock layers lies in the roadway roof and floor. On the actual conditions, there is top coal in the roof or hard and soft coal and rock layers in the floor, their thickness combination is in a scope. This is another necessary condition.

The criteria of rock burst occurrence are successfully applied in the danger prediction of rock burst on coal mine.

Keywords: Tectonics stress; Rock burst; Critical horizontal stress; Lateral loading coefficient.

ANALYZING SCALE AND PRESSURE DEPENDENT PROPERTIES OF FRACTURE USING CT SCANNER

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Fracture serves as a main flow path for hydrocarbons in tight fractured reservoirs. Therefore, it is crucial to characterize fracture to understand physical phenomena concerning fluid flow through fracture. The main factor affecting mechanical and hydrologic characteristics of fracture is aperture distribution rather than the roughness of one fracture surface.

This study utilized an X-Ray CT Scanner to determine fracture aperture distribution, and to investigate the effect of sample size and confining pressure on fracture aperture distribution. The aperture distribution of artificially generated fracture of a Spraberry sandstone was measured under various confining pressures (1.4, 2.4, 4.1, 5.5, 6.9, 10.3, 13.8, and 17.8 MPa). The fracture aperture follows log normal distribution. As confining pressure increases, the modal value of the fracture aperture decreases, but the fracture aperture still follows log normal distribution. The effect of sampling size and confining pressure on aperture distribution was examined using fractals. In Figure 1, although fractal dimension D and amplitude parameter A do not reach the stationary threshold due to the small size of the specimen, both parameters fluctuate within a band, and within a certain size of sampling area (between 1500 and 2400 mm^2). Confining pressure affects both parameters, so both parameters differ under different confining pressures for a reference sampling size. In addition, mechanical properties of fracture-such as fracture displacement and normal stiffness as a function of normal stress-were also derived using an X-Ray CT Scanner.

Fracture aperture distribution is a function of sample size and confining pressure. Therefore, sample size should be larger than the stationary threshold. The stationary threshold which determines the minimum sample size should be decided according to the appropriate confining pressure to characterize fracture properly on a laboratory scale or field scale. An X-Ray CT Scanner was proved to be able to measure mechanical properties of fracture as well as aperture distribution. The results of this study will lead to more thorough study on the hydrologic and mechanical properties of rock fracture.

Keywords: Fractal dimension; amplitude parameter; X-Ray CT Scanner.

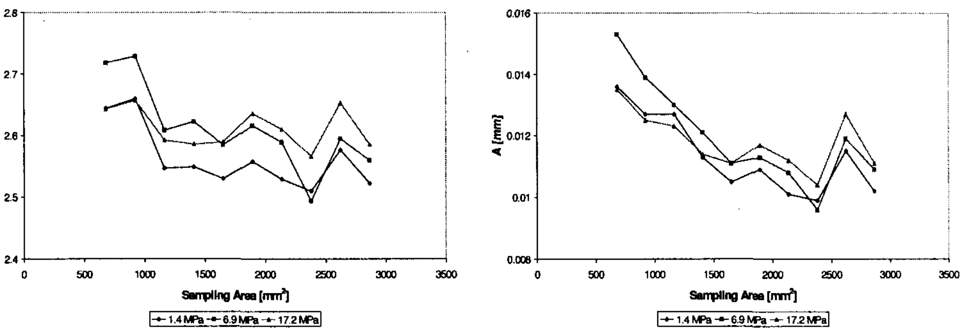

Fig. 1. Change of fractal dimension D and amplitude parameter A as a function of sampling area and confining pressure.

EFFECTS OF SHEARING PROCESSES ON FLUID FLOW AND PARTICLE TRANSPORT IN A SINGLE ROCK FRACTURE

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The effects of different shearing processes (translational and rotary shearing processes) on fluid flow and particle transport in a single rough rock fracture were numerically investigated in this study. A pair of digitized surfaces of a 250×250 mm rough concrete fracture replica was numerically manipulated to simulate both translational and rotary shearing processes of the sample without considering normal loading and asperity damage, with a 1 mm shear displacement interval (up to 20 mm) and with a 0.5° shear angle interval (up to 10°) for translational and rotary shear, respectively. The evolutions of the aperture distributions during the shearing processes were determined in each shearing step and the evolutions of the fracture transmissivity fields were calculated using the cubic law. The radial fluid flow situation, which is commonly used in the laboratory for coupled shear-flow tests of single fractures, was considered. The particle transport was simulated using the Particle Tracking Method, with the particles motion following the fluid velocity fields during shear, which were calculated using the Finite Element Method (FEM).

The results of flow and particle transport simulations show that shearing processes make rough fractures much more permeable, producing a significant decrease in travel time of the particles, especially at the start of the shear processes. Translational shear causes more anisotropic and heterogeneous flow with significant high transmissivity channels in the direction perpendicular to the shear direction through which particles traveling in this direction can travel faster and without being delayed by bypassing low transmissivity areas. On the other hand, during the rotary shear, radial flow becomes rapidly isotropic and particle transport is more homogeneous. The travel time is greatly reduced at the start of rotary shear due to an almost uniform dilation over the whole fracture sample surface. Figure 1 shows the comparison of transmissivity evolution and the change of particle paths for radial fluid flow between translational and rotary shear. The flow and particle transport simulations in this study provide a first step towards a better understanding of particle transport in coupled stress-flow processes under rotary shear. The findings from this study also may have an important impact on the interpretation of the results of coupled hydro-mechanical and tracer experiments for measurements of hydraulic properties of rock fractures.

Keywords: Rock fracture; coupled shear-flow test; particle transport; finite element method (FEM); particle tracking method.

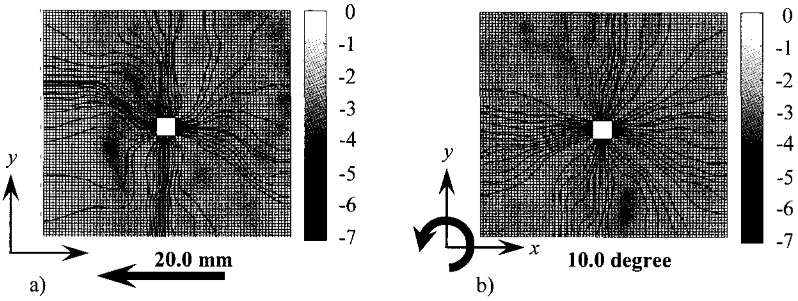

Fig. 1. Transmissivity evolution and the change of particle paths for radial fluid flow during a) translational and b) rotary shear.

STUDY ON INTERACTIVE MECHANISMS OF TWO CRACKS UNDER COMPRESSIVE CONDITIONS

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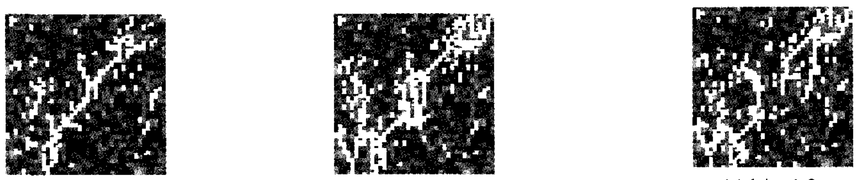
Interactive mechanisms of two cracks which are responsible for the stability of many underground large caverns are studied by cellular automata. In recent years cellular automata due to their locality and parallelization are considered as an efficient method to simulate complex system and study mechanisms of physical phenomena. Cellular automata are composed of lattice, states, neighbors and local rules. Thus rock samples can be discretized into cell elements that are connected with beam elements. Based on equilibrium equations, constitutive equations and deformation equations local rules are attained, which overcomes the shortcoming of arbitrary decision of local rules. Maximum tensile strain criterion and Mohr-Coulomb criterion are introduced to judge the beginning of tensile failure and shearing failure respectively. And based on damage mechanics and experimental results deformation and strength properties and damage evolution law of damaged cell elements are estimated. So propagation and coalescence of cracks can be simulated based on this model. Interactive mechanisms of two cracks under compressive conditions are very complex and are influenced by relative geometric position such as rock bridge angle and dip angle of cracks, friction and closure of surface of cracks and distance between inner ends of two cracks and so on. This paper mainly studied the influences of crack geometric position and distance between inner ends of two cracks on the interactive mechanisms. The simulated results are in good accordance with the existing experimental results. In the meantime the results discover more complex mechanisms which needed to be verified by more experiments. The presented cellular automata method in this paper proves to be a promising method to simulate rock fracture.

Keywords: Interaction; crack growth; coalescence; cellular automata.



(a) h/g=0.4 (b) h/g=0.8 (c) h/g=1.2

Fig. 6. Coalescence patterns of samples with bridge angle of 63 degree and different h/g.

GEOLOGICAL SETTING OF THE ROCKBURST OF QINLING TUNNELS IN CENTRAL CHINA

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Two extra-long tunnels have been excavated in a corridor of about 200 m in the central part of mainland China. One tunnel, with two tubes, is in the railway line from Xi'an to Ankang, and the other tunnel, with one tube having been excavated, is in the road line. The tubes are nearly parallel to each other and more than 18 km long. The maximum overburden of the tunnels is over 1600 m. During excavation, severe rockbursts have occurred in several sections.

The rockburst mainly occurred in migmatic gneiss sections with an overburden of more than 700 m. The dry and large strength wallrocks, with large initial geostatic stress kept in, are readily subject to rockburst. Severe rockburst occured in sections, where the initial horizontal stress is about 20 MPa - 30 MPa and the overburden is no less than 1000 m. The rockbursts occurred at the Qinling tunnels are of self-initiated type (Kaiser & Maloney 1997).

The main conditions of the rockbursts of Qinling tunnels are the relatively intact hard rocks, large initial geostatic stress and the beneficial effect of the foliation in the gneiss to wall rock failure along excavation surface, especially the failure of spalling. The regional geological setting of the wall rocks contributes not only to the occurrence of the features of the rock mass, such as foliation in the gneiss, magnitude of initial geostatic stress kept in intact rock mass, but also to the frame of the relatively intact unit of rock mass being surrounded by extensively deformed belts. As a result, the large initial geostatic stress kept in sound hard wall rocks of Qinling tunnels are the remains of the earlier geological stress, and the geological stress are significantly relaxed if the rocks are intensively superimposed by ductile to brittle deformations. The lower initial geostatic stress in the extensively deformed belts is due to the extension geological setting during the uplifting of the Qinling orogen.

Keywords: Geological setting; rockburst; occurrence; tunnel; China.

LOCALIZATION OF WATER FLOW IN A SHEARED FRACTURE AS ESTIMATED BY LARGE FRACTAL FRACTURES

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Large synthetic fractures of 0.2 m to 12.8 m were created on a computer by a new spectral method so that the ratio of the power spectral density (PSD) of the initial aperture (the aperture when the surfaces are in contact at a single point) to that of the surface height, determined for a tensile fracture of 1 m, may be approximately reproduced. With the application of shear displacements (δ) of 12.5 mm, 25 mm and 50 mm to these fractures, the initial aperture was determined for a square area with a side length of from 0.1 m to 6.4 m. The standard deviation (SD) of the initial aperture remarkably increases with shear displacement, which suggests that shear displacement may greatly affect the permeability of the sheared fractures. By taking aperture data at constant intervals to establish a flow area, we simulated macroscopic water flow both parallel and perpendicular to the shear displacement for fractures during closure by solving Reynolds equation, and determined the hydraulic aperture and the distribution of local flow rate to investigate the hydraulic heterogeneity in the water flow.

The results showed that the coefficient of variation in local flow rate significantly increases with both shear displacement and fracture size. Thus, the hydraulic heterogeneity in a sheared fracture increases with both shear displacement and fracture size. Furthermore, as shear displacement increases, a fewer channels with a greater area and at the same time ridges of nominal contact areas with a greater area are formed perpendicular to the shear displacement. Thus, water flow in a sheared fracture is more localized with the shear displacement.

A theoretical relation between the mean aperture normalized by the SD of the initial aperture and the percentage of the nominal contact area to the nominal fracture area showed that the percentage of the nominal contact area is governed by the mean aperture normalized by the SD of the initial aperture and that the nominal contact area rapidly increases with a decrease in the normalized mean aperture. Accordingly, when a sheared fracture is closed to have the same mean aperture, the nominal contact area rapidly increases with the shear displacement since shear displacement significantly increases the SD of the initial aperture. The PSD of the initial aperture in the directions of macroscopic flow both perpendicular and parallel to the shear displacement showed that shear displacement remarkably increases the amplitudes of the components of the initial aperture with wavelengths of greater than twice the shear displacement, while those with wavelengths of less than twice the shear displacement rapidly decrease with spatial frequency. This means that the amplitudes of channels with wavelengths of greater than 2δ increase with the shear displacement and that the wavelengths of channels and ridges increase with the shear displacement. Accordingly, as shear displacement increases, the nominal contact areas are clustered within a greater distance and at the same time a fewer channels with greater amplitudes are developed, which result in localization of water flow in a sheared fracture.

Keywords: Sheared fracture; Localization of water flow; Large synthetic fracture; Hydraulic heterogeneity.

ROCK BURST CHARACTERIZATION FOR UNDERGROUND CONSTRUCTIONS

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During the construction of underground openings, understanding the probability of the rock burst is important to save the life of laborers and the project. The relation between tangential stress (σ_θ) around underground opening and the rock material strength is valid to reveal the importance of the risk. σ_θ can be calculated by some procedure published into the literature, and rock material strength can be determined from different laboratory or in situ experiments. In this study, rock material strength is determined by the textural analysis of rock material. The quantification of rock texture is made by texture coefficient (TC) and the rock material strength is derived by a fuzzy model between TC and point load strength index (I_s).

The relation between σ_θ and I_s is formed by using fuzzy modelling technique and a fuzzy model is constructed in order to get the risk of rock burst for different conditions. This fuzzy model has 2 input parameters that are σ_θ and $I_{s[TC]}$, and 1 output parameter that is rock burst risk. This fuzzy model uses I_s as an input parameter for rock material strength, and also it is possible to use the estimated I_s by aforementioned fuzzy model between I_s and TC. In order to understand the reliability of the fuzzy model for rock burst characterization, 46 different scenarios have been produced for different σ_θ and I_s values. The fuzzy model results are compared with the results of Russenes (1974) model, and this comparison that is given in Figure 1 shows the importance and reliability of fuzzy model. Consequently, it can be easily concluded that TC and σ_θ is enough to evaluate the possible rock burst activity for underground openings as an output of this research.

Keywords: Fuzzy Modelling; Rock Burst; Tangential Stress; Texture Coefficient.

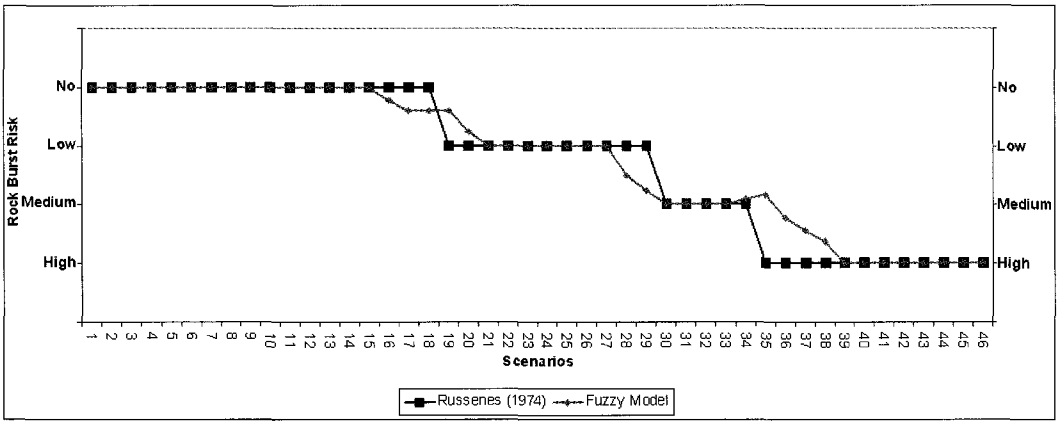

Fig. 1. Comparison of Russenes (1974) and fuzzy model' s results.

ROCK BURSTS, EXPERIENCE GAINED IN DEEP TUNNELS AND MINES

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The phenomena of rock burst are well known in the deep coal, ore and diamond mines all over the world. Based on years of experience, in the mining industry it has been tried to improve the predictability of these phenomena, to optimize the excavation methods and to minimize the risks or the consequences. Examples of these results which are interesting for tunnelling are presented in this report.

With the construction of deep tunnels having an overburden of 1000 m up to 2500 m the so called hazard scenario "rock burst" appeared more frequently. This phenomena is distinguished between loosening caused by discontinuities or by increases in stress in massif rock and real rock bursts.

The presentation shows technical expertise gained by the construction of long and deep tunnels in Switzerland during the past 20 years. These tunnelling projects are the Furka Tunnel, the Vereina Tunnel, the Lötschberg and the Gotthard Base Tunnel.

For the phenomena occurring in the various geological conditions, different excavation methods and cross sections the rock burst phenomena and the applied measures are described.

At the end examples of the Lötschberg and Gotthard Base Tunnels are described more detailed. Gained experience, measures and interpretation of phenomena are summarised for loosening caused by increases in stress during the excavation with tunnel boring machines (TBM) and for extreme rock bursts during the heading by conventional means.

These are

- The possible impacts on a TBM excavation, on possible clamping of a shielded machine and the theoretically most favourable shape of the cutter head are shown.
- Based on the geotechnical calculation models of the rock burst occurrence for the heading by drill and blast, measures for the reduction of the effects of the rock bursts and for the worker safety are defined. Beside structural measures support elements for the absorption of the dynamic loads of a rock burst are important. At the same time the possibility of forecasting rock bursts against the geological prediction is described.

Keywords: Gotthard Base Tunnel; deep tunnels; rock burst phenomena.

RESEARCH ON THE COUPLING SUPPORT MECHANISM OF SOFT ROCK TUNNEL AT GREAT DEPTH

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With the shallow coal resources increasingly decreasing, the depth of coal mining increases step by step. Under the complicated mechanics environment at great depth, deformation of tunnel within soft rock has come to the stage of non-linear plastic deformation, and the deformation field is non-linear mechanics field. So the failure mechanism of tunnel within soft rock is quite different from that of hard rock.

Failure of tunnel within soft rock is mainly attributed to the uncoupling of mechanics characters between support and surrounding rock, and often firstly occurred on key position. Then the whole support system is failed. According to different deformation mechanism, failure mechanism of tunnel within soft rock can mainly be categorized as following four types: i) strength uncoupling between the support and surrounding rock; ii) positive rigidity uncoupling between the support and surrounding rock. iii) negative rigidity uncoupling between the support and surrounding rock. iv) structural deformation uncoupling between the support and surrounding rock.

From the deformation and failure mechanism of tunnel within soft rock at great depth, the goal of coupling support of tunnel within soft rock at great depth is to realize harmony deformation through coupling support on certain position of surrounding rock which often experience large plastic deformation. Additionally, the deformation limit of surrounding rock can efficiently promote the self supporting ability of surrounding rock and realize the function of synthesized support system and maintain the tunnel stability.

By the numerical simulation analysis, it can be realized the maximum strengthening effect of bolts to surrounding rock when the rigidity coupling between bolt and surrounding rock. we can think that the strengthening range of bolt will surpasses that of traditional opinion.

The result shows that, when both strength and rigidity coupling between bolt-mesh and surrounding rock are realized, the stress concentration parts of surrounding rock will transform and spread towards low stress parts, so strength on the support and surrounding rock deformation will be uniformity.

The key role of anchor support is to utilize the strength of inner rock of tunnel. The anchor support mobilized the strength of inner rock which supplied the support load of surrounding rock so as to decrease deformation amount of surrounding rock.

According to the research results above, the characteristics of coupling support of tunnel within were: i) efficiently utilize surrounding rock's self-support ability in a maximum; ii) Promote support ability of rigid bolt in a maximum; iii) control the surrounding rock by utilizing the strength of inner rock; iv) optimize the combination of surrounding rock and support so as to achieve the best coupling support status.

Keywords: Tunnel within soft rock; great depth; key position; coupling support; numerical simulation.

DAMAGE ASSESSMENT OF EDZ IN ROCK AROUND CIRCULAR OPENING BY ACOUSTIC EMISSION

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In this paper, acoustic emission (AE) measurement and AE source location were conducted on model of granite block with circular opening under uniaxial compression, and numerical analysis using RFPA (Realistic Failure Process Analysis) was applied to investigate failure or damage mechanism of Excavation Disturbed or Damaged Zone (EDZ) in rock around excavated opening. The AE source location was made by measuring the five channels AE data in Seoul National University, Korea of Republic. This paper describes a series of AE experimental tests performed on rock with a circular of excavation shape. The AE measurement and AE source location can detect fracture initiation, critical energy release due to propagation of fracture and damage process. AE studies were used to demonstrate how influences the deformation behavior of rock and damage assessment of the EDZ on anisotropic rock for cavern. Based on the simulations and experimental test, Some following results are obtained: (1)From experiments, the accumulated events of AE and AE source location can detect fracture initiation, critical energy release due to propagation of fracture, and damage process or boundary; (2) The whole progressive fractural or damage process including crack initiation and propagation was also validated by the numerical method using RFPA code; (3) The numerical analysis using RFPA was one of the effective tools to supplement the limitation of experimental work to evaluate the damage of EDZ in rock around underground opening.

Keywords: Acoustic emission; AE source location; Damage mechanism; Numerical modeling; Fracture Initiation and propagation; Stress concentration.

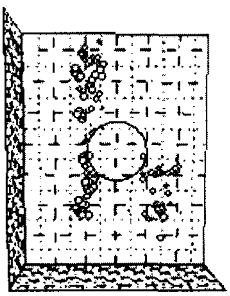

Fig. 1. AE test results.

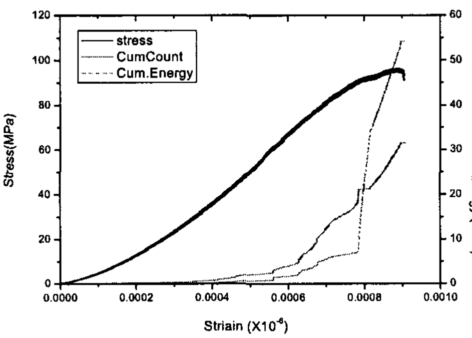

Fig. 2. The stress-strain and AE distribution of the sample.

VISUALIZATION AND QUANTITATIVE EVALUATION OF APERTURE DISTRIBUTION, FLUID FLOW AND TRACER TRANSPORT IN A VARIABLE APERTURE FRACTURE

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The heterogeneous void geometries (aperture distribution) in a single fracture have a strong influence on fluid flow process and the solute transport. In order to understand and to predict flow and transport phenomenon through fracture, this paper describes a newly developed method for the simultaneous evaluations of fracture aperture, concentration distribution of tracers and fluid velocity in a fracture by means of light-emitting diode (LED) transmission measurement and particle image velocity (PIV) combine system. It was utilized to accomplish the visualization of flow and solute transport, and to evaluate quantitatively the effect of fracture aperture correlation on temporal flow and transport process through a fracture (see Fig. 1 and Fig. 2).

Keywords: Tracer transport; Visualization; Fracture; Aperture distribution; Flow and tracer test; Light-emitting diode (LED); Particle Image Velocity (PIV).

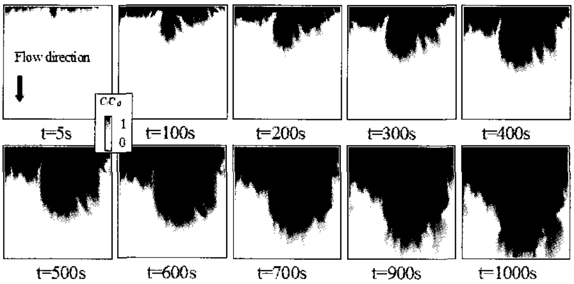

Fig. 1. The temporal change of tracer relative concentration (C/C_0) in a fracture (Q = 3.333mm^{3}s^{-1}).

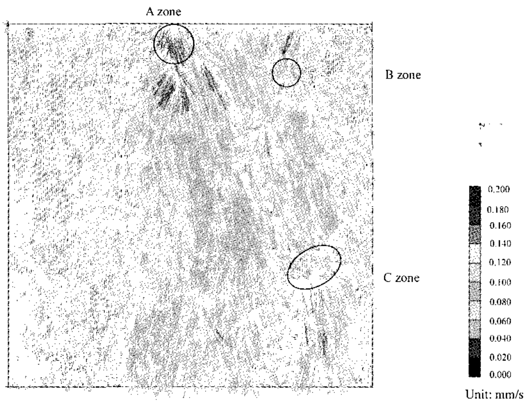

Fig. 2. Trace two-dimensional velocity field of fracture replica investigated by PIV method (central region of fracture, area: 70mm×100mm; A, B, and C zones mean high aperture values zone).

SEISMIC SOURCE THEORY OF ROCK BURST AND ANALYSIS OF BURST PROCESS

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The unity of rigidity theory, energy theory and instability theory of rock burst was discussed. Then seismic source theory of the rock burst was put forward, which considers rock burst as a dynamic process which is composed of three sub-processes: the formation of the seismic source, the formation and transmission of stress waves and the failure and burst of coal mass near the free face under the effect of stress waves. The energy causing rock burst is supplied by the seismic source and the damages are caused by stress waves. The theory of seismic source discloses the mechanism of rock burst by studying the process of its occurrence. After that the process of rock burst was analyzed, which disclosed law of seismic source generating. Calculation formula of the biggest stress peak of stress waves was deduced. Stress wave propagating decay law was presented. Breakage action of stress waves on coal mass near free face was explained. In detail, strain localization of coal mass results in formation of fracture zone that causes rapid softening of coal mass, which provides the condition of the instability failure of coal mass. The instability failure is threshold of rock burst kinetics process and the sustaining of this process leads to the occurring of bigger fracture plane. At the same time, a great deal of elastic energy is released and spread through surrounding coal and rock mass in form of stress waves. When the stress waves reaches free face, the coal mass of the free face will break and acquire certain kinetic energy under the action of stress waves.

Keywords: Rock burst; Seismic source theory; Process; Stress wave.

INDUCED FRACTURING IN THE OPALINUS CLAY: AN INTEGRATED FIELD EXPERIMENT

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The Opalinus Clay is under consideration as a potential host rock for deep geological disposal of radioactive waste in Switzerland. However, building a geological repository produces a system of induced fractures resulting in an Excavation Disturbed Zone (EDZ). A new experiment, EZ–B, was initiated at Mont Terri to improve the understanding of the EDZ formation in the Opalinus Clay. In the experiment, a new test niche was excavated in a stepwise manner and the rock mass response monitored via surface and borehole methods. The main objectives of the experiment were to obtain a comprehensive description of the EDZ formation in response to the niche excavation and to identify the key mechanisms that influenced its formation.

The methods employed in the field investigation included geological mapping of the niche surfaces, drill core logging and analysis, pore water pressure monitoring, geodetic displacement measurements, seismic single- and cross-hole measurements and tomography, optical imaging of the niche and in the boreholes, and atmospheric monitoring in the vicinity of the niche.

The field investigation consisted of three stages: pre-, syn-, and post-excavation. In the pre-excavation stage, two sets of boreholes were drilled: one for pore water pressure monitoring and one for geophysical monitoring. Geological mapping, borehole geophysical and displacement measurements, and optical imaging of the niche were carried out to establish the local geology and zero measurement datums. In the syn-excavation stage, the niche was excavated in seven steps. Following each excavation step, the same measurements were again carried out. In the post-excavation stage, a third set of boreholes was drilled orthogonally to the niche axis. The same borehole measurements as those of the previous stages were again carried out. In addition, repeat measurements of the entire dataset were (and continue to be) carried out in this stage to monitor the evolution of the niche EDZ. Each element of this dataset will then be amassed and integrated to develop an experimental model of the niche EDZ, which will be used as a tool for the location of significant changes and the determination of the time over which these changes have occurred. The experimental model will then be used to identify critical EDZ fracture mechanisms and factors via numerical modelling. In this paper, the approach used in the field investigation along with preliminary results will be presented.

Keywords: Rock; EDZ; Field Investigation; Opalinus Clay; Mont Terri.

11. DAMS AND SLOPES

11.1. General

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STABILITY ANALYSIS OF A POTENTIALLY TOPPLING OVER-TILTED SLOPE IN GRANITE

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This paper presents the stability analysis of an over-tilted slope in an ornamental pink granite quarry. Due to the small dimensions of the quarry the eastern first bench of it was performed following the occurrence of a fault dipping around 70° counter-slope. According to this, a 14-meter high slope dipping 70° towards the slope was excavated (Fig. 1.a). Experience dictates that this sort of slopes tends to instability. In this way a stability analysis was due in order to analyze its stability and to study whether this slope can be continued one bench deeper.

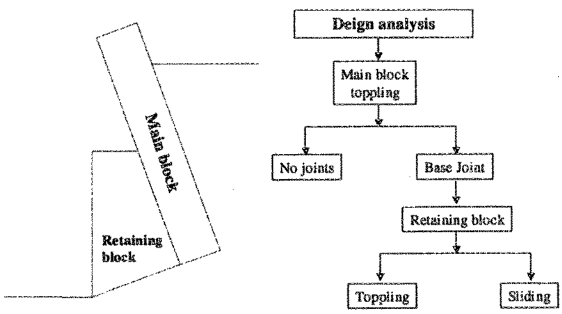
The very good quality granite rock mass involved in the slope has been studied, discontinuity data taking was performed and some laboratory testing was done in order to characterize the main significant parameters involved in the study and the geometry of the slope.

A stability analysis of this first bench was performed by means of the calculations of safety factors against toppling of the slope. These calculations have been done contemplating various possibilities regarding the occurrence or not of joints normal to the slope as observed somewhere else in the rock mass. This has shown that although the stability of the first bench can be assured, if the quarry continues deepening the slope will probably fail. Under these circumstances the second bench of the quarry is designed, taking into account the different possible instability mechanisms according to the geometry and flowchart presented in Figure 1.b.

Then a design is proposed and analyzed able to ensure the stability of the two-bench slope. It is interesting to remark how this design is able to avoid the toppling of the retaining block, for this type of mechanism, if happens, is more destructive than sliding. These analytical results have been compared to numerical ones (UDEC) showing a good agreement.

Keywords: Stability analysis; over-tilted slope, toppling, slope design.

(a)



(b)

Figure 1. (a) Picture of the slope whose stability is analyzed in the presented case study. (b) Basic design geometry and flow-chart to perform safety factor calculations.

PREDICTION OF POST-FAILURE MOTIONS OF ROCK SLOPES INDUCED BY EARTHQUAKES

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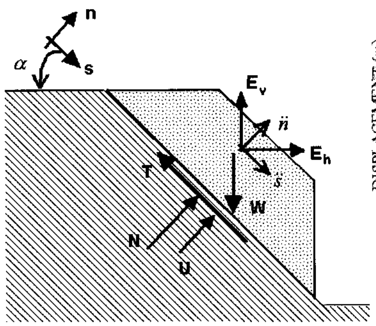
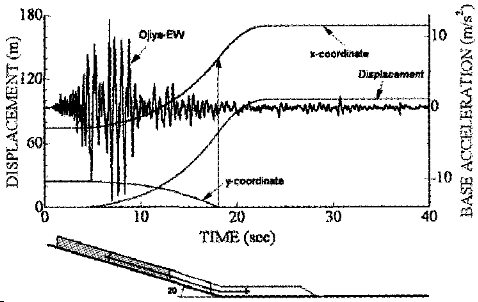
Mountainous areas are often associated with problems of slope instability and complex landslides. In this article, a theoretical model for the simulation of the landslides with the consideration of Bingham type yielding criterion together with water pressure increase due to ground shaking resulting from earthquakes has been presented and it is applied to laboratory rock slope models, actual landslides caused by 1999 Chi-chi earthquake, 2004 Chuetsu earthquake and 2005 Kashmir earthquake. Furthermore, the possibility of motion of the unstable mass of Kuzulu landslide in Turkey in the case of an earthquake similar to that of 1992 Erzincan earthquake has been investigated. The computational results yielded likely displacement, velocity and acceleration responses of landslide bodies, which may be used for the assessment of impact effects of landslides on structures. Although the method is simple, it is very effective and reliable for assessing the motions of landslides and their effect on structures.

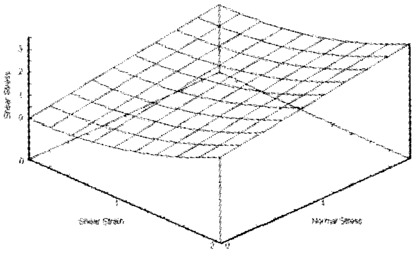
Keywords: Failure; rock slope; earthquake.



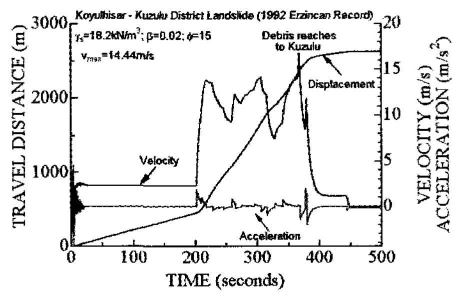
(a) Mechanical model



(a) Shiraiwa landslide



(b) Bingham-type yield criterion



(b) Kuzulu Landslide

Figure 1. Mechanical model for post-failure motions of rock-slopes.

Figure 2. Computed responses for Shiraiwa and Kuzulu landslides.

CALCULATION OF DETERIORATION DEPTH OF ROCK SLOPE CAUSED BY FREEZING-THAWING IN KOREA

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Freezing and thawing cycle is one of the major mechanical weathering-induced factors in rock mass. This natural process accelerates rock weathering process by breaking down the parent rock materials and makes soil or weathered rock formation in a rock slope surface zone. It can also cause reduction of the shear strength in slopes. It is important to calculate the deterioration depth caused by freezing-thawing for a slope stability analysis.

In this study, deterioration depths of rock slope due to freezing-thawing were calculated using the 1-D thermal conductivity equation. The temperature distribution analysis was also carried out using collected temperature distribution data for last 5 years of several major cities in Korea. The analysis was performed based on the rock types distributed in study areas. Thermal conductivities, specific heats and densities of the calculation rocks are tested in the laboratory. They are thermal properties of rocks as input parameters for calculating deterioration depths.

We calculated deterioration depths of rock slopes for 8 rock types in 4 areas. The results showed that the deterioration depth of rock in Korea ranged from 7.8 to 9.2 meters. Since the difference in latitudes of Korea is not large, the influence of thermal temperature distribution on deterioration depth is not significant. The deterioration depth is in proportion to thermal conductivity and specific heat by rock type. The mudstone of which the thermal conductivity is significantly lower than other rocks, showed the least deterioration depth. In further studies, deterioration depths in wider areas and rock types will be estimated in order to reflect the results on the analysis of slope stability.

Keywords: Deterioration; rock slope; freezing-thawing.

SLOPE STABILITY ANALYSIS AND DETERMINATION OF STABLE SLOPES IN CHADOR-MALU IRON MINE

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One of the critical aspects of mine design is slope stability analysis and determination of stable slopes. In Chador-Malu iron mine, which is one of the most important iron mines in the central of Iran, the field investigation and operator reports showed that instability problem will happen and it is vital to perform a comprehensive slope stability analysis and determine the stable slope. For the first step the existing rocks hosting pit were divided into six zones which have same geotechnical characterization. At the second step, geological map was prepared according to existing geological profile and then based on this map and classified zones, geotechnical map was prepared. Then, the value of MRMR (Mining Rock Mass Rating) was determined for each zone. Owing to the fact that Chador-Malu iron mine is located in a high tectonic area and rock mass is completely crushed, it was assumed that rock mass is a continuum medium and Hock–Brown failure criterion was employed to estimate the geomechanical parameters. After that, the value of cohesion (c) and friction angle (φ) were calculated for different geotechnical zones and relative graphs and equations were derived as a function of slope height, considering the fact that these parameters vary with depth in Hoek-Brown new edition criterion. To assess the stability of slope, 14 sections were prepared around the pit. Stability analysis using numerical and limit equilibrium methods showed that some instability problem will take place by increasing the slope height. Two softwares, Flac 2D ver. 5.0 for numerical method and Geoslope ver. 5.15 and Slide ver. 5.014 for limit equilibrium method were employed. These stability analyses demonstrated that noticeable failures will occur if necessary remedial work does not perform. The most problems will be due to existing of weak zones such as Talus and altered Albitite. To solve this problem and determine the stable slope two approaches were considered, determining the stable slope for each zone and the second one, evaluating the stable slope for prepared sections. In the first method, using the aforementioned softwares, different slopes were modeled. In these models, the slope angle and height were changed from 20° through 75° and from 15 m (the height of one bench) through 225 m (maximum height of pit at the western wall), respectively. The value of safety factor against slope angle for different slope height was plotted and then repeated for all geotechnical zones. In the second method, using trial-and-error approach, the angle of unstable zones were changes to reach to a stable condition. By this method, the layout of pit was expanded.

Keywords: Slope stability; mine; case study.

FIRE DAM CONTRUCTION FOR UNDERGROUND OPENINGS

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Mine fires are one of the major hazards of underground mining such as coal, metal, and non-metal in terms of the safety, economic, and environmental aspects, and it is classified mainly two broad groups that are open and concealed fires. Open fires occur in airways, faces, and other openings, hence they affect the quality of the mine-air flows, quickly and directly. Concealed fires occur in area where is difficult to access the fire area. If the source of fire does not known or the fire reaches uncontrollable dimensions, it is impossible to interfere of the fire directly. Therefore, intake and return air way of underground mine must be closed with fire dams in order to prevent the oxygen entrance in the fire area. The purpose of the fire dam constructions is; to stop the flow of air into the fire area, to put out or prevent the spreading of the fire, and to hold a probable methane explosion in the fire area. The material used for constructing fire dams must be economical, air tightness, easily transportable, applicable, and having efficient strength properties to resist a probable explosion. The mechanical and physical properties control the fire dam material desired. Different materials have been used to build up fire dam such as natural anhydrite, synthetic anhydrite, hemihydrates gypsum, cement, stone dust, bentonite, clay, etc. In this research, gypsum material has been investigated in order to understand the suitability of the material for underground fire dam structures. The experimental studies were carried out to understand the uniaxial compressive, tensile, and bending strengths of the material for different water/gypsum ratio and setting times. The water/gypsum ratio used in the experiments were 0.5, 0.6, 0.8, and 1.0, and the setting times were 1 hour, 8 hour, 12 hour, 16 hour, 1 day, 7 day, 14 day, 21 day, and 28 day, respectively. The uniaxial compressive, tensile, and bending strengths of gypsum material change depending on the parameters of water/gypsum ratio and setting times of the mixture. The best results obtained from the experiments are valid for 0.5 water/gypsum ratio as given in Figure 1. The results show that the uniaxial compressive, tensile, and bending strengths of the gypsum material decrease with the increasing trend of the water/gypsum ratios.

Keywords: Fire Dam; Mechanical Properties; Gypsum Material.

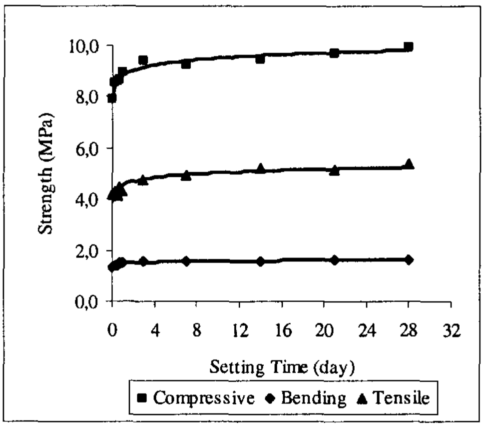

Fig. 1. The variation of strength related setting time for 0.5 water/gypsum ratio.

ANALYSES ON THE ROCK SLOPES USING HAZARD AREA ESTIMATION SYSTEM FOR ROCK MASS FAILURE DEBRIS

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Hazard and risk management of rock mass failure, which can lead to natural disaster-scale damage and fatalities, is of considerable importance. Hazard maps are believed to be a good approach to the management of slope disasters, but a precise and effective understanding of the hazard area is first required. We have previously proposed a reliability prediction method for rock mass failure hazard areas using Type I quantification theory, a form of multivariate statistical analysis, and have since developed a hazard area estimation system using these analysis results. The process of calculation and analysis using this system is as follows.

1) A DEM (digital elevation model) is constructed, with the elevation displayed by color classification.
2) A failure point is specified by mouse click, and slope conditions, such as geology and topography, can be input.
3) The prediction value of the hazard area is computed from the input slope conditions.
4) The reliability prediction is calculated from the computed hazard area.
5) The calculated reliability prediction is classified by color and displayed as part of the DEM.

In this study, we verified our prediction system by applying it to actual failure examples, and evaluated the debris-scattering pattern. Understanding the debris scattering pattern is the key to building a hazard map.

Our results demonstrated that this prediction method can be applied to actual failure examples. When we treated the debris-scattering pattern in the prediction as a rectangle describing total horizontal reach and spread width, the actual debris fell within 40–60% of the predicted value (Fig. 1). However, the actual debris scattering pattern may spread farther than the predicted value depending on the slope conditions.

This system has potential for use as a preliminary screening method to determine and rank the risk of rock mass failure in a slope. For slopes that prove to need urgent countermeasures, a detailed examination should follow using a numerical analysis method such as discontinuous deformation analysis.

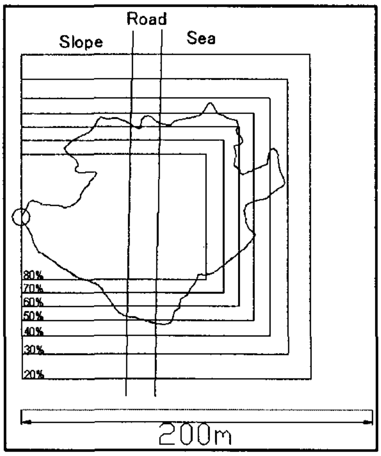

Fig. 1. Hazard area and the debris scattering pattern.

Keywords: Slope disaster, rock mass failure, hazard map, risk assessment, multivariate statistical analysis, reliability prediction.

STUDY ON THE DYNAMIC RESPONSE AND PROGRESSIVE FAILURE OF A ROCK SLOPE SUBJECTED TO EXPLOSIONS

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Drill and Blast is a commonly used method for excavations of open pits. During blasting excavation, the additional dynamic load induced from blasting may result in the failure of rock slopes. In order to ensure the stability of rock slopes and control the blasting design, determining the factor of safety, and safety threshold velocity of them are usually necessary and important for the engineering utilization. For this purpose, studying the dynamic response, such as the velocity, acceleration, displacement as well as the stress of rock slopes under explosions is always the first step. In the present paper, the dynamic response of jointed rock slope under explosions is simulated using universal distinct element code (UDEC). In numerical modeling, the input dynamic load is adopted as the velocity history close to the explosive source obtained by field measurements directly. Then the simulated results are compared with field measurements. It is shown that the deviation between the simulated peak particle velocity and that of site monitoring is less than 20%, which is indicated that the simulated results agree well with the measured results. Therefore, UDEC can be used to simulate the dynamic response of jointed rock slope effectively.

In addition, the progressive failure process of the rock slope under explosions is also simulated by UDEC. During the numerical simulation, the slope is stable initially under the above described input load. Then, keep the dominant frequency of the input velocity history as constant, and amplify the magnitude gradually until the slope fails. Hence, the simulation results for progressive failure of the slope subjected to explosions are obtained. It is observed that the failure process of the slope can be divided into three phases, the formation and growth of local failure area as well as coalescence of sliding plane.

Keywords: Jointed rock slope, Explosions, Dynamic response, Progressive failure.

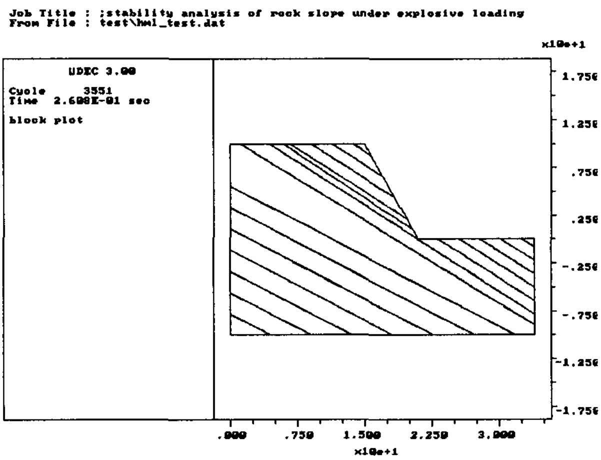

Fig. 1. UDEC model of the rock slope.

APPLICATION OF ANNS TO PERMEABILITY ANALYSIS AT THE SHIVASHAN DAM, IRAN

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This study investigates potential of artificial neural networks (ANNs) to predict permeability at the ShivaShan hydroelectric earth dam on northwestern of Iran.

ANNs are computer systems composed of a number of processing elements that are interconnected in particular topology which is the problem dependent(complexity of problem). ANNs have the ability to learn from examples using different learning algorithms.

In this paper, we applied BP networks to predict of permeability (with Lugeon test). Finally using SOM (self-organizing map), we could classify the dam rock site in three zone.

Keywords: Neural networks; Lugeon test.

NEW IMPLEMENTATION APPROACH OF THREE-DIMENSIONAL SLOPE STABILITY ANALYSIS USING GEOGRAPHICAL INFORMATION SYSTEM

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Currently, even though some computer programs are commercially available for three-dimensional (3D) slope stability analysis, practical application of the 3D method in engineering issues is still lacking. A suitable method that using a common data form for 3D analysis is urgently required. A Geographical Information System (GIS), as a computer-based system, allows easy data updating, modeling, nice result presentations, and complex spatial analysis. It is ideal and anticipant to embed the 3D models within GIS environment.

In this paper, a new GIS raster-based 3D deterministic model is proposed to accomplish the combination of the column-based 3D method and the GIS raster data. Multiple column-based 3D methods can be adopted in this model by extracting all slope-related information from study area to create a set of corresponding raster data layers. The raster data can be efficiently analyzed within a column-based 3D model to calculate the 3D safety factor. A GIS-based system is developed to implement all processes. A comparison with other existing programs using two widely used examples validates the correction of the proposed algorithm while the effect of the number of columns using in discretization of the slope on resultant safety factors is also discussed.

This approach is shown to have the ability to simplify the data-input procedure and the column discretization, and to expedite the safety factor computations. The use of GIS data form and GIS functions provides the convenience in renewal of the data and in multiple cases study. Safety factors obtained by this model showed a little overestimation compared with other published 3D methods due to the use of more columns. The number of the columns using in discretization of the slope was found out to be 20000 for making the resultant safety factor fixed.

Keywords: 3D slope stability analysis; column-based method; GIS raster-based 3D model.

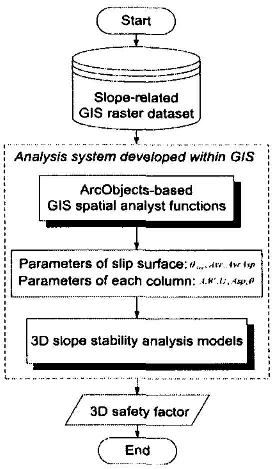

Fig. 1. Flowchart of computational procedures.

INVESTIGATION ON DAM FOUNDATION GROUTING PROCESS

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In this paper, we propose a mathematical model, which can express the injection process of the grouting in dam foundation. Then, we predict injection duration time and grout takes, etc. obtained from early injection process data of an existing dam, and we examine the validity of our proposed model. If we can predict the whole injection process in dam foundation obtained from early injection data, we can rationally determine the injection parameters such as injection pressure, flow rate and grout mix proportion.

Eq. (1) is the proposed model based on the cube law between crack width and grouting flow rate.

$$\frac{Q_t}{P_t} = \frac{Lu_0}{0.98}\left(1 - \frac{\beta}{D_0}t\right)^3 \frac{1}{\mu} \tag{1}$$

Fig. 1 shows an example of observed injection data and calculated data. When we investigate the results of the proposed model, injection patterns are classified into three (Table 1), and we make detailed investigation of the parameters of the proposed model according to three classified injection patterns.

Keywords: Dam Foundation; Grouting; Grout Mix Proportion; Mathematical Model.

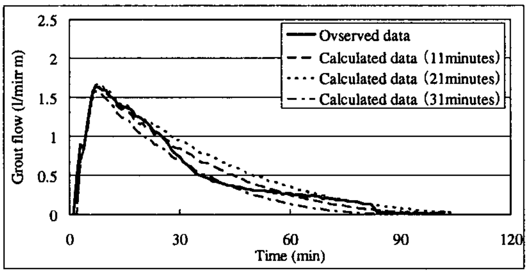
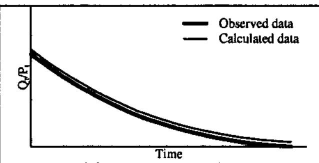
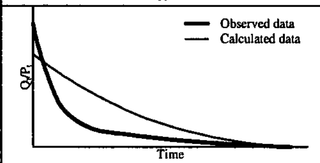
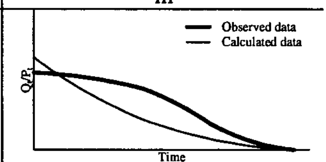


Fig. 1. An example of comparison of observed data and calculated data.

Table 1. Pattern types of injection charts.

Pattern	I	II	III
Grouting injection process			
Number of data	37	31	5

SLOPE DEFORMATION CHARACTERISTICS AND INSTABILITY ANALYSIS

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In large-scale slopes which have great influence on the safety of important buildings, large scale deformation monitoring system was established for detecting unstable signs and taking timely measures to safeguard the stability of slope. After establishment of a deformation monitoring system, handling and analyzing deformation information for controlling stable state of slope is an important work of engineering technicians.

The Wuqiangxi Hydropower Station in Yuanshui River in western Hunan Province is a large hydropower station with capacity of 1,200,000Kw. The height of the dam is 87.5m. The navigation building is a three-stage ship-lock, situated in the left bank. The height of the left bank ship-lock slope is 165m. It's geologic structure is very complex. For the safety of the ship-lock slope, a large large-scale deformation monitoring system is established on the ship-lock slope. The system formed an integrated monitoring network based on a combination of point, line and plane.

The authors analyzed the deformation characteristics of the left bank ship-lock slope of Wuqiangxi Hydropower Station, based on Information Visual Inquiry and Analysis System.

The stable state of the slope could be graded. Each grade could be distinguished by the deformation information. All desirable distinguishing sign for each grade was programmed. For each situation, according to inquiry and analysis of deformation information, combined with expert experience, especially experience on-the-spot experts, the sign analysis method and criterion of the research slope were determined. Then, on-the-spot technicians could analyze slope stable state dynamically, using the slope sign analysis branch system. With the promotion of understanding degree of experts for the slope, criterion could be change dynamically. The Slope Instability Sign Analysis System was established for the left bank ship-lock slope.

We analyzed the stability of the Wuqiangxi Hydropower Station left bank ship-lock slope based on Slope Instability Sign Analysis System. The slope unstable sign analysis indicated the stability of the slope belonged to slight unstable type, its tendency should be paid attention to. The actual application in the left bank ship-lock slope of Wuqiangxi Hydropower Station shows that the Slop Deformation Monitor Information Analysis System has strong practicality and reliability.

Keywords: Slope; Stability; Monitoring; Deformation characteristics analysis; Instability analysis.

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11.2. Theoretical and Numerical Analyses

NUMERICAL SEISMIC STABILITY SAFETY EVALUATION FOR ROCK SLOPES

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Rigid block ultimate equilibrium method is usually used to determine the safety factor against sliding for rock slopes. The disadvantage of this method is that the elastic material properties can't be considered. When the earthquake is taken into account, the quasi-static forces are usually used to simulate the earthquake loads and the earthquake wave propagation properties can't be considered.

The strength against sliding gradually decreased method based on FEM is used to determine the safety factor under the static loads by many researchers. The limit state is defined as that all the elements on the sliding surface are sliding. Some researchers take the ratio between the total force against the sliding and the total sliding force as the safety factor for the seismic stability analysis when the earthquake process is simulated.

The shear strengths are decreased mainly due to the time effectiveness, but the earthquake usually occurred in a few seconds. In other words, the shear strengths can be taken as constants in the earthquake process. But we need to consider that the earthquake may occur in any time and the initial state is different at different time.

According to the above concepts, a new FEM seismic stability safety evaluation is suggested. The shear strengths are divided by the different strength-reduction ratios and the seismic analysis is made when the static analysis finished. The seismic safety factor is corresponding to that all the elements on the sliding surface are sliding under earthquake loads. The earthquake waves are put on the artificial boundaries and the viscous radiation conditions are used to simulate the outwards wave absorption.

An example is given to explain how to evaluate the seismic stability safety for rock slopes by FEM. The stability of rock slope with four broken line sliding surfaces is analyzed, and the results show that under the static loads, 1.35 can be regarded as the biggest safety factor of static stability in the linear elastic state and 1.6 can be regarded as the safety factor of static stability in the limit stability state, and under the dynamic loads, 1.15 can be regarded as the dynamical stability safety factor.

Keywords: Strength-reduction ratio; Seismic stability; Safety factor; Artificial boundary; Rock slope.

STUDY ON THE PREDICTION OF THE HAZARD AREA DUE TO ROCK SLOPE FAILURE BY USING NEURAL NETWORK SYSTEM

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Slope disasters occur every year. Especially, rock slope failure is one of the serious slope disasters. It occurs gustily, quickly and largely somewhere. The failure affects enormously not only social loss and economical loss but also loss of human life. Therefore, it is necessary to improve skill of maintenance for rock slope failure. However, there are a lot of places that rock slope failure is apprehended in Japan and it is difficult to take measures directly to protect rock slope failure. In addition, in limited means and time, we must develop effective system to manage rock slope failure. Hence, it is essential to make the hazard map in order to decide a priority for measurements and inform people of the hazard area due to rock slope failure. Here, as an attempt to the prediction, we used neural network system. This system was one of the multivariate analysis. Neural network system is a kind of data mining and is well suited to problems involving matching input data to output data. This is an appropriate system in this case because the connection between slope conditions and the hazard area due to rock slope failure is opaque. In order to assess the hazard area, we selected travel distance and spread width as objective variable and failure height, failure volume, failure width and slope angle as explaining variable. Explaining variable was selected on the basis of correlation between explaining variables and objective variables. We applied these variables to neural network system and predicted the hazard area. In addition, we picked out optimal combination of explaining variable on the basis of sensitivity analysis. We tried to improve the prediction of the hazard area. Finally, we could recognize that neural network system was efficient. We concluded that we could evaluate the prediction of the hazard area by using neural network system.

Keywords: Rock slope failure; hazard area; hazard map; neural network system.

SIMULATION ANALYSIS OF TOPPLING FAILURE OF ROCK SLOPE BY DISTINCT ELEMENT METHOD USING BONDING THEORY

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As is known, there are many fractures in rock slope, and these fractures are often the cause of failure. Especially, the mechanism of toppling failure depends on fractures in rock slope. In this paper, Two-dimensional simulation analysis and visualization for toppling failure by distinct element method are carried out. In relation to this simulation analysis, the tensile force of rock mass can be tried to be expressed by bonding theory.

An analysis object in this study is a rock slope failure which arose at Amatoribashi-nishi site in Wakayama Prefecture. This slope is a steep cliff of a mountain located on the side of a national road which runs along the coast, and is composed of sandstone with the height of 17m. The fractures between each blocks was greatly opening (Figure 1). In March, 1999, it failed. The failure is classified into toppling failure.

Figure 2 shows the failure behavior obtained by the analysis. It was proven that the aperture of the fracture in back of the block A3 gradually increases, and that the block topples, although the failure was not completely brought to an end. And, it can be also confirmed that a thin block in front of the block A1 is being detached with toppling of the block A3.

Keywords: Distinct Element Method; Rock Slope; Bonding Theory.

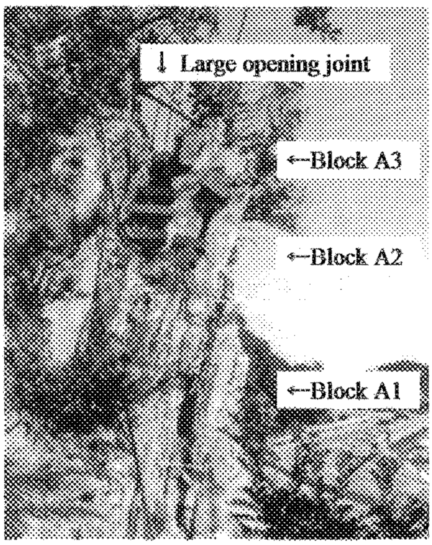

Fig. 1. The side-view of the slope.

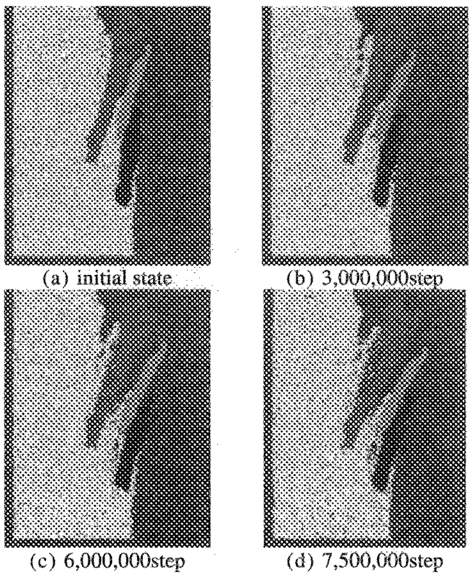

Fig. 2. Failure behavior.

SEISMIC RESPONSE ANALYSIS AND EARTHQUAKE-INDUCED SLOPE FAILURE– A CASE STUDY OF LAS COLINAS LANDSLIDE, EL SALVADOR

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Ground motion was found to be the primary factor in earthquake-induced slope failure. Seismic-induced ground motion is often sufficient to trigger landslides in slopes that have low to moderate stability in static conditions. The minimum ground acceleration or earthquake magnitude to trigger landslides depends on the earthquake duration, terrain characteristics, and geotechnical parameters of the potential sliding mass.

The primary goal of this research is to evaluate the topographic effect of ground motion and seismic-induced slope stability in the Balsamo Ridge area where the most damaging landslide destroyed the neighborhood of Nueva San Salvador in El Salvador during the January 13, 2001 earthquake. The topographic characteristics of the hill increased the magnitude of ground vibration. Peak ground acceleration and displacement at the ridge top were significantly amplified. The shaking-induced displacements are shown in Fig. 1. The northern slope near the crest had a horizontal displacement up to 105 cm and vertical displacement about 30 cm. This simulation is an agreement with the actual failure area of the Las Colinas landslide during the January 13, 2001 earthquake. The analysis indicates that the slope near the crest on the north side is the critical area in which the slope is most likely to fail. The sensitivity analysis of the effect of horizontal earthquake acceleration on the stability of the Las Colinas slope is indicated that the critical horizontal seismic coefficient of the landslide was 0.24~0.38. The horizontal acceleration during the January 13, 2001 earthquake may reach 0.96 to 1.52g.

Keywords: Seismic, earthquake, slope stability, landslide, topographic effect, El Salvador.

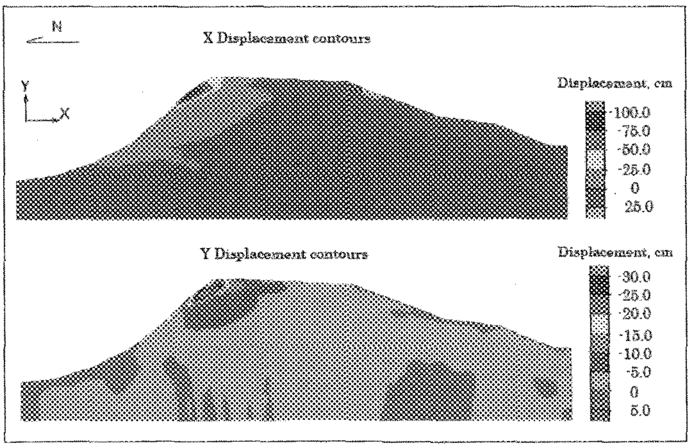

Fig. 1. Contour map of seismic-induced displacement. The maximum displacement occurred at the north side of the ridge near the crest. The maximum horizontal displacement (X) is 105 cm, and the maximum vertical displacement (Y) is 30 cm.

A NEW GROUND WATER ANALYSIS METHOD WITH RAINFALL FOR SLOPE STABILITY EVALUATION

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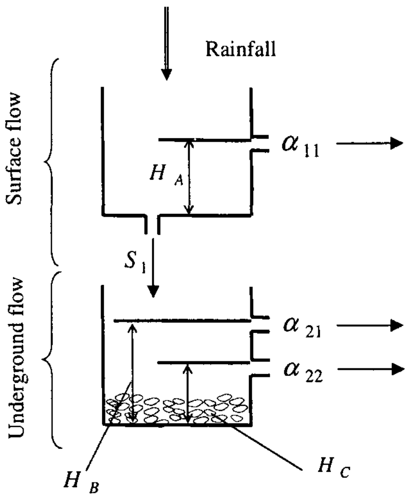
In Japan, slope failure due to rainfall is a common disaster, which leads to damages and losses. So, we have to reinforce the instable slopes. If we can foresee the dangerous of the slope, this reinforcement might be more effective. In order to foresee a slope collapse, it is indispensable to understand the underground water behavior. This research develops saturated and unsaturated seepage flow analysis into a dynamic method involving the water budget conception by setting the tank model at the side.

Keywords: Slope failure; Saturated and unsaturated seepage analysis; Tank model.



$\alpha_{11} + \alpha_{21} + \alpha_{22}$: Flow to river

S_1 : Infiltration to underground

Fig. 1. Tank model.

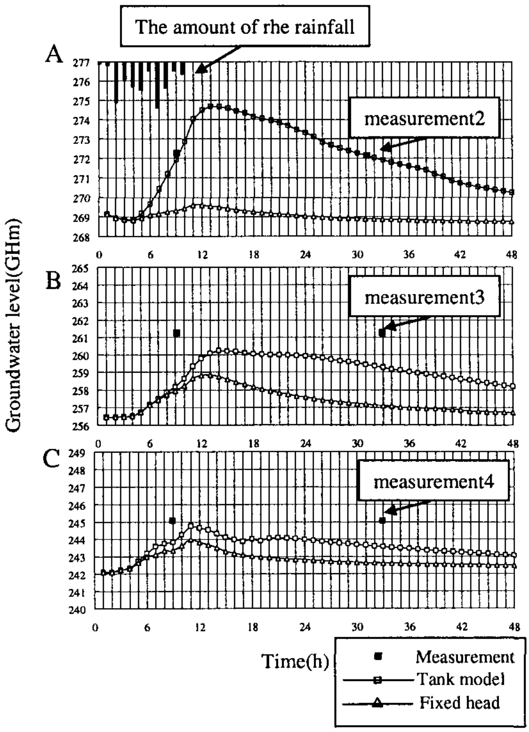


Fig. 2. The difference between evaluations using saturated and unsaturated seepage analysis with tank model & fixed head.

THREE-DIMENSIONAL STABILITY OF SLOPES WITH BUILDING LOADS

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Three-dimensional slope stability is necessary to slopes with building loads.Based on extended Spencer method,an approach considering building loads is put forward.

$$S=\sum[N_i(-n_x\cos\beta+n_z\sin\beta)+T_i(-m_x\cos\beta+m_z\sin\beta)-(W_i+Q)\sin\beta]=0$$

$$M=\sum[-(W_i+Q)x_i-N_in_xz_i+N_in_zx_i-T_im_xz_i+T_im_zx_i]=0$$

$$Y=\sum(N_in_y+T_im_y)=0$$

Considering difficulty in determining three variables including safety factor(F),angle of side force with horizon(β),angle of anti-slip force with y coordinate axis(ρ),genetic algorithm is utilized with the table as followings.

Table 1. Implementation of genetic algorithm.

F										β							ρ							
0	1	0	0	0	0	0	0	1	0	0	1	1	0	0	1	1	0	1	0	0	0	1	0	0

Based on the assumption of elliptical critical slip surface,with a case as background,the critical slip surface is determined and corresponding safety factor is calculated.

Keywords: Dimensional slope stabilty;building load;extended Spencer method;genetic algorithm.

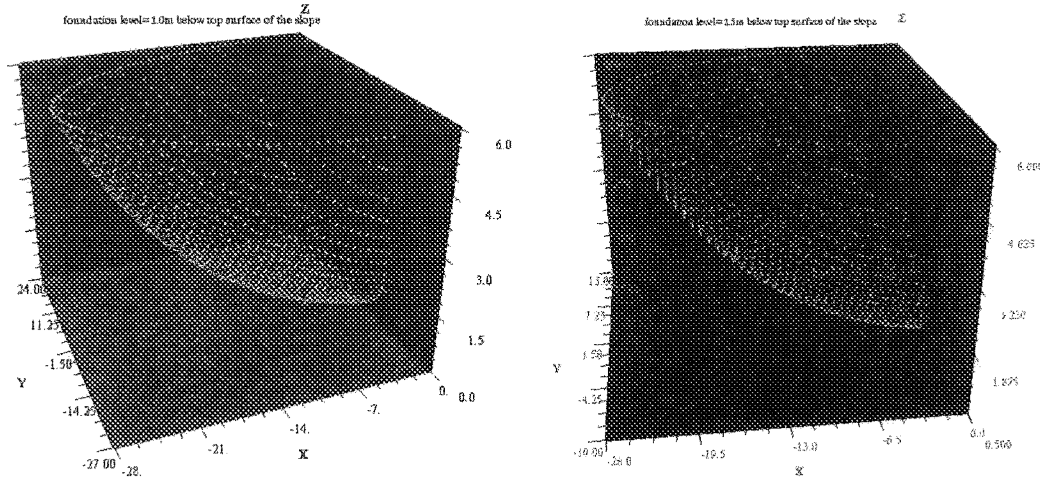

Fig. 1. Critical slip surface with different foundation levels.

ON REFINED FEM SOLUTION TO SEEPAGE IN ARCH DAM FOUNDATION

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On the basis of the senior author's experiences in study on solution and control techniques for problems of seepage and the facts of the very strong inhomogeneity, anisotropy and intricate performance of seepage, complicated arrangements of grout curtains and drainage galleries located in both longitudinal and transverse directions and consisted of densely distributed numerous drainage holes, which are with small diameter, long and have overwhelming impact on control of seepage, and other agents and control measures involved in the seepage field in the rock masses of arch dam foundations and abutments, the strict theory and numerical techniques for solution to problems with finite element method are discussed and presented in detail. The numerical procedures for the exact simulations of the seepage behaviors of every type of drainage opens and different kinds of drainage holes are proposed by the authors are strict, precise and reliable in both aspects of theory and algorithm. The results from the proposed method are robust and the method has been proved to be an effective and powerful approach and it has been successfully introduced into engineering practices of dams, underground openings, slops and foundation constructions. Finally, in order to verify the ability of the application and algorithm proposed and to show the reasonable results calculated, a practical example of seepage in a high arch dam foundation and abutments with a whole large 3D model including the foundation and the two abutments of the arch dam is strictly solved and analyzed. The optimum design scheme of seepage control in the dam foundation and abutments have been also discussed and suggested in detail.

Keywords: Seepage, anisotropy and inhomogeneity, arch dam foundation, drainage curtain, drainage substructure, boundary conditions.

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11.3. Field and Laboratory Studies

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COMPREHENSIVE BACK ANALYSIS TECHNIQUES FOR ASSESSING FACTORS AFFECTING OPEN STOPE PERFORMANCE

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A methodology and a number of software tools have been developed to assess the operational and geotechnical factors affecting the performance of open stoping operations. Open stope performance is generally measured by the ability to achieve maximum extraction with minimal dilution. The paper describes the tools and techniques for collecting and analysing common factors affecting performance, such as drill and blast, development undercutting, stress induced damage, rock mass quality, and large scale geological features.

Common back analysis techniques, such as empirical stability graph methods, are limited in their ability to identify and quantify the relative contributions of the various factors that influence excavation performance. The paper proposes a methodology that enables the evaluation of a variety of contributing factors simultaneously. The proposed methodology also enables the evaluation of spatial variability in various parameters under consideration.

The proposed methodology relies on the use of mathematically implicit surfaces, defined by radial basis functions, to generate volumes and iso-surfaces of various parameters and/or candidate criteria. These surfaces and/or volumes can be used to query the rock mass volume using intersections and unions of the implicit surfaces and are similar to Boolean operators.

The results of these queries can be used to develop hypotheses to ascertain the relative influence of the contributing factors on stope performance. These can be confirmed by conducting critical reviews of stope performance data, and then used to assist development of forward analysis criteria, design parameters and changes to implementation practices. Example data has been collected and analysed with some results presented

Keywords: Back analysis; open stoping; dilution; over-break.

PRESENTING A TECHNICAL-ECONOMICAL SOLUTION FOR "ROCKFALL" CONTROL IN SECTION I OF ROCK SLOPE FACING TEHRAN-FASHAM ROAD

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Tehran-Fasham road is one of the main routes connecting the capital, Tehran, to the north-western satellite towns and villages in the province of Tehran. Some parts of this road (between Kms.20 to 20+500) have been constructed through rocks (using a trench). Every year, especially at the beginning of spring, some rock blocks separate from the main rock mass and fall down to the road, causing human and property damage and so, making it necessary to take some measures to control the falls.

For the above purpose, considering geomechanic characteristics of the rock, topographic conditions of the area, and situation of the road, the project was divided into 3 sections (section I: Kms.20 to 20+100, section II: Kms.20+100 to 20+350, section III: Kms.20+350 to 20+500)(In this paper we are dealing with section I only).

For present the stabilization approach, first, the geometrical properties of the rock mass discontinuities were determined. There were three joint sets and one bedding in this rock mass (Table 1). Considering stereo plot of rock mass discontinuities, road slope face plane, Rackfall software and site observation, it was determined that rock slope, as a whole, is stable and the main factor for the rockfall, is tectonized nature of rock mass and erosion.

Then, the maximum rock block weight that can fall on to the road was calculated. It was 4 tones.

Finally, by using the Rockfall software and considering techno–economy and time constraint, Catchfence & Drape has been considered the most suitable protective measures for this rock slope.

Keywords: Rock slope, Stereographic projection, Rock block, Rockfall, Fasham Road.

Table 1. Geometrical properties of rock mass discontinuities.

Discontinuities	Dip(degree)	Dip direction(degree)	Persistence(m)	Spacing (m)
B	20	097	>20	1.5
J1	85	115	10-20	1.3
J2	72	214	10-20	1
J3	81	338	10-30	0.7

A CASE STUDY OF DEFORMATION MEASUREMENTS OF SLATES AT JAVEH DAM SITE IN IRAN

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One of the most important parameters in analyzing the mechanical behavior of rock masses and to assess the stability of the structures built in and founded on rock is the deformation modulus. Different methods exist for determining the deformation characteristics of rock masses, the most reliable being the use of in-situ testing methods. Flexible Dilatometer tests (FDT) and Plate Load Tests (PLT) are two conventional methods to determine the in-situ modulus of deformation.

A site investigation program was carried out near the city of Sanandaj in Kurdistan province north-west of Iran. It is proposed to construct a dam at this location on the Javeh River in order to provide water for drinking and agricultural purposes. The investigations are aimed to provide geotechnical parameters in order to analyze the suitability of this location and also to obtain required design parameters. The type of rock in this area can mainly be described as dark grey to grey slate or slaty limestone with occasional calcite veins and is generally strong to very strong. As part of the program several plate load and dilatometer tests were carried out in specially prepared galleries. Both tests are based on the theory of elasticity which considers the rock mass as an elastic, isotropic and homogeneous medium. Due to these assumptions, the modulus derived from both tests should be similar. In real conditions however, this is not the case and the modulus determined from PLT is usually greater than values derived from the FDT. A number of uncertainties are associated with each of the above mentioned tests. The aim of this paper is to discuss the results obtained from these tests and compare the results from the two mentioned methods.

Keywords: Deformation Modulus, In-situ test, Plate Load Test, Dilatometer.

SAFETY FACTOR ASSESSMENT METHOD OF ROCK SLOPE USING CENTRIFUGE MODEL TEST

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Sudden falls of rock mass in the steep slope sometimes causes severe damages to traffic facilities in cold region, such as Hokkaido, Japan, and occasionally results in the loss of casualties. The sudden falls are sometimes triggered by the development of sheeting cracks in the rock mass. The assessment of the potential sudden fall is, however, difficult due to the 3-D composition of crack sheets as well as the configuration of the rock slope. In this study the Authors propose a method for the evaluation of the stability of steep rock slope by means of 3-D laser survey of rock slope geometry, mechanical tests on the rock samples, and centrifugal loading tests on the model rock slope.

Photo 1 shows the model of the rock slope in the scale of 1/30 formed with cement mortar; the original rock slope is formed with Neogene pyroclastic rock in Hokkaido, Japan. The details of the geometry of rock slope including the composition of crack sheets was determined based on the digital data obtained through the 3-D survey. A series of centrifugal loading tests on the model, where the test parameter is the depth of crack, makes it possible to calculate the factor of safety regarding the model scale, centrifugal acceleration at failure, failure strengths of both actual and model rock mass. Shown in Fig. 1 is the calculated value if the factor of safety plotted against the depth of crack. This figure makes it possible to evaluate the critical crack depth which gives the factor of safety of unity.

Keywords: Rock slope; Centrifuge; Safety factor.

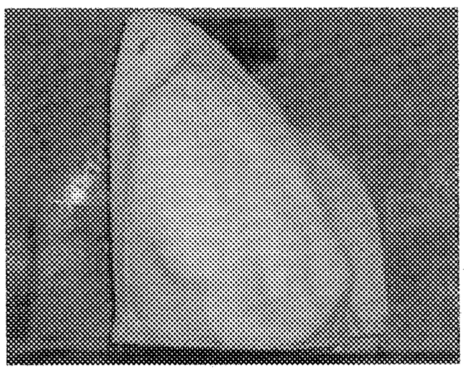

Photo 1. Model of rock slope formed with cement mortar.

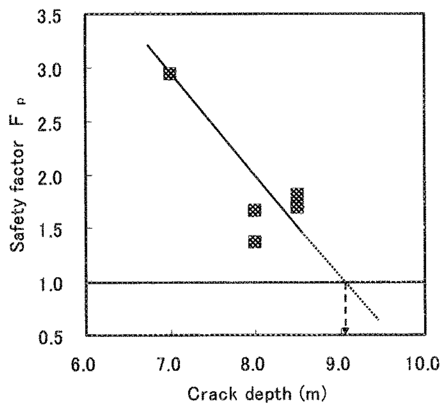

Fig. 1. Calculated factor of safety as a result of centrifugal loading tests.

CASE STUDY ON THE CAUSES FOR THE FAILURE OF LARGE-SCALE ROCK SLOPE COMPOSED OF METASEDIMENTARY ROCKS IN KOREA

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A large-scale rock slope composed of metasedimentary rocks in Korea often exhibits complex formations and geological structures. For the rock slope design, generally, surface geological survey, geophysical explorations and geotechnical investigations are not sufficient to find out these complex site conditions accurately. Thus, landslides occur frequently during construction. In such cases, suitable and economical stabilization methods should be selected and applied through reliable investigations and analysis considering the geotechnical properties.

The rock slope of this study, one of the largest cut slopes in Korea with a length of 520.0 m and maximum height of 122.0 m, consists of metasedimentary rocks. And a case study on the causes of large-scale rock slope failure was carried out by analysis of landslides history and site investigations during construction.

When the slope with the original design slope of 0.7:1.0 (H:V) was partially constructed, the slope failure was occurred due to the factors such as poor conditions of rocks (weathered zone, coaly shale and fault shear zone), various discontinuities (joints, foliations and faults), severe rain storm and so on. The types of failures were rockfall, circular failure, wedge failure and the combination of these types. So, the design of slope was changed three times to ensure long-term slope stability.

This paper is intended to be a useful reference for analyzing the stability of rock slopes whose site conditions are similar to those of this study site such as geological structures and geotechnical properties.

Keywords: Slope stability; large-scale rock slope; metasedimentary rocks; landslides history; discontinuities.

Fig. 1. Slope failure occurrence at Zone 2.

PROBABILITY OF ROCK SLOPE FAILURES AT PART OF A MOUNTAIN ROAD, SAUDI ARABIA

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Al-Baha descent road of 32 km long lies at one of the harshest terrains in Saudi Arabia. A number of rock slope failures have occurred along the road. The natural rock slopes along the descent road has been divided into stations, and each station is subdivided into polygons of natural rock slopes, each polygon characterized by its own attitude. Only 4 km sector has been studied as an example. Stability analyses of all natural rock slopes for each polygon have been conducted, and 4 types of stability zones with a characteristic factor of safety have been derived accordingly. Probability analysis of the main three types of failure has been performed, and subsequently divided the road into zones of different levels of probability of failure. Relative risk factor (R_r)was introduced to calculate the probability of failure. This technique outlined the zones of high probability of failure and support measures should be added to these zones. GIS technique was used to illustrate the stability and total probability of failure (P_{total}) conditions. It was found that areas of high probability of failure are associated with increase of support measures.

Keywords: Mountain roads, rock slopes stability, probability of failure, GIS.

WATER PRESSURE TESTS FOR DAM FOUNDATIONS

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Grouting is a procedure to improve the strength of dam rock foundations and/or to improve water-tightness by injecting cement-based grouts. Recently in Japan, geological conditions at dam sites have become more complicated and there are often difficulties with dam foundation grouting. Besides, we should construct dams at lower cost and with less impact on the environment than in the past. Therefore, we are required to reduce the cost of dam foundation grouting. We have been trying to establish an effective and economical dam foundation grouting.

Many efforts have been successful in rationalizing grouting application. But almost no effective ways of rationalizing the grouting work itself have been proposed. Under these circumstances, Japanese dam engineers have studied methods that reduce the cost of dam foundation grouting, by adopting a water pressure test with monotonous pressurization (MPT-MP), which can reduce the time required for permeability evaluation during grouting.

This paper introduces case studies of the MPT-MP and verifies its precision and cost reduction effects. The results have shown that the WPT-MP is fully applicable as a test to replace the conventional water pressure test with stepwise pressurization (WPT-SP). The results also show that adoption of the WPT-MP reduces the cost of WPT by between 20% and 30%. When the MPT-MP is conducted in an unsaturated region, permeability might be over-estimated, mainly because of the effects of unsteady seepage. Therefore, saturated-unsaturated seepage analysis using the finite element method was performed to study the impact of unsteady seepage of a water pressure test (WPT) in an unsaturated region. Numerical study has clarified that in a case where the test section of the WPT-MP is located in an unsaturated region, the permeability is over-evaluated under the effects of unsteady seepage (see Figure 1).

Based on these results, precautions necessary to perform appropriate dam foundation permeability evaluations using WPT have been proposed.

Keywords: Dam foundation grouting; Water pressure test; Saturated-unsaturated seepage analysis; Unsteady seepage.

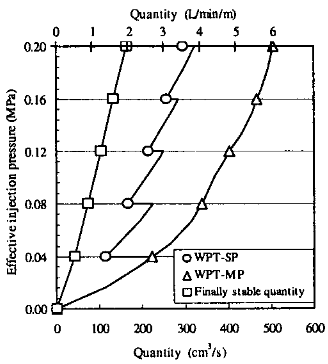

Fig. 1. Calculated P-Q curves.

MEASUREMENT OF THE DYNAMIC BEHAVIOR OF UNSTABLE ROCK BLOCKS EXISTING IN THE ROCK CLIFF

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Generally, the displacement and strain of the rock mass under the static condition have been measured to evaluate the status of unstable rock blocks existing in the rock cliff. After the rock block collapse accident occurred at Toyohama tunnel where many holy human lives were lost, many measurements for the static behaviors of rock blocks have been conducted, to evaluate the unstable state in those blocks. However, it is impossible to measure unstable state realistically. The authors got a chance to measure the unstable rock blocks at the cut off construction of a rock cliff from the beginning of the construction to the artificial collapse of the rock blocks.

The time from the occurrence of the displacement to the collapse of rock blocks is so short that the prediction of the outbreak of the collapse is very difficult. To predict the outbreak of the collapse, some signs that occur before the occurrence of the displacement must be measured. Therefore, a new idea for the measurement was proposed in the following.

The occurrence of the displacement needs the change of the crack state of the part connected to the rock cliff. In other words, the displacement occurs by the accumulation of these small changes of the crack state, and these small changes make some influences to the dynamic property of the rock block. Therefore, in addition to the static behavior measurement, the rock blocks' dynamic behavior at this cut off construction of the rock cliff should also be measured. In this measurement, a three-component seismometer and a highly sensitive vertical displacement meter are utilized for the dynamic behavior and static behavior measurements, respectively.

From the measurement results, some knowledge can be concluded as presented in the following.

The first one is the changes in the peak frequency around 3 Hz. It become lower and lower, with the progress of this cut off construction. This sign occurred before the change of the static behavior.

The second is the changes in the Fourier spectrum values. Comparing the Fourier spectrum values of the stable rock block to that of the unstable rock block, it was found that the former one was larger than the latter one before the beginning of the construction. However, it became less than the latter one just before the collapse of rock block.

The third is the changes in the locus property of the rock block vibration. Before the beginning of the construction, the locus did not show any directionality, while just before the artificial collapse, it showed an apparent propensity to a certain direction.

These changes could be inferred as some signs of the collapse. If the mechanism of these phenomena could be clarified, it is feasible to predict the outbreak of the collapse of the rock blocks more accurately in the future.

Keywords: Dynamic; rock block; rock cliff.

THE DEFORMATION MECHANISM AND DYNAMIC STABILITY ON CREEPING SLOPE OF FUSHUN WEST OPEN CAST SIDE-SLOPES

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Fushun west open pit coal mine slope, located in Fushun city, P.R. China, is one of the most noted large-scale open pit coal mine slopes with problems related to the safety stability (Fig. 1). Characterization of the structural geometry of the slope creep deformation and reconstruction of its development history are believed to be pivotal in understanding what has happened and what will happen to the slope.

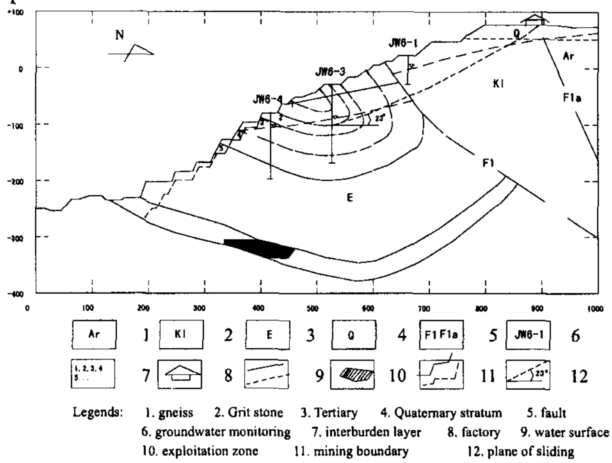

Fig. 1. Fushun West Strip Mine W600 hydrological geology profile chart.

Base on the Creep Test of soft sheaf, a mechanics creeping model was proposed (Quation (1)), and the deformation Mechanism of soft sheaf is discussed.

$$\tau = \frac{A_0}{(1+\delta \cdot t^{\alpha})}\gamma^m (1+\frac{\sigma_m}{H}) = c + \sigma_m \cdot tg\Phi \qquad (1)$$

where τ is the shear stress;A0-is the instantaneous shear modulus; t represent the time during shear stress; γ is the shear strain; σm is the normal stress; H is tensile strength; m is strain strengthen factor; c is cohesion; Φ represents the friction angle; δ and α is constants of experiments.

By analyzing the data of experiment result, the parameters of equation (1) can be gained. That is A0=13.60MPa, H=60kPa, m=0.440, δ=0.417, α=0.080.

According to time effect of long term strength, Fushun West Open-pit Coal Mine Slope is exemplified to study dynamic stability of side slope. The results show that the stability heightens greatly after reinforcement and dewatering(The analysis result is listed in Table 2.). Additionally, the method for predicting the instability of this kind slope is also discussed in this paper.

Table 2. Results of shearing creeping test and stable coefficient in the weak layer.

Time	t(year)	0.002 1991	1 1992	2 1993	4 1995	6 1997	8 1999	10 2001	15 2006
Shear	c (kPa)	17.03	10.82	10.24	9.70	9.40	9.10	9.03	8.74
index	Φ(°)	18.37	11.46	10.84	10.26	9.93	9.71	9.54	9.23
Stability	Highest water pressure in blood season	1.128	0.960	0.928	0.917	0.911	0.891	0.886	0.883
coefficie	Water pressure in non-blood season	1.348	1.088	1.051	1.034	1.024	1.008	1.004	0.999
	After the control engineering	1.542	1.322	1.287	1.272	1.262	1.245	1.242	1.237

Keywords: Deformation; dynamic; rock slope.

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12. OTHER APPLICATIONS

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EVALUATION OF THE EXCAVATION DAMAGE ZONE (EDZ) BY USING 3D LASER SCANNING TECHNIQUE

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A 3-D laser scanning technique is presented in this paper for the evaluation of the excavation damage zone (EDZ) in a tunnel. Compared to traditional methods, the 3-D laser scanner can quickly capture a great amount of digital 3-D information of an object with high resolution and good accuracy, and then perform different tasks related to tunnel documentation and analysis with the help of IT technology.

The scanning system used in this study can rotate 360 degrees in both horizontal and vertical directions with a scanning radius of up to 55 meters. Each scan takes just a few minutes, and covers up to hundreds of square meters with the optimal scanning resolution of about 3 mm. 4 parameters was recorded at each scanning point, i.e. 3-D reference co-ordinates (X, Y, Z) and reflex intensity. Therefore, objects in space can quickly be recorded in three dimensions and used to produce a 3-D digital model and an associated 3-D gray-scale image. By measuring a few reference points for each scan, the 3-D digital data of the scanned objects can be transformed into a geo-reference space, which enable us to locate any parts of a tunnel in real space.

In this case study, selected sections of a tunnel in crystalline rock at Äspö hard rock laboratory in Sweden was scanned by both systems. By using the laser scanning data, different features of EDZ are characterized by newly developed methods. For example, a differential model between the theoretical tunnel and the resulting blasted tunnel is used to evaluate the efficiency of the blasting. A 3-D digital model of the tunnel was generated for the documentation of the trace of the blasting holes which easily can be used to control the quality of the drilling of the blasting holes in three dimensions. In addition, fractures in different scales, from several meters to a few millimeters, are characterized by laser scanning data, and compared with the traditional methods for fracture analysis on tunnel surfaces. More detail results presented in this paper show a promising method for 3-D documentation of a tunnel, and might also provide a way to solve the well-known bottle-neck problem for site characterization and data acquisition in rock engineering.

Keywords: 3-D laser scanning; EDZ; tunnel documentation; fracture mapping.

NUMERICAL SIMULATION OF ICE-ROCK INTERFACE UNDER SHEAR LOADING

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Rock-ice fracture in cold regions is often an important problem for rock engineering design and construction. In order to assess the safety against failure of ice filled rock joints, a numerical method to study this problem is carried out in this paper, three models, i.e. ice-rock, rock-rock and rock-ice-rock specimen with known parameters has been used to simulate under static loading, at same time, the damage and fracture processes was modeled and anlysised using RFPA2D (Rock Failure Process Analysis) code. In this study, a newly proposed mechanical model is used to simulate the fracture behavior of ice-rock specimens under the shear loading. In this numerical model, rock and ice is regarded as two kind brittle material (different heterogeneity index) respect, an elastic finite element program is employed as the basic stress analysis tool while the elastic damage mechanics is used to describe the constitutive law of micro-level element. The maximum tensile strain criterion and Mohr–Coulomb criterion are utilized as damage thresholds. The heterogeneous stress field is obtained from numerical simulation, thus it is found that heterogeneity of mechanical properties has significant effect on the stress distribution in rock. The crack propagation processes simulated with this model shows good agreement with those of experimental observations in the pre-exist literature. It has been found that the shear fracture of rock-ice observed at the macroscopic level is predominantly caused by tensile damage at the microscopic level.

Keywords: Ice, Rock, Shear Strength, Numerical simulation; Elastic damage mechanics; Microscopic level.

IMPROVEMENT OF ROCK STRATA FOR FOUNDATION OF REACTOR BUILDINGS

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This paper presents improvement of rock-strata below the foundation of two reactor buildings for a 2 × 1000 MW Nuclear power plant by compaction grouting. The site is located on the East coast of India in Gulf of Mannar. Extensive confirmative geological, geophysical and geotechnical investigations were carried out at founding level after exposing the founding rock strata. Based on seismic refraction survey, three major and distinct layers of rock are identified below the founding depth of the reactor buildings: weathered rock of about 4.5m thickness, semi-weathered rock of average thickness 10m and hard rock occurring at a depth of about 15m below founding level. The geological mapping and borehole drilling were carried out which revealed that the founding stratum is basically a weathered granite hypersthenes with varying degree of weathering and occasional presence of calcareous material.

In addition to the routine laboratory tests on rock core samples, a number of field tests such as gamma logging, seismic cross-hole tests, Percolation test and cyclic plate load tests were carried out to assess the influence of calcareous material on the overall quality of the foundation rock stratum. Consolidation grouting has been carried out under reactor buildings upto 15m depth and surrounding area up to 10m depth. In the first stage, the grouting is carried out with pressure of 1 kg/cm^2 within 1 to 4m depth, in the second stage with pressure of 1.5 kg/cm^2 within depth of 4 to 7m and in the final stage with pressure of 2.5 kg/cm^2 within a depth of 7 to 15m below Reactor Building and 7 to 10m in surrounding area. The results of seismic cross-hole tests carried out before and after compaction grouting show considerable improvement of shear wave velocity of foundation rock stratum, which indicates the effectiveness of the consolidation grouting on improvement of quality and integrity of the rock strata to support the proposed reactor buildings. The estimated values of safe bearing capacity and settlement for the anticipated load intensity show the adequacy of the improved rock strata to support the proposed reactor buildings.

This paper describes detailed description of rock strata, method adopted for consolidation grouting, results of in-situ tests before and after improvement, and assessment of adequacy of the founding strata.

Keywords: Improvement of rock strata; Rock characterization; Reactor foundation; Consolidation grouting; Seismic cross-hole test; In-situ testing.

FEASIBILITY ANALYSIS OF PHYSICAL AND CHEMICAL SOFT ROCK MODIFICATIONS

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On the basis of the systematized study on the available study methods and existing problems in soft rock and soft rock engineering, and on the material composition, layer-crystal structure and swell-shrink mechanism of soft rock, a new thought to realize the soft rock engineering stability by using two kind of modification methods to improve the harmful properties of physic-chemical soft rock in put forwarded. And the feasibility of the methods is analyzed systematically.

(i) It is called electro-chemistry modification to use electro-chemistry method changes the layer-crystal structure and the cationic exchanging capability of the soft rock mineral. The electro-chemistry properties of the soft rock minerals and a series of surface reaction processes during modification are theoretically discussed. A test has showed that realizing the soft rock electro-chemistry modification is feasible.

(ii) It is called coated surface modification to coat the soft rock blocks or its grains by using a kind of hydrophobic material prevents the drying-wetting-mudding process in engineering surrounding rock. The surface structure, surface energy, surface functional group and surface area of the soft rock minerals are explained systemically. These offer a physical, chemical and mechanical basis of the mineralogy for the soft rock coated-surface modification. The hydrophobic property, coating firmness and physical-chemistry combination principle with the soft rock mineral and the cement grout of a kind of organic silicon compound modification agent with the structure of $YRSiX_3$ and a kind of naphthyl cement reinforcing agent have been studied. The interrelated tests have showed that the soft rock coated-surface modification is feasible.

Keywords: Physic-chemical soft rock; electro-chemistry; coated-surface; modification.

GIS SYSTEM DEVELOPMENT FOR SURFACE SUBSIDENCE PREDICTION DUE TO COMPLEX TABULAR EXTRACTIONS

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Surface subsidence due to underground mining produce various forms damage at the surface environment. In order to assess its damage accurately, subsidence characteristics should be known, not only the trough shape on final stage but also movements and deformation induced by extraction sequence are the most significant factor for protecting environment against subsidence damage. It is well known that damage to structural facilities is generally caused by differential movements, and horizontal compressive-tensile strains. It is not a simple problem of predicting the response of a structure to a particular value of subsidence.

Most of the prediction methods advanced so far for calculating surface subsidence resulting from underground mining. However, the movements and deformation characteristics of the ground surface induced by complex tabular extractions is a very complicated problem, which is influenced by panel geometries as well as dip directions, width to depth ratios, superimposed panel and extraction sequences. Moreover, to predict subsidence on the large extraction area, complex panel formation, and spatial-temporal analysis are main difficulties encountered in solving problems. It is indispensable to handle the complicated spatial geometry panel and distribute the subsidence parameters. In recent years, Geographic Information System (GIS) as a flexible platform can be used to develop applications, integrate spatial models and handle complex cases. In the case of subsidence due to complex underground extraction, GIS would be efficiently and effectively applicable the mining subsidence if its prediction system could be properly developed. Thus, the development of GIS based system need to be applied for predicting rigorous subsidence.

In this paper, a new prediction system has been established to calculate spatial-temporal subsidence at any surface point with any shape of extraction covering a wide range of mining geometry by combining stochastic model and GIS. Based on a three-dimensional (3D) GIS-polygon, a prediction method using GIS has been developed to handle subsidence prediction caused by complex tabular extractions. All the calculations are implemented by a computational program, where the components of GIS are used to fulfill the spatial-temporal analysis function. The verification of GIS based prediction model has been ascertained by comparison with SEH of empirical model and influence function model for the standard mining data. The established system is packaged as an extension tool in GIS, called *MSDAS-GIS* aimed to support the input and output interfaces directly accessing spatial data and attributes data. A case study for evaluating subsidence damage to water reservoir dams induced by complex tabular extractions is presented. As a result, the developed system can be expected as an effective and efficient method for environmental protection against subsidence damage.

Keywords: Underground coal mining; Mining damage; Spatial-temporal subsidence prediction; GIS.

APPLICATION OF THE DESIGN PARAMETERS FROM STATISTICAL ANALYSIS

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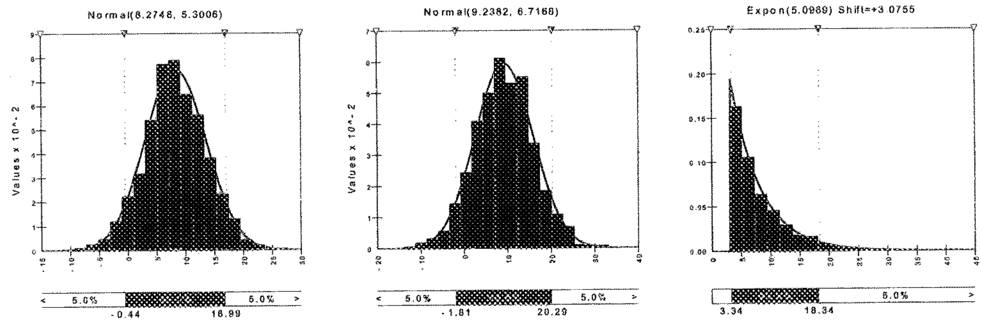
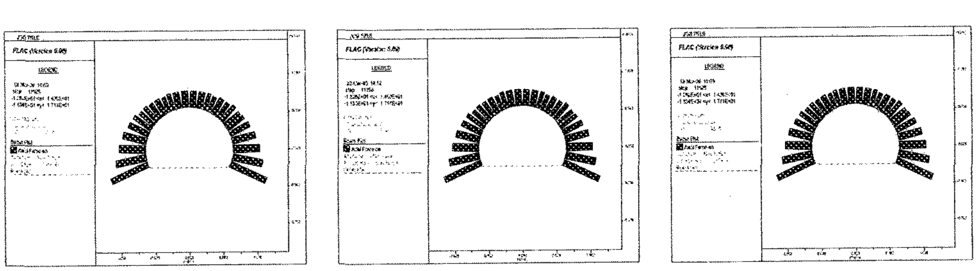
Input parameters estimated from the results of the statistical processing using samples with errors lower the confidence in an analysis. In spite of the biased characteristics of population caused by errors of the samples in statistical processing, especially the statistical processing in case of the input parameters having well known distribution characteristics is used for the method of estimating the input parameters. In this study, statistics processing is executed with considering the character of the distribution. As a result, considering the characteristics of the distribution is more reasonable in estimating input parameters for numerical analysis.

Keywords: Statistical processing; numerical analysis.



(a) Case Ⅰ (b) Case Ⅱ (c) Case Ⅲ

Fig. 1. Monte Carlo simulation.



(a) Case Ⅰ (b) Case Ⅱ (c) Case Ⅲ

Fig. 2. Beam axial force.

STABILITY ANALYSIS OF SPREAD-FOOTING FOUNDATIONS ON WEAK ROCK USING NON-LINEAR FEM MODELLING

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Analysis of stability (mainly bearing strength and settlement) under a footing on regularly bedded, jointed and layered model rock mass is conducted using 3D Non-linear FEM analysis. The inputs required for the FEM modeling were imported from the laboratory results of the measurements. Rock mass were modeled as elastic-plastic with Drucker- Prager failure criteria for plane strain condition. The joints and the weak layers in the floor rock mass were modeled in order to enable separation of zones in the models. The footing settlements correspond to the maximum applied bearing pressure on floor strata (for different sizes and shapes of footing plates and also under varying anisotropy conditions of floor strata) as obtained from the experimental results and FEM investigations, were compared and the maximum deviation was observed as 14 % whereas the minimum was even less than 3 %. From FEM analysis it is interpreted that maximum stress concentration occurs at the tip of the footing plate all along the boundary. The stress concentration extends maximum to a distance 2 to 3 times footing plate width in all direction of floor strata. It is further interpreted that maximum vertical settlement occurs near to the vicinity of the footing plate. From the FEM analysis of jointed rock it is examined that there is a slight movement of the two blocks separated by the joint. The joint plane acts as the rupture plane.

Keywords: Stability; foundation; FEM.

ROCK DAMAGE ZONE ANALYSIS USING BACK-CALCULATED CRITICAL STRAIN

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The evaluation of rock damage zone is gaining importance as increasing massive underground constructions such as underground oil tank, nuclear wastes storages and so on. In this study, the rock damage zone under blasting load was analyzed based on the continuum damage model proposed by Liu and Katsabanis (1997) using the measured strain at surrounding rock.

The yield strength of a material under a dynamic load such as a blasting pressure is generally different from that under a static load, since a dynamic load larger than the static yield strength of the material may not lead to the yield or failure of the material. Therefore, the behavior of a material under dynamic load has to be investigated under the effect of loading rate which can consider both the magnitude and the duration of the dynamic load simultaneously.

To evaluate critical strain of the rock, field tests and measurements were performed in Jangmok-ri, Geoje-si, in South Korea. Strain gages were used to measure deformations in rock under blasting pressure and the critical strain was back-calculated using the measured maximum tensile strain value among strain gages.

A user-defined subroutine was coded to analyze rock damage around blasting hole. To estimate the damaged zone during blasting based on back-calculated critical strain, numerical analyses using the user-defined subroutine combined with ABAQUS/Explicit were performed.

Using the proposed method, followings were obtained in this study;

1. The material property of critical strain is the one of most important factors to evaluate rock damage zone.
2. The critical strain can be back-calculated using measured strains with time during blasting.
3. The field excavation designs can be optimized to minimize the rock damage zone using the proposed method.

Keywords: Damage Zone Analysis; Blasting; Critical Strain.

PARAMETER IDENTIFICATION AND PREDICTION OF SUBSIDENCE USING ARTIFICIAL NEURAL NETWORKS AND FEM DATABASE

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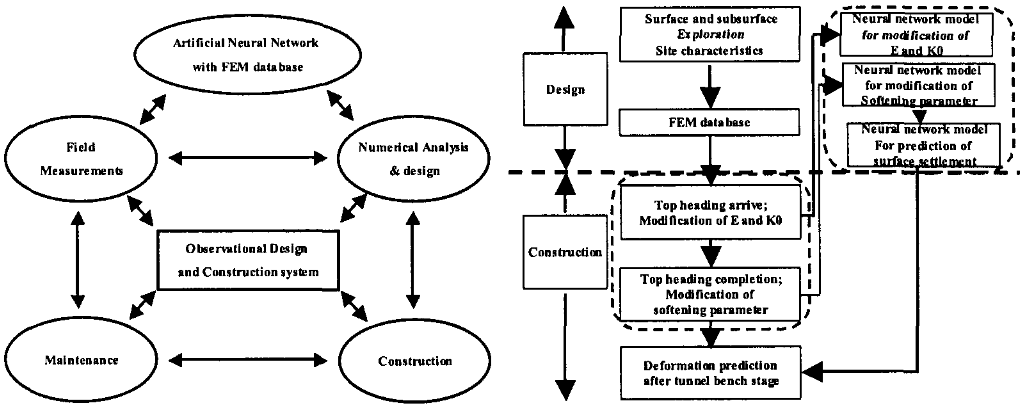
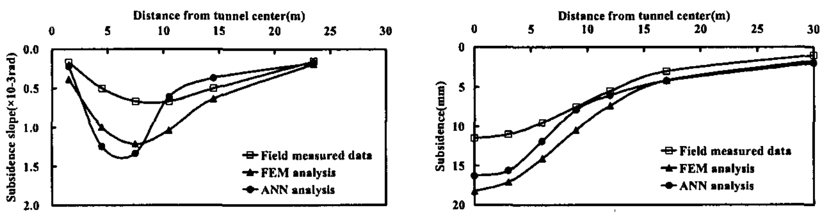
In tunnel design/construction, deformation behavior is evaluated by numerical analysis. It is important to identify material properties and adjust ground behavior modeling for a good predation in urban tunnel design. Model parameters for FE analyses are identified such at the results of these calculations agree with the available measurement data as well as possible. In this paper, we discussed the parameter identification and subsidence prediction using artificial neural network (ANN) and FEM database (Fig. 1). This paper has developed two ANN models, which one is optimal ANN model for parameter identification and another is subsidence prediction. Based on the identified material parameter on construction, the ground movements have predicted using FEM analysis and ANN ones (Fig. 2). This paper performed a case study to verify the application to the actual NATM tunnel in parameter identification and prediction problem. The proposed method offers a practical way for predicting final displacement of shallow NATM tunnel at earlier stages of construction, enabling rational safety management scheme to be employed.

Keywords: Artificial Neural Network; FEM database; Parameter Identification; Prediction.



(a) Scheme of neuru-numerics-measure (b) ANN application in NATM tunnel design/construction

Fig. 1. ANN application with FEM database in NATM tunnel design/construction .



(a) Surface subsidence (b) Subsidence slope

Fig. 2. Comparison between ANN results, FEM ones and measured ones.

EXPERIMENTAL RESEARCH ON PENDULUM IMPACT PROPERTIES OF FROZEN CLAY

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The frozen clay of the certain frozen shaft is regarded as the experimental soil sample. Impact experiments of frozen clay are made in order to measure the impact toughness in the different temperatures and the different impact energy. Through the test results, it is seen that the impact toughness of frozen clay will increase with the temperature decreasing and the impact energy increasing. At the different temperature, the impact toughness that its impact height is 1.36m increases 2000J/m^2 or so than that of impact height is 0.9m.The uneven extent of sample may increase with the temperature decreasing, and the range of unevenness in the plane is $\pm$3mm or so. The results indicate the destruction of frozen clay belongs to the brittle fracture.

Keywords: Impact toughness; Frozen clay; Temperature; Experiment.

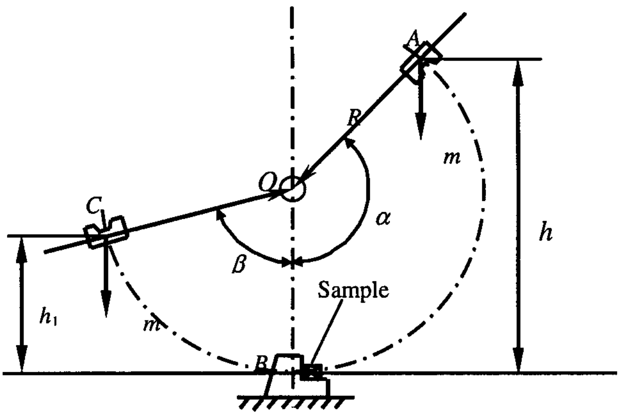

Fig. 1. The principle of impact.

Table 1. Test result of impact toughness experiment.

Number	Temperature, °C	impact toughness, J/m^2 (1.36m)	impact toughness, J/m^2 (0.9m)
1	-3	8482.1	5813.4
2	-5	8554.5	6012.7
3	-7	8631.9	6054.1
4	-10	8791.2	6134.5
5	-12	8919.9	6264.1
6	-15	9048.2	6334.2
7	-17	9152.0	6748.3

EXPERIMENTAL STUDY ON THE MECHANICAL BEHAVIOR OF THE RIB ARCH STRUCTURE

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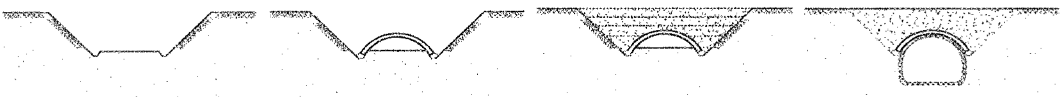
A new semi cut-and-cover method was developed to excavate the shallow overburden tunnel. This method uses the rib arch structure that consists of H-beams and braced walls. This rib arch structure is constructed above the tunnel section before excavation of the main tunnel. To verify mechanical behavior of the proposed rib arch structure, the large-scale model tests were performed to reduce the scale effect. In this paper, the developed method and some experimental results of the behavior of the rib arch structure and the backfill ground are introduced. The soil loads on the rib arch structure, displacement of H-beam and braced wall, earth pressure are monitored to investigate the stability of the method. From the results, it can be concluded that the newly developed semi cut-and-cover method using the rib arch structure, BRAM (Braced Rib Arch Method), is effective for the excavation of the shallow overburden tunnel with guaranteed the stability of its structure.

Keywords: H-beam; Braced wall; Shallow overburden tunnel; Rib arch structure; Large-scale model test.



(a) (b) (c) (d)

Fig. 1. Construction sequences of the suggested method.

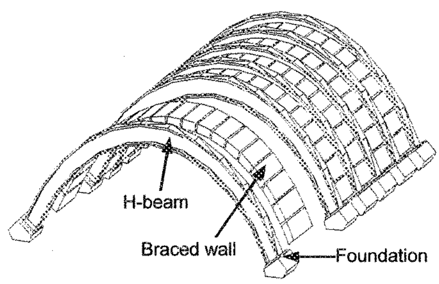


Fig. 2. Sketch of the rib arch structure.

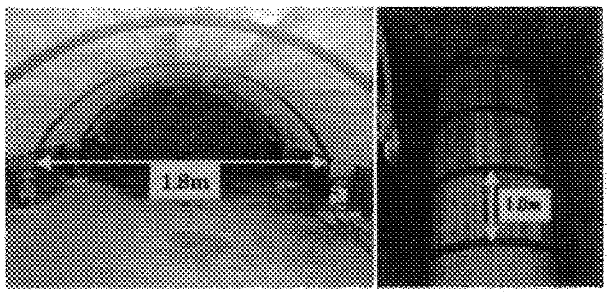
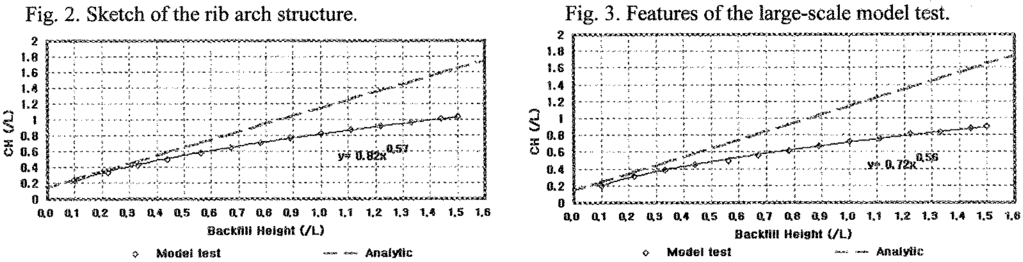


Fig. 3. Features of the large-scale model test.



(a) Slope angle: 60° (b) Slope angle: 90°

Fig. 4. The conversion height with backfill height.

EFFECTS OF COMPOSITION AND MICROSTRUCTURES ON ELASTIC STRAIN ENERGY IN CLASTIC ROCK

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Composition and microstructures of rock are the important parameters to influence its strength and elastic strain energy. In experimental laboratory, some clastic rocks including argillaceous sandstone, siltstone, fine sandstone and medium & coarse-grain sandstone are tested to study rock strength and elastic strain energy index. Correspondingly, composition and microstructures of each rock specimen are analyzed in thin sections to determine mineral composition, content, grain size, cement and contact modes between grains. From the experimental researches, it is shown that the elastic strain energy and strength of rock have an intimate relationship with its composition and microstructures. With the increasing of percentage composition and average size of major detritus grains, uniaxial compressive strength, Young's modulus and elastic strain energy index are all increasing. The elastic strain energy index and rock strength are affected by the microstructures such as average size of major detritus grain (quartz, feldspar and rock debris), coment and contact modes between grains. Values of uniaixal compressive strength, Young's modulus and elastic strain energy index are increasing with the growing of average grain size of major detritus according to

$$Y = aX^4 + bX^3 + cX^2 + dX + e, \tag{1}$$

where Y is uniaixal compressive strength, Young's modulus or elastic strain energy index, X is the average size of main detritus grains, $a>0$, $b<0$, $c>0$, $d<0$ and $e>0$. Especially, when the average grain size of major detritus is about 0.3mm (medium-grain sandstone), the elastic strain energy index and the rock strength come to the peak value. Generally, sandstones with siliceous agglutinated component have higher strength than those with argillaceous cements, and sandstones with line-contact between grains have stronger mechanics attributes than those with point-contact or no-touch.

Keywords: Clastic rock; Composition; Microstructures; Elastic strain energy index; Burst potential; Dissipated energy.

STRESS VARIABILITY AROUND LARGE STRUCTURAL FEATURES AND ITS IMPACT ON PERMEABILITY FOR COUPLED MODELLING SIMULATIONS

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A clear understanding of rock stress and its effect on permeability is important in a coupled simulation where fluid production causes a significant increase in the effective stress within a reservoir. Changing the in-situ rock stress state can alter in reservoir properties. As a result, porosity and permeability could be affected due to the rearrangement of rock particles and the redistribution of sensitive pore structures.

It is known that permeability is more sensitive to stress changes than porosity. Permeability reduction can range between 10% and 30% as reported in previous publications, this is within the elastic range of the material under investigation. Once the material reaches its yield strength a dramatic increase in permeability can then occur or further reduction depending on the mode of failure of the rock type in question. This study reports on permeability reduction under increasing stress (within the elastic range) for a number of rock types, the samples tested are from the Cooper Basin of South Australia; the standard Berea Sandstone is included for validation purposes. The results are used within a newly developed finite element coupled code in order to estimate permeability sensitivity to stress changes for predicting compaction and subsidence effects for a cylindrical wellbore model.

Furthermore, understanding rock mass stress away from the borehole is a major obstacle in the exploration and development of hydrocarbons. It is standard practice in the petroleum industry to use drilling data to determine the orientation and estimate the magnitudes of principal stresses at depth. However, field observations indicate that the orientation of the principal stresses is often locally perturbed by and around discontinuities, such as faults or formation boundaries (Kattenhorn et al., 2000; Maerten et al., 2002). Numerical stress methods have been successfully employed to model the effect of displacing faults on the surrounding rock mass. 3D distinct element code has been used to show how displacing faults generate stress variation in 3D about a fault plane (Camac et al., 2004), verified with field observations. In this work consideration is made of this variability and its effect on wellbore subsidence and compaction models. A series of models were run which incorporate the stress variability expected around an example fault under normal stress field conditions. The models show that the initial stress state conditions associated with a fault give rise to a variation in the stress path during reservoir production and resultant permeability changes are measured. The extent of the influence of lateral changes around large-scale structural features is thereby assessed and the work demonstrates the importance of incorporating this initial stress variability for production purposes.

Keywords: Stress sensitive permeability, fault, subsidence.

APPLICATION OF ROADHEADER IN HIGH STRENGTH ROCK FORMATIONS

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Roadheaders are one of the main cutting machines for rapid and efficient tunnel and roadway drivages. On the other hand, the applications of these machines are limited to the geology and geotechnical properties of rock formations.

The main objective of this paper is to investigate the performance of a roadheader in high strength rock formations in Küçüksu sewerage tunnel constructed by STFA for Istanbul Water and Sewerage Administration (ISKI). The project consists of a sewage plant having a capacity of 291702 m^3/day, three shafts and two tunnels having final diameters of 2.2m and length of 95.8 and 1037.2 m. The tunnels were excavated using SM1 model shielded Herrenknecht roadheader. The machine had a cutting power of 90 kW and total power of 224 kW. The cutting head is axial type having 36 conical cutters of 75° tip angle. Küçüksu sewerage tunnel driven from Küçüksu to Hekimbaşı is located in the Anatolian part of Istanbul . The tunnel route passes throughout Büyükada formation which consists of limestone, siltstone and andezite dykes.

Firstly, the geomechanical properties of intact rock (UCS, petrography) and rock mass properties (RQD, water income) of the tunnel zones are found out, then, the in-situ performance of roadheader are recorded in order to find the relationship between the performance of the roadheader and geotechnical properties of rock.

Secondly, during the field studies, analysis of the breakdowns and cutter wear types are determined in detail.

This paper gives special attention for roadheader usage in high strength rock formations and may be summarized as; that intact rock compressive strength (UCS) is well correlated with net cutting rate of the roadheader used in Küçüksu sewerage tunnel.

Keywords: Roadheader; High strength rock; Machine stoppages; Cutter wear rate.

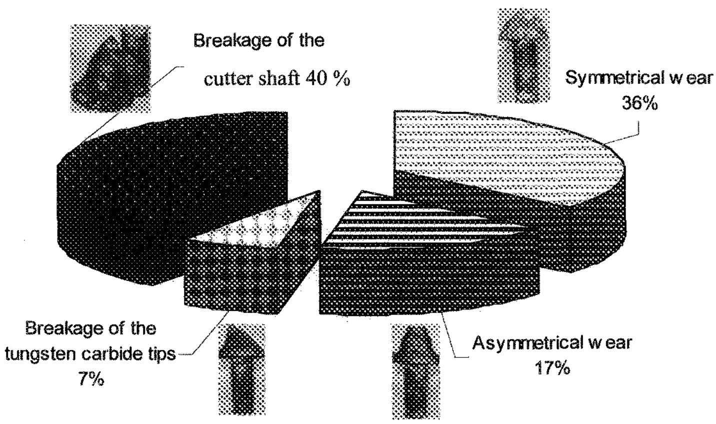

Fig. 1. Classification of cutter wear in high strength rock formation, Küçüksu.

THE MEASUREMENT OF PRESENT CRUSTAL STRESS IN LIJIN OIL FIELD AND ITS APPLICATION

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It has great meaning to make clear the crustal stress status in oil-field exploration and development. The crustal stress of LiJin oil field was measured systematically by several methods in this paper. The methods include anisotropy of acoustic velocity method, wellbore breakouts method and working site monitoring method. The predominant direction of horizontal maximal stress σ_{hmax} is between NE10°-NE45°.

Based on the characteristics of reservoir at spot, more effective fracture network is needed to improve the supply of water. Key problem is to select the candidate of hydraulic fracturing well. Generally speaking, when the difference of horizontal stress is large than 7MPa, the direction of fracturing fractures is parallel to the direction of the σ_{hmax}. The fracturing fractures are vertical if the vertical stress σ_v is the maximum stress.

According to the state of stress field and condition of development, some principles should be carefully considered to select fracturing candidate. Firstly, in order to obtain more sufficient supply of water, the candidate should be near the injection well. Secondly, it is preferred to select the well that the line linked the candidate well and the injection well should be parallel to the direction of σ_{hmax} approximately

Recently, well Li853-23 has been hydraulic fractured. The line linked the well Li853-23 and the injection well is parallel to the direction of σ_{hmax} with water well Li853 almost. The detail information is showed Figure 1. The hydraulic fracture of Li853-23 should be oriented about NE 40° controlled by crustal stress. The result of hydraulic fracturing is that the well naturally flows for 23 days. The initial oil output is 17.5ton/day. The stable production is 12ton/day that is approximate two times bigger than the average level of the whole region. The production has lasted 8 months up to now.

Keywords: Crustal stress measurement; hydraulic fracturing; Lijin oil field; selecting well.

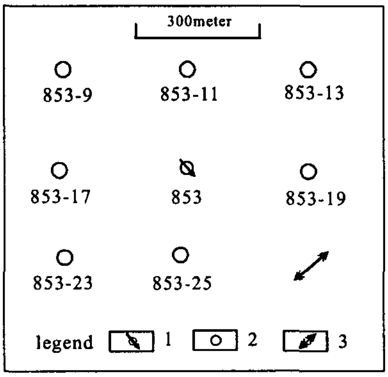

Fig. 1. Position graph of oil and water well
1-water well, 2-oil well, 3-orientation of σ_{Hmax}.

STUDY ON THE BACK ANALYSIS OF MULTI-PARAMETERS

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In geotechnical engineering, back analysis is usually carried out on the basis of the measured displacements during the project construction. By means of the monitoring data including displacements, geomechanical parameters of the rock mass are calculated. Thereby, the stability of the underground works can be reevaluated reasonably. Owing to the complexity of the geotechnical materials, the back analysis for multi-parameters is necessary.

This paper presents the direct formulation method, which is the general one of the back analysis methods. The typical nonlinear least square equation for back analysis can be written in the following form,

$$\min f(x) = (u(x) - \bar{u})^T (u(x) - \bar{u}) \tag{1}$$

where $\bar{u}$ is the measured displacement vector caused by excavation and u(x) is the corresponding displacement vector calculated by numerical method. The geomechanical parameters are denoted by vector, x. The function u(x) is nonlinear and min *f(x)* is the nonlinear least square problem.

Finite element method (FEM) is utilized to perform the normal numerical analysis. Gauss-Newton method and quasi-Newton method are employed to solve the optimization problem by iteration. A finite element based back analysis code is developed. In the code, the above optimization methods are implemented, elasto-plastic constitutive relation is included and the excavation process in geomechanics is simulated.

Two numerical examples are shown and part of the results is presented in Table 1 where three cases are studied. Firstly, cohesion c_1 and c_2 for two materials are back calculated. Then elastic modulus, cohesion and the angle of internal friction are obtained separately by the presented back analysis method.

Keywords: Back analysis, Gauss-Newton method, quasi-Newton method, FEM.

Table 1. Material parameters identified by the presented technique.

material parameters	Initial value-1	Converged value-1*	Initial value-2	Converged value-2**	Initial value-3	Converged value-3***
E_1(GPa)			5	20.0(20)		
c_1(MPa)	0.6	1.51(1.5)[1]	0.6	1.5(1.5)		
φ_1 (degree)			40	50.6 (50.3)		
E_2(GPa)					1	3.67(3.75)
c_2(MPa)	0.1	0.38(0.4)			0.1	0.24(0.40)
φ_2 (degree)					30	46.3 (41.5)

*Iteration number is 11; **Iteration number is 7; ***Iteration number is 12. [1]The value in parentheses is the corresponding true value

CRUSTAL STRESSES IN RYUKYU ISLANDS OF JAPAN

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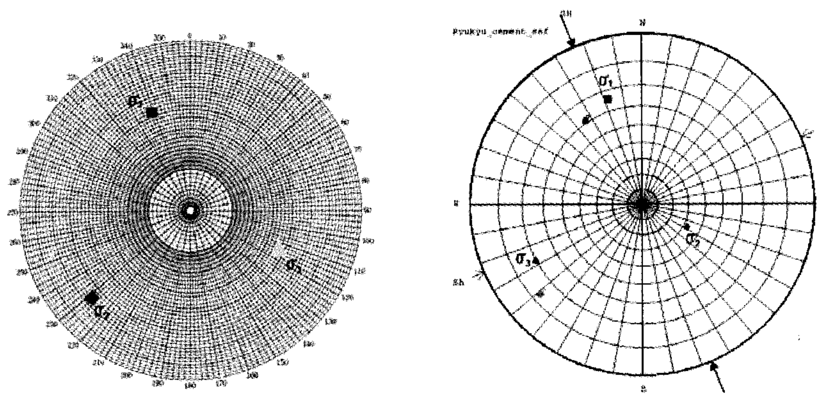
There is almost no in-situ stress measurements in Ryukyu islands in contrast to other regions of Japan. In this study, an attempt was made to infer the crustal stresses in Ryukyu Islands from AE method, the striations of faults and focal plane solutions. The AE method is the first application to this part of Japan. The fault striations measurements were mainly carried out on the main island and some data were gathered from some publications available in literature. The focal plane solutions were gathered from renowned institutes in Japan and USA. The results obtained from the AE method are compared with the striation method as well as with the in-situ stress measurements at the site of Okinawa Sea-water Pumped-Storage Power Plant. The comparisons clearly indicated that the results are quite similar to each other in both magnitudes and orientations. The maximum horizontal stress generally acts perpendicular to the arc axis although some of inferred maximum horizontal stress directions are parallel to the arc axis. This fact may arise from the choice of the causative fault plane. The compressive horizontal stresses are high in the vicinity of the Ryukyu trench while its magnitude decreases along Okinawa trough. Although the results of stress inferences by indirect methods are very promising, it is desirable to increase actual stress measurements for better understanding of the crustal stresses in Ryukyu Islands.

Keywords: Rock stress; focal plane solution; striation method.



(a) AE Method

(b) Striation method

Fig. 1. Comparison of stress state obtained from AE and striation methods.

RESEARCH OF THE SEEPAGE LAW OF ADSORPTIVE GAS IN SINGLE COAL FRACTURE

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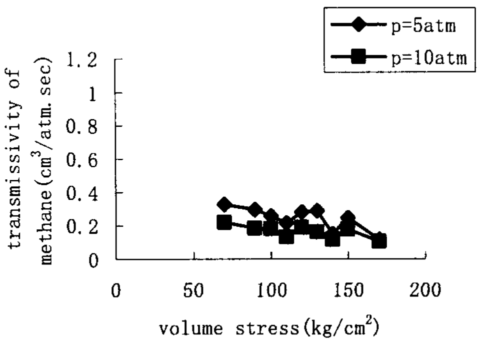
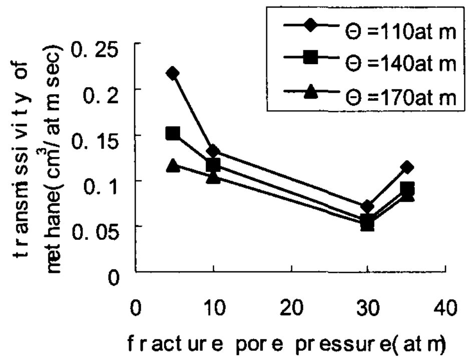
Coal bed methane is a crucial factor in safe coal mining. The methane reserved in coal has two types: free and adsorption. In this paper, the experiment of methane seepage law in single coal fracture is discussed. The hydraulic conductivity of fracture is related to the volume stress and pore pressure. The pore pressure has a critical value on the influence of seepage. Under the critical value, the hydraulic conductivity is decreasing with the increase of pore pressure; above the critical value, the hydraulic conductivity is increasing with the increase of pore pressure. This can be explained by figure 1. At the end of the paper, a experimental formula of hydraulic conductivity (equation 1) which has considered the effect of both adsorption and volume stress is put forward. This research will help in treating the coal bed methane engineering.

Keywords: Seepage law; Adsorption; Hydraulic conductivity.

$$K_f = K_{f0} p^{-\eta} \exp\left\{ -b\left[\frac{\sigma_1 - \beta p}{K_n}\right] - c\left[\frac{1-\nu_r}{E_r}(\sigma_2 + \sigma_3) - \frac{2\nu_r}{E_r}\sigma_1\right]\right\} \tag{1}$$



(a) transmissivity of methane vs. volume stress



(b) transmissivity of methane vs. fracture pore pressure

Fig. 1. The relationship of gas transmissivity and volume stress/pore pressure.

METHOD OF SUSCEPTIVITY ANALYSIS OF PARAMETERS AND ENGINEERING APPLICATION

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During the process of numerical simulation, it is important for us to input the parameters accurately. It will influence the calculated result whether available or not. There is a saying what is called "input the gold, output the gold; input the waste, output the waste". For the rock material consisted in the underground station groups, it is complicated medium and there is plenty of unpredictable factors in it. Its parameters are discrete. Therefore, how to reduce the error of parameters is the key problem for improving the reliability of stability analysis result. Using the susceptivity analysis method, the susceptibility function and factor were defined in a dimensionless form.

$$S_k(\alpha_k) = \left| \frac{d\varphi_{k(\alpha_k)}}{d\alpha_k} \right| \frac{\alpha_k}{Q} \qquad k = 1,2,\ldots\ldots n$$

where $Q = f(\alpha_1, \alpha_2, \cdots \alpha_n)$ was the system performance to be analyzed. $\alpha_k (k = 1,2,\ldots\ldots n)$ was the parameter that affected the system performance. $\varphi_{k(\alpha_k)} = f(\alpha_1^*, \cdots, \alpha_{k-1}^*, \alpha_k, \alpha_{k+1}^*, \ldots\ldots, \alpha_n^*)$ was the system performance when other parameters were constants and α_k was changed in the possible range.

On the base of Langyashan pumped storage station project, several operating modes were computed in the possible range of influential factors that would influence the stability of underground caverns. Deformation modulus, Poisson ratio, cohesion, internal friction angle and horizontal stress were mainly concerned. The sensibility to the influential factors of side wall displacement and plastic zone was respectively discussed with the forementioned susceptivity analysis method. Then the sequence of the influential factors was obtained. The most sensitive factor of the side-wall displacement was the lateral pressure coefficient. The deformation modulus reflected the stiffness of rock, it was in the secondary place and the internal friction angle and cohesion was in the next place. Poisson ratio was the worst sensitive of all. The most sensitive factor of the side wall plastic zone was the internal friction angle; the deformation modulus was the worst sensitive factor of all. The conclusion was useful to the choice of parameters when the stability of underground caverns would be analyzed.

Keywords: Susceptivity; susceptibility factor; mechanical parameters; stability of caverns.

STRESS AND PRESSURE CHANGES ANALYZED WITH A FULLY COUPLED RESERVOIR MODEL

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A coupled stress-flow model has been developed using a hybrid numerical model scheme to apply to large petroleum geomechanics problems. The major characteristics of the model are the following: first, it is a fully coupled model, based either on the simultaneous solution method or the iteratively coupled method; and second, it accounts for an infinite or a semi-infinite domain using the displacement discontinuity method to introduce the far-field strata to the reservoir simulation model. The program has been verified and validated, and several applications are shown in this paper.

Keywords: Finite element model, petroleum geomechanics, coupled flow-stress, displacement discontinuity.

NUMERICAL METHOD FOR MIXED FAILURE OF ROCKLIKE MATERIALS BASED ON VIRTUAL MULTI-DIMENSIONAL INTERNAL BONDS

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In VMIB (Virtual Multi-dimensional Internal Bonds) model, solid materials are considered to consist of randomized mass particles in microscopic. These mass particles are connected with virtual normal and shear bond or with normal bond possessing both normal and shear stiffness. The bond will evolve with itself deformation. The bond evolution law in microscopic determines the macro response of material. In a deformed solid material, the virtual bonds deformation are different, say some being stretched and some being compressed. The nonlinear behaviors of material in macroscopic result from the bond stiffness evolution induced by itself deformation in microscopic. In the tensile and compressive cases, the evolution laws of bond stiffness are different. When bond is stretched, both the normal and shear stiffness are dominated by themselves deformation. However, when bond is compressed, the normal stiffness is subjected to both its rotation angle and deformation while the shear stiffness is only subjected to the rotation angle. To reflect this micro mechanism, the presented paper provides a mixed evolution function of bond stiffness, which makes the VMIB model applicable to the mixed failure behaviors of materials. Finally, the presented method is used to simulate the failure process of a beam and the results agree with the observation.

Keywords: Numerical simulation; Rocklike material; Virtual Multi-dimensional Internal Bonds (VMIB).

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AUTHOR INDEX

Rock Mechanics in Underground Construction

This proceeding contains over 300 contributions in rock mechanics and engineering presented in the 4th Asian Rock Mechanics Symposium in Singapore in November 2006 with presenters from all over Asia and many other parts of the world. There are 7 keynote papers summarizing the state of the art of various rock engineering topics including underground rock caverns. The technical papers cover a wide range of rock mechanics and engineering topics including rock tunnels, caverns, mining, rock slopes and dams, rock blasting, rock burst and failure, rock properties, rock mass, rock joints, and block theory. A large number of valuable rock engineering case studies are also reported.

This volume should serve as a useful reference for the engineers and researchers in rock mechanics and rock engineering.

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