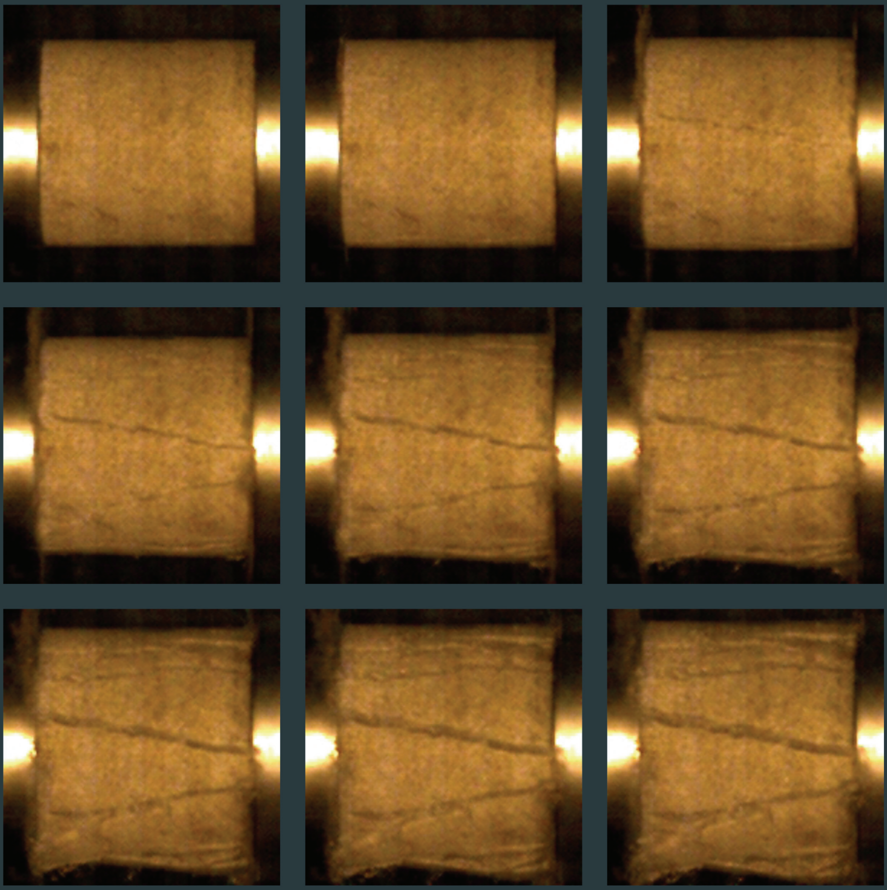

# Advances in Rock Dynamics and Applications

Editors: Yingxin Zhou & Jian Zhao

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A BALKEMA BOOK

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# Preface

Dynamics has been an important part of mechanics in various disciplines. Rock dynamics, too, is an important part of rock mechanics, where an increased rate of loading induces a change in the mechanical behaviour of the rock materials and rock masses.

The study of rock dynamics is important because many rock mechanics and rock engineering problems involve dynamic loading ranging from earthquakes to vibrations to explosions, and rock failure under those dynamic loads as well as dynamic failure under static loads. However, due to the additional "4th" dimension of time, dynamics has been a more challenging topic to understand and to apply. It remains, at least in the discipline of rock mechanics, a relatively uncultivated territory, where research and knowledge are limited.

In 2008, the Commission on Rock Dynamics was set up within the International Society for Rock Mechanics (ISRM). One of the aims of the Commission is to share and exchange knowledge in rock dynamics research and to produce documents on the study and engineering applications of rock dynamics.

In the summer of 2009, the ISRM Commission on Rock Dynamics organised its first workshop in Lausanne, Switzerland. It was at this workshop that participants felt that there was a lack of a comprehensive knowledge base and the Commission should organise researchers to prepare a document summarising the state-of-the-art. This edited book is a direct result of that discussion.

The book aims to provide a summary of the current knowledge of rock dynamics for researchers and engineers. It consists of 18 chapters contributed by individual authors. The topics chosen are wide-ranging, covering fundamental theories of fracture dynamics and wave propagation, rock dynamic properties and testing methods, numerical modelling of rock dynamic failure, engineering applications in earthquakes, explosion loading and tunnel response, as well as dynamic rock support.

The editors would like to thank all the contributing authors. The editing effort by Ms Haiying Bian is greatly appreciated. The CRC Press team, particularly Mr Janjaap Blom and Mr Richard Gundel, also provided publishing support.

Yingxin Zhou and Jian Zhao
March 2011

Chapter 1

# Introduction

*Yingxin Zhou and Jian Zhao*

## 1.1 SCOPE OF ROCK DYNAMICS

Rock dynamics, as a branch of rock mechanics, deals with the responses of rock (materials and masses) under dynamic stress fields, where an increased rate of loading (or impulsive loading) induces a change in the mechanical behaviour of the rock materials and rock masses. Figure 1.1 is an example showing the different failure behaviours for a rock material under static and dynamic loads.

Differing from static mechanics, dynamic stresses are in the forms of stress waves propagating in the loaded medium with time, and therefore the response of rock is influenced by, and interacts with, the stresses in motion. Rock dynamics deals not only with the end effects of the forces, but also the processes of the forces acting on the rocks. In these processes, both forces and objects are in motion. Rock dynamics specifically examines the processes of dynamic motions of both the forces and the rocks, at different scales varying from micro particles to rock blocks.

Rock dynamics as a science subject covers a wide scope related to forces, and responses of rock, in the time domain. It deals with the distribution of stress fields,

*Figure 1.1* Rock specimens after failure under static (left) and dynamic (right) loads.

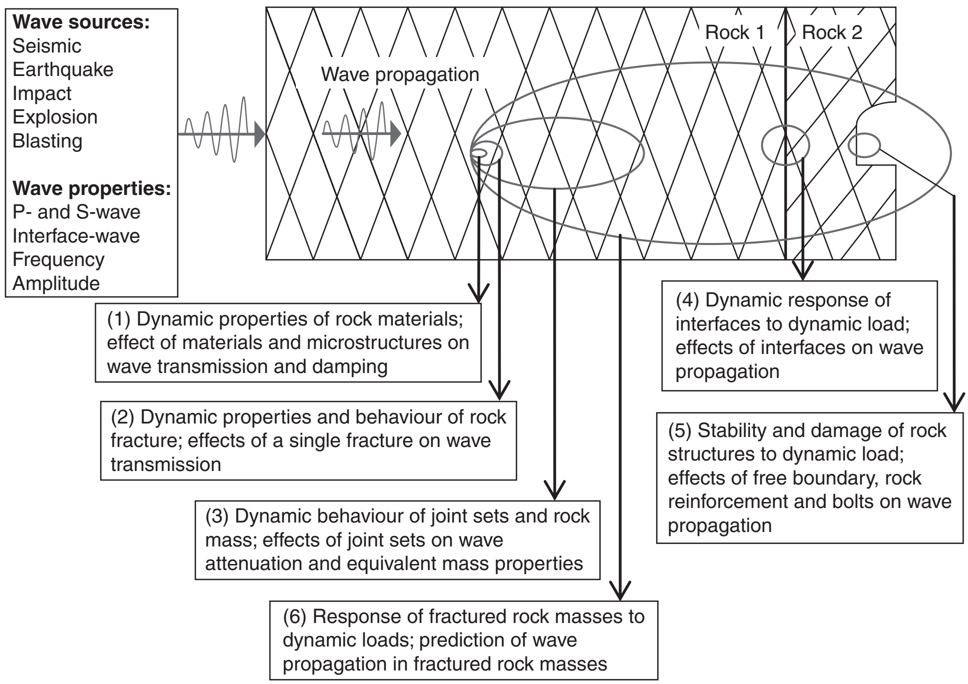


*Figure 1.2* Typical rock dynamic problems in tunnels and caverns (after Zhao *et al.*, 1999).

responses and properties of rocks, and dynamic behaviour coupled with the physical environment.

Sources of dynamic loads include explosion, impact, and seismic events. These loads are typically given in the form of time histories of particle acceleration, velocity, or displacement.

The distribution of a dynamic stress field is in the form of a stress wave moving in the loaded medium, including the propagation behaviour of the stress wave. Stress wave propagation in rock masses is governed by wave transmission and transformation across the discontinuities (rock joints) in the rock masses.

The response of rock materials and rock masses under dynamic stress field includes displacements of rock at particle scale, material fracturing and failure, and large movements at discontinuities. Rock fracturing, for example, is a dynamic micro-scale process leading to macro-scale deformation and failure.

Rock dynamic behaviour is often coupled with, and frequently induced by, the physical environment, e.g. water and temperature. Changes of physical environment may alter the stress fields as well as the properties of the rock materials and rock masses, hence leading to dynamic responses of the rocks.

Rock dynamics has applications in mining, energy, environmental and civil engineering, when dynamic loads and behaviours are encountered. Figure 1.2 illustrates typical rock dynamics issues related to the construction and utilisation of a storage cavern. Some of the applications are summarised in, but not limited to, the items below.

a) Construction: rock excavation and fragmentation by blasting and by mechanical means, stability of rock mass and rock support under various dynamic loads, protection of rock falls, use of seismic waves for ground exploration;
b) Energy and mining: rock burst and support in deep mines, fracturing of hot rock in geothermal fields, effects of water injection and induced seismic events; and
c) Environment: earthquake effects on slopes and landslides, hazard and risk control due to explosion and blast, effects of blasting vibrations on existing structures, seismic damage to structures in and on rocks.

## 1.2 ISRM COMMISSION ON ROCK DYNAMICS

Understanding the effects of dynamic loading on rock and built structures (e.g. tunnels and caverns with their associated reinforcement and support) is essential in dealing with the various rock dynamics problems such as dynamic support design and safety assessment. However, guidance and standards in dynamic analysis and design are generally lacking, and much of the research work done on rock dynamics for military purposes has not been easily available for the general public. For example, there are no existing standard methods for rock dynamic testing, and in rock engineering practice, guidelines and design methodologies for dealing with dynamic problems are generally lacking.

It was against this background that the International Society for Rock Mechanics (ISRM) established a Commission on Rock Dynamics in January 2008. The aim of the Commission (ISRM 2010) is to:

i) Provide a forum for the sharing and exchange of knowledge in rock dynamics research and engineering applications, including organising commission meetings, workshops, seminars and short courses;
ii) Co-ordinate rock dynamic research activities within the ISRM community as well as with other research and professional organizations; and
iii) Produce reports and guidelines on the study and engineering applications of rock dynamics covering fundamental theories, dynamic properties of rock and rock mass, testing methods, tunnel response, and support design.

Specifically, the Commission's work scope (ISRM 2010) covers:

i) Characterisation of dynamic loading sources,
ii) Rock dynamic properties and their determination,
iii) Propagation of dynamic stress waves in geological media,
iv) Rock damage criteria and damage assessment, and
v) Dynamic rock support design.

Under the work plan of the Commission, and resulting from its first workshop held in Lausanne, Switzerland in June 2009, Suggested Methods for determining the dynamic strength parameters (uniaxial compression and the Brazilian tension) and fracture toughness of rock materials have been drafted, all based on the split Hopkinson pressure bar (SHPB) techniques. In addition, a thorough literature review was conducted by members of the Commission, and formed the basis for the workshop discussions and content of this book.

## 1.3 ABOUT THIS BOOK

This book is partially a result of the activities of the ISRM Commission on Rock Dynamics. Several contributions are made by non-members. The book is intended to present some recent advances in rock dynamics and engineering applications. It is to be used as a reference for research.

This book consists of 18 chapters representing rock dynamics research and applications. Efforts have been made to be as consistent as possible, in terms of uses of symbols, style and references.

While each chapter is independently prepared by individual authors, the 18 edited chapters have been organised into roughly five sections. Chapter 1 provides an introduction to the topic and background of the ISRM Commission on Rock Dynamics and this edited book, while Chapter 2 provides an overview of the state of the art in rock dynamics research. Chapters 3 to 7 discuss various testing techniques for determining the dynamic properties of rock material. Chapters 8 to 10 focus on some fundamental theories related to rock fracturing under dynamic loads and wave propagation in geological media. Chapters 11 to 14 deal with numerical modelling using some of the most advanced numerical techniques of both continuum and discontinuum methods focusing on micromechanics modelling of rock dynamics problems. Finally, Chapters 15 to 18 present some applications in interpretation of seismic effects, tunnel responses under explosion loading and dynamic rock support.

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Chapter 2

# An overview of some recent progress in rock dynamics research

*Jian Zhao*

## 2.1 INTRODUCTION

Dynamics, as a branch of mechanics, deals with dynamic load (stress), deformation (strain) and failure (fracturing) in relation to time. Hence rock dynamics covers a wide scope, ranging from the initiation of dynamic loads, forms of dynamic loads, transmission and attenuation of dynamic loads, rock fracturing and damage under dynamic loading, to support of rock under dynamic conditions.

This chapter provides a summary of recent progress in some areas of rock dynamics. It covers stress wave propagation and attenuation, loading rate effects on rock strength and discontinuous micromechanics modelling of dynamic fracturing.

## 2.2 STRESS WAVE PROPAGATION AND ATTENUATION

Dynamic loads are generally presented in the form of stress waves. Stress waves, similar to other physical waves, attenuate during propagation, particularly at discontinuities. Since the rock masses are generally discontinuous, containing joint sets, stress wave attenuation at joints is the dominating cause of overall wave attenuation in rock masses. Current researches on wave propagation in rock masses have been focused on wave transmission and transformation across joints.

### 2.2.1 Dynamic loads and stress waves

Dynamic loads are generally the loads applied in a short duration, including impact, cyclic, explosion, and earthquake. For example, impact load, perhaps the most common dynamic load, is the load generated by knocking/hitting of one object onto another object, with very short time duration.

As distinct from static loads, which are generally treated as constant without change in time, dynamic loads change with time. An impact load typically rises quickly from zero to peak and ends in zero, within a very short loading duration. Therefore, they are in the form of waves. Typical forms of dynamic loads are illustrated in Figure 2.1.

The dynamic loading is applied at a point/plane in stress wave forms, and the stress moves further and applies to the next points/planes. The wave propagates at a speed that is governed by the medium in which the wave travels. This speed is

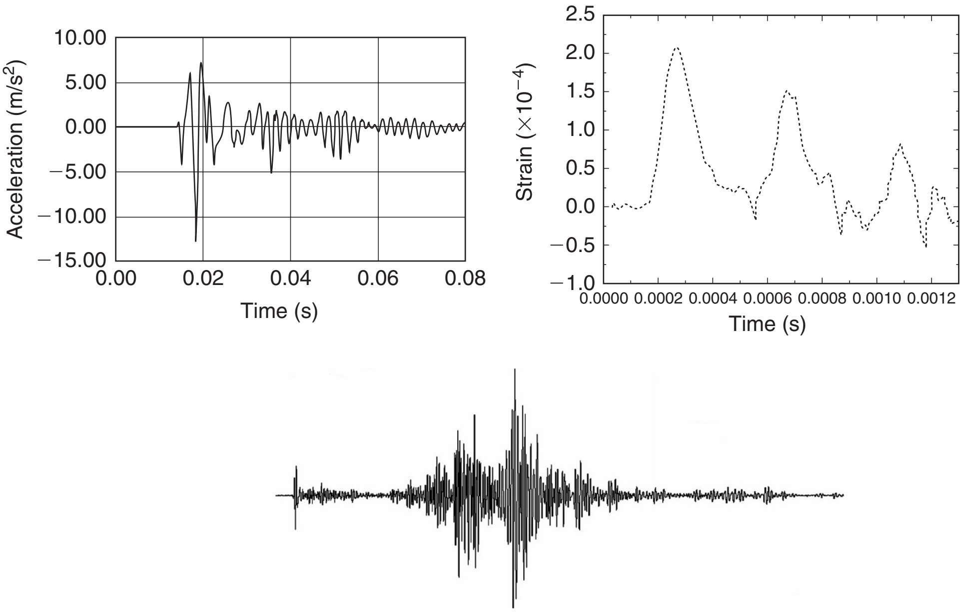


*Figure 2.1* Various types of dynamic loads and their waveforms. Top left: blast wave measured from an explosive test (Zhao *et al.*, 1999b). Top right: impact wave measured from a SHPB test (Li, Ma and Huang, 2010). Bottom: ground acceleration of the Turkmenistan earthquake measured on 6 December 2000 (Landes, Ritter and Wedeken, 2009).

*Table 2.1* Typical compressional wave velocities of various rocks.

| *Igneous Rock* | *P-Wave Velocity (m/s)* | *Sedimentary Rock* | *P-Wave Velocity (m/s)* | *Metamorphic Rock* | *P-Wave Velocity (m/s)* |
|---|---|---|---|---|---|
| Granite | 4500–6500 | Conglomerate | 1500–4500 | Gneiss | 5000–7000 |
| Diorite | 4500–6700 | Sandstone | 1500–5000 | Schist | 4500–6500 |
| Gabbro | 4500–7000 | Shale | 2000–4600 | Phyllite | 4500–6000 |
| Rhyolite | 4500–6000 | Mudstone | 2000–4600 | Slate | 3500–4500 |
| Andesite | 4500–6500 | Dolomite | 3500–6000 | Marble | 5000–6000 |
| Basalt | 5000–7000 | Limestone | 3500–6000 | Quartzite | 5000–7000 |

generally known as seismic velocity, and is the speed of the wave passing through the medium. For a specific medium, seismic velocity is a constant, unless the medium becomes discontinuous. The two most common seismic waves are the compressional (P) wave and the shear (S) wave. Typical values of P wave velocities in rocks are given in Table 2.1.

When a stress wave travels in a medium (solid or fluid), stress is applied to particles of the medium. The particles are accelerated to oscillate around their original positions. The speed of particle movement is termed the particle velocity, and it is the physical speed of particles moving back and forth in the direction the stress passing through.

Particle velocity should not be confused with the seismic velocity, as the latter has a much larger value. The particle velocity is governed by the magnitude and speed of the load. A high particle velocity is generally produced by a high amplitude of the stress wave. Peak particle velocity is often used as a key parameter assessing the failure and stability of rock masses and engineering structures in and on the rocks.

When a stress wave propagates across a rock mass, its amplitude is mainly attenuated at the presence of joints, due to the discontinuity in particle movements. Wave attenuation at joints accounts for a great deal of wave attenuation in a rock mass.

### 2.2.2 Theoretical approaches for wave propagation

There are mainly four models for studying the influences of joints on elastic wave propagation. They are the layered medium model (LMM), the displacement discontinuity model (DDM), the wave scattering model (WSM), and the equivalent medium model (EMM), as summarised in Table 2.2.

With the LMM, which is also termed as the perfect bonded interface model or the displacement continuity model by some researchers, both the stresses and displacements across the joint are continuous (Ewing, Jardetzky and Press, 1957; Brekhovskikh, 1980). There are two kinds of treatment of joints within the LMM. The joint can be modelled as a perfectly bonded interface, or as a layer of the filled weak medium sandwiched between two fully-bonded interfaces

The DDM treats each joint as a non-welded interface of zero thickness. It was originally developed by Mindlin (1960) and applied to seismic wave propagation by Schoenberg (1980). The basic assumption of this method is that, as a wave propagates through a joint, the particle displacements are discontinuous. The displacement discontinuity is equal to the stress divided by the specific joint stiffness. When the joint specific stiffness approaches infinity, the interface becomes a perfectly welded boundary, which can also be modelled with the LMM. When the joint specific stiffness approaches zero, the interface becomes a free surface. For joints with viscoelastic deformational behaviour, the particle velocities as well as the particle displacements are discontinuous (Pyrak-Nolte, Myer and Cook, 1990a). When the joint is filled with viscoelastic material, e.g., saturated sand or clay, due to the existence of the initial mass of the filled joint, besides the particle displacements and velocities, the stresses across the joint are also discontinuous.

The WSM treats the joint as a plane boundary with a distribution of small cracks and voids (Achenbach and Kitahara, 1986; Hudson, 1981; Hudson, Liu and Crampin, 1996). The wave reflection and transmission across a joint is the result of wave scattering through all cracks. According to this model, the stress waves propagating through the joint are considered to be uniformly scattered by the cracks, provided that the crack size is small compared with the wavelength. Apparent wave attenuation due to the scattering of energy at cracks is considered as the principal attenuation mechanism. The wave propagation is determined by crack geometry, crack distribution, crack density, saturation and other parameters. If cracks are filled with liquid, intrinsic attenuation can be taken into account based on the viscous dissipation by the filling liquid.

The EMM (White, 1983; Schoenberg and Muir, 1989; Schoenberg and Sayers, 1995; Li, Ma and Zhao, 2010) treats problems from the viewpoint of entirety. From the EMM, a material and the contained joints together are approximated by an equivalent

*Table 2.2* Theoretical approaches for wave propagation in discontinuous medium.

| *Models* | *Boundary equations to describe the joint* | *Relations* | *Applications* | *Advantages and Disadvantages* |
|---|---|---|---|---|
| LMM Layered medium model | $\Delta u_i = \dfrac{1}{f(d, M_r, M_f)}\sigma_{i3}$ (For filled joint) $\Delta u_i = 0$ (For perfectly bonded joint) | $k$ and $\eta$ can be obtained from $M_r$ and $M_f$, or $G_c$, $M_r$ and $M_c$; $M_f$ can be obtained from $G_c$ and $M_c$. | Filled joint; perfectly bonded joint. | A: accurate. D: very complex. |
| DDM Displacement discontinuity model | $\Delta u_i = \dfrac{1}{f(k, \eta)}\sigma_{i3}$ | | Non-perfectly bonded joint with thickness much smaller than wavelength. | A: simple. D: valid only when joint thickness is much smaller than wavelength. |
| WSM Wave scattering model | $\Delta u_i = \dfrac{1}{f(G_c, M_r, M_c)}\sigma_{i3}$ | | Joint containing a great number of cracks. | A: accurate. D: geometry and distribution of cracks are difficult to obtain. |
| EMM Equivalent medium model | | Changes of equivalent moduli due to the presence of joints are a function of parameters used in boundary equations of LMM, DDM or WSM. | Estimate the overall influence of joints on wave transmission | A: convenient in engineering applications. D: loss of joint discreteness and accuracy. |

Note: $d$ is the joint thickness, $M_r$ is the mechanical properties of the rock material, $M_f$ is the mechanical properties of the filled medium, $k$ is the joint specific stiffness, $\eta$ is the joint specific viscosity, $G_c$ is the geometric and distribution properties of the cracks, $M_c$ is the mechanical properties of the cracks.

continuous, homogeneous and isotropic medium. Thus, stress waves propagate as if the jointed medium is continuous, homogeneous and isotropic. The effect of joints is lumped into effective moduli of the equivalent medium. The methods for calculating the effective moduli are mainly based on the geometry, structures, distributions of the joints, and the filling contained in the joints.

Wave propagation across a single joint has been extensively studied. However, joints in nature are in parallel form as joint sets. Multiple wave reflections among joint sets have great effect on wave propagation (Schoenberger and Levin, 1974; Cai and Zhao, 2000). The overall reflected and transmitted waves are the result of the superposition of reflected and transmitted waves arriving at different times. A simplified method was proposed by ignoring multiple wave reflections as a short-wavelength

*Table 2.3* Different methods applicable for studying wave propagation in jointed rock masses.

| *Methods* | *Domain application* | *Dimension* | *Analyticity* | *Material damping* |
|---|---|---|---|---|
| MC | Time | 1D | Semi-analytical | Not considered |
| SMM | Frequency and time | 1D and 2D | Semi-analytical | Considered |
| VWSM | Frequency and time | 1D and 2D | Semi-analytical | Considered |
| SAM | Frequency and time | 1D | Analytical | Considered |

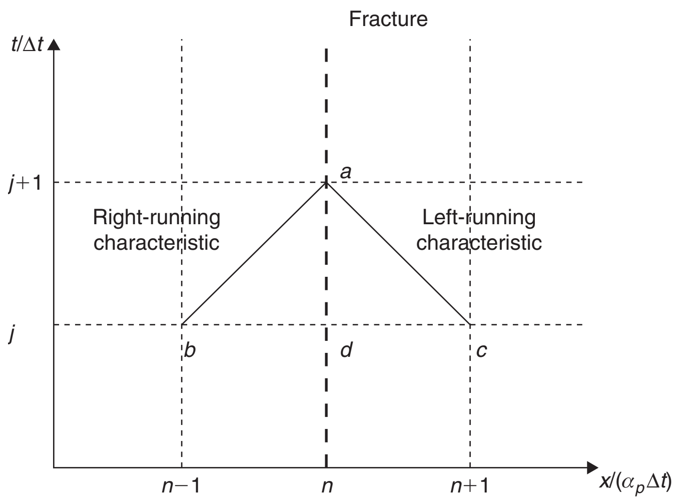


*Figure 2.2* Characteristics in the nondimensional *x-t* plane (Cai and Zhao, 2000).

approximation (Pyrak-Nolte, Myer and Cook, 1990b; Myer *et al.*, 1995). The transmission coefficient across one joint set is calculated as the product of transmission coefficients of individual joints. However, laboratory experiments (Hopkins, Myer and Cook, 1988; Pyrak-Nolte, Myer and Cook, 1990b; Myer *et al.*, 1995) found that the simplified method was valid only when the first arriving wave was not contaminated by multiple wave reflections. When the incident wavelength is comparable to or larger than the joint spacing, the simplified method is not applicable. So far, there are four methods which take into account multiple wave reflections among joints, i.e. the method of characteristics (MC), the scattering matrix method (SMM), the virtual wave source method (VWSM), and the superposed analytical method (SAM). The characteristics of the methods are summarised in Table 2.3 and discussed in detail in Chapters 9 and 10.

The MC (Achenbach, 1973) is a mathematical tool for studying wave propagation across different layers, where multiple wave reflections are taken into account. Based on a one-dimensional wave equation, relations between particle velocity and stress along right- and left-running characteristics can be built (Fig. 2.2). Combined with the DDM, Cai and Zhao (2000) introduced the MC to study wave propagation across parallel joints with linear elastic deformational behaviours. Joints with nonlinear and

Coulomb slip behaviour (Zhao and Cai, 2001; Zhao, Zhao and Cai, 2006) were also studied with the MC.

The SMM, which is also termed the propagation matrix method, was originally used to study electromagnetic wave propagation (Collin, 1992), and adopted to study wave propagation across rock joints (Aki and Richards, 2002; Perino, Barla and Orta, 2010). When an elastic wave impinges on a discontinuity, a scattering phenomenon takes place and can be described by a scattering matrix. When more parallel joints are present, the scattering matrices of each one are combined according to a standard algorithm in order to describe the behaviour of the complete structure, with due consideration of all multiply-reflected waves. The global scattering matrix contains the global reflection and transmission coefficients of a set of parallel discontinuities.

The VWSM, combined with the EMM, is introduced initially for studying normally incident wave propagation across one joint set, where multiple wave reflections among the joints were considered (Li, Ma and Zhao, 2010). The VWS exists at each joint surface and produces a new wave, which is equal to the reflected wave, at each time when an incident wave propagates across the VWS. The VWSM is extended to study the effects of discretely jointed rock masses combined with the DDM (Zhu *et al.*, 2011). With the DDM, VWS exists at the joint position and represents the mechanical properties of the joint. It produces one reflected wave and one transmitted wave each time a normally incident wave arrives at the joint, two reflected waves and two transmitted waves each time an obliquely incident wave arrives at the joint.

Solutions for the MC, the SMM and the VWSM are not explicitly expressed and can be regarded as semi-analytical. The Superposed Analytical Method (SAM) is a new and explicitly expressed analytical method, where multiple wave reflections among joints are superimposed in the analytical solutions (Zhu, 2011). Assuming, but not limiting, that the background rock media of the opposite sides of each joint are identical, the mechanical properties are the same for every joint, and joints are equally spaced, the reflection and transmission coefficients across $2^n$ joints, which are considered as basic solutions, can be expressed as a function of the reflection and transmission coefficients across $2^{n-1}$ joints. Detailed description of the analytical solutions can be found in Zhu (2011).

$$R_{2^n} = R_{2^{n-1}} + \frac{{T_{2^{n-1}}}^2 R_{2^{n-1}} e^{i4\pi\xi}}{1 - {R_{2^{n-1}}}^2 e^{i4\pi\xi}}, \tag{2.1}$$

$$T_{2^n} = \frac{{T_{2^{n-1}}}^2 e^{i2\pi\xi}}{1 - {R_{2^{n-1}}}^2 e^{i4\pi\xi}} \tag{2.2}$$

where $R$ and $T$ are reflection and transmission coefficients, respectively, and $\xi$ is the nondimensional joint spacing, which is defined as the ratio of joint spacing to the wavelength.

In the SAM, the reflection and transmission coefficients across other numbers of joints can be derived through basic solutions. This analytical method can be applied to joints described by different models only if the reflection and transmission coefficients across a single joint are available. It should be noted that this method can also be used to study the general cases where the background rock media of the opposite sides of each joint are different, the mechanical properties are different for every joint, and joints are not equally spaced. Besides, the method can be extended to study obliquely incident wave propagation across one joint set by using a matrix.

Some of the 1D and 2D analytical methods to take into account multiple wave reflections among joints, which are currently available and applicable to studying wave propagation across rock joints and rock masses, are summarised in Table 2.3. Depending on the problem to be solved, a specific model or method can be chosen and adopted.

The study on dynamic stress wave propagation across joints at present is limited to the assumption that the joint may deform (linear and non-linear) but is not damaged. This is often not true in reality. A joint could be crushed and sheared when the stress wave is imposed on it. Damage to the joint contact interface will consume energy and reduce further the wave transmission. Such interaction between wave transmission and joint damage has not been considered so far in the studies. It is envisaged that further research will explore this interaction by combining the works on wave propagation across joints and material fracturing/failure at joint surfaces.

### 2.2.3 Numerical modelling of wave propagation

Compared with theoretical and experimental studies, numerical modelling provides a convenient and economical approach to study wave propagation across a jointed rock mass, especially for complicated cases where theoretical solutions are impossible to obtain and experiments are difficult to conduct.

The representation of joints is a key difficulty in numerical modelling for wave propagation across jointed rock masses. In the finite element method (FEM), joints are often treated as individual elements called joint elements (Goodman, Taylor and Brekke, 1968; Ghaboussi, Wilson and Isenberg, 1973). Boundary interfaces are often used to model joints with the FEM and boundary element method (BEM) (Beer, 1986) or between BEMs (Crotty and Wardle, 1985; Pande, Beer and Williams, 1990). Joints are treated as slide lines in the finite difference method (FDM) (Schwer and Lindberg, 1992). In the discrete element method (DEM), a rock mass is represented as an assembly of discrete blocks and joints as interfaces between the blocks (Cundall, 1971; Shi, 1988).

The finite boundary of the computational model will cause elastic waves to be reflected and mixed with the original wave, which will make analysis of the modelling results more difficult. To solve these problems, an artificial boundary condition that can simulate a computational model without any finite boundaries is needed. This kind of boundary condition is also called a non-reflection boundary condition, which can eliminate the spurious reflections induced by the finite boundary. A number of non-reflection boundary conditions have been proposed in the past. For example, vicous boundary element (Lysmer and Kuhlemeyer, 1969), strip element (Liu and Achenbach, 1994) and infinite element (Gratkowski, Pichon and Razek, 1995) are implemented in FEM and DEM to realize non-reflection boundary.

The universal distinct element code (UDEC), a 2D DEM numerical program, has been widely adopted to study wave propagation across jointed rock masses. Lemos (1987) performed a study on S-wave attenuation across a single joint with Coulomb slip behaviour using UDEC. Brady *et al.* (1990) performed UDEC modelling on the slip of a single joint under an explosive line source. Chen (1999) verified the capability of UDEC to model the responses of jointed rock masses under explosion loading. Zhao *et al.* (2008) carried out numerical studies of P-wave propagation across multiple nonlinearly deformable joints with UDEC.

The Distinct Lattice Spring Model (DLSM) can also be used to study wave propagation across jointed rock masses (Zhu *et al.*, 2010). DLSM is a microstructure-based numerical model, which is meshless and has advantages in modelling dynamic problems including stress wave propagation.

### 2.2.4 Laboratory and field investigation

Pyrak-Nolte, Myer and Cook (1990a, 1990b) conducted experiments on wave propagation across one single joint and one joint set. It was found that joints had significant effects on wave propagation. The joint functioned as a high-frequency filter, i.e., only waves with low frequency can transmit across the joint. However, the multiple wave reflections among the joints were not studied in their research. Zhao *et al.* (2006a) carried out a series of laboratory tests to study wave propagation across one joint set. The transmitted pulses across joints are captured and compared with the results computed with the method of characteristics (MC). Generally, experimental results agree well with those obtained by the MC.

Wave propagation across a filled joint is also performed, where the incident wave is generated through a modified SHPB (Li and Ma, 2009). It is found that the joint width and water content have significant effect on wave transmission through a filled joint.

A two-dimensional physical model to investigate an elastic plane stress wave propagating across joints is established at EPFL (Wu *et al.*, 2011). Different from previous tests, this experimental apparatus can produce plane wave in 2D plates. It can also be used to study obliquely incident wave propagation across a joint set and multiple joint sets.

Cross-hole techniques have been used in a variety of geomechanical exploration and monitoring applications (Auld, 1977; McKenzie, Stacey and Gladwin, 1982; McCann and Baria, 1982; King, Myer and Rezowalli, 1986). The cross-hole method has been found to provide a particularly promising in situ test for studying wave propagation across jointed rock masses and the geomechanical characteristics of jointed rock masses. It was found that the propagation of stress waves in a rock mass containing joints is strongly influenced by the state of stress, changes in temperature, and degree of water saturation. Watanabe and Sassa (1996) performed site geological observation to detect the joints.

There are many criteria to relate stress wave and the performance of rock masses (e.g., Dowding, 1984, 1985, 1996). Among them, the PPV is used as a main stability criterion for engineering structures in and on rocks. Zhao *et al.* (1999b), Chong *et al.* (2002), and Zhou (2011) reported in situ experiments in jointed rock masses to investigate the rock joint effects on wave propagation. It was found that the PPV attenuates with the increase of distance from the charge centre, and the increase of incident angle between the joint strike and the wave propagation path.

### 2.2.5 Wave across multiple joint sets

Wave propagation across multiple joint sets will be further complicated due to the intersecting of joint sets. With the EMM, Schoenberg and Muir (1989) and Schoenberg and Sayers (1995) incorporated the effects of multiple sets of parallel fractures by representing them as group elements. However, EMM have two limitations: loss of discreteness of wave attenuation and intrinsic frequency-dependent properties at individual fractures.

Due to the complexity of wave propagation across multiple joint sets, analytical solutions are difficult to obtain. Hence, numerical modelling and experimental tests are more suitable for studying wave propagation across multiple joint sets with the consideration of joint spacing, number of joint sets and joint sets intersecting angles. While research continues with numerical and physical modelling to obtain wave transmission and transformation across joints and joint sets, future work should also be directed to using numerical methods to simulate multiple jointed rock masses to obtain equivalent parameters for wave propagation, by considering joint frequency and distribution, joint shear and normal stiffness. The requirement for engineering applications is to be able to predict wave attenuation in a rock mass with known common rock mechanics characteristics.

## 2.3 LOADING RATE EFFECTS ON ROCK STRENGTH

Dynamic loads are usually associated with high amplitude and short duration stress pulse or a high loading rate. Mechanical properties of rock materials, including compressive strength, tensile strength, shear strength and fracture toughness, are affected by the loading rate. A proper understanding of the effect of loading rate on rock strength is important in the analysis of mechanical behaviour. Rate effect has been studied experimentally by many researchers (e.g. Abbott, Cornish and Weil., 1964; Stowe and Ainsworth, 1968; Lindholm, Yeakley and Nagy, 1974; Goldsmith, Sackman and Ewerts., 1976; Grady *et al.*, 1977; Li *et al.*, 2001; Zhang *et al.*, 2001; Lok *et al.*, 2002; Backers *et al.*, 2003; Zhang and Hao, 2003; Li *et al.*, 2004; Fuenkajorn and Kenkhunthod, 2010; Liang *et al.*, 2011). All these dynamic tests exhibit a general trend of increase in strength with increasing loading rate. However, the test results are rather scattered because of the complexity of rock types and rock properties.

This section will focus on the observations of rate effects on rock material strength from experiments and the studies of rate dependent mechanisms.

### 2.3.1 Dynamic tests on rock strengths

A fundamental difference between dynamic tests and quasi-static tests is that inertia and wave propagation effects become more pronounced at higher strain rates. Some excellent reviews about the testing methods of strain rate effect on many engineering materials such as concrete, ceramics, rock, silicon carbide and composite materials etc., are presented by Field *et al.* (2004), Gama *et al.* (2004) and Ramesh (2008), and also in Chapters 3, 4, 5 and 6 of this book. Ramesh (2008) classified the common impact tests into four categories according to the objective of the experiment, high-strain-rate

*Table 2.4* Dynamic strength tests and apparatus.

| *Strain Rate* ($s^{-1}$) | *Test Apparatus* | *Testing Principle* | *Applicability* |
|---|---|---|---|
| $\leq 10^2$ | Specialized hydraulic servo-controlled machines | Dynamic load applied by movement of a piston hydraulically driven by gas or oil | Uniaxial compression (e.g. Green and Perkins, 1968; Zhao *et al.*, 1999a); dynamic triaxial compression (e.g. Li, Zhao and Li, 1999)<br>Direct tension (e.g. Yan and Lin, 2006; Asprone *et al.*, 2009); dynamic Brazilian indirect tension (e.g. Zhao and Li, 2000)<br>Punch shear test (e.g. Zhao, Li and Zhao, 1998)<br>Shear of rock joints (e.g. Barbero, Barla and Zaninetti, 1996; Kana *et al.*, 1996) |
| $10^0 \sim 10^3$ | Drop-weight machines | Gravitational potential energy | Flexural loading (e.g. Banthia *et al.*, 1989)<br>Impact and fragmentations (e.g. Whittles *et al.*, 2006) |
| $10^1 \sim 10^3$ | Hopkinson pressure bar | One-dimensional stress wave propagation theory | Uniaxial compression (e.g. Li *et al.*, 2000; Li, Lok and Zhao, 2005; Cai *et al.*, 2007; Zhou *et al.*, 2010)<br>Triaxial compression (e.g. Christensen, Swanson and Brown, 1972; Li *et al.*, 2008; Frew *et al.*, 2010)<br>Direct tension (e.g. Cadoni, 2010; Huang, Chen and Xia, 2010a)<br>Brazilian indirect tension (e.g. Wang, Li and Song, 2006; Cai *et al.*, 2007; Dai and Xia, 2010)<br>Flattened Brazilian disk (FBD) tension (e.g. Wang, Li and Xie, 2009)<br>Semi-circular bend (SCB) test (e.g. Dai, Xia and Luo, 2008)<br>One-point impact test (e.g. Belenky and Rittel, in press)<br>Spalling test (e.g. Erzar and Forquin, 2010) |
| $>10^3$ | Gas gun | High-pressure gas driven projectile | Equations of state (e.g. Shang, Shen and Zhao, 2000) |

experiments, wave-propagation experiments, dynamic failure experiments and direct impact experiments. Experimental techniques to obtain the strength of rock materials under dynamic loading are summarised in Table 2.4.

Ordinary hydraulic servo-controlled testing machines can load specimens at strain rates up to $10^{-3}\ s^{-1}$, but some specialized hydraulic servo-controlled machines such as those developed by Green and Perkins (1968), Logan and Handin (1970), Perkins, Green and Friedman (1970), Zhao *et al.* (1999a), Yan and Lin (2006), Asprone *et al.* (2009) and Cadoni (2010), can achieve strain rates up to $10^2\ s^{-1}$. However, the medium strain rate range (between $10^0$ and $10^2\,s^{-1}$) is very difficult to investigate. The primary approach to testing in this range uses drop-weight machines (Charlie *et al.*, 1993), but great care must be taken in interpreting the data because of the coupling between machine vibrations and wave propagation. The classical experimental technique in the high strain rate range of $10^1 \sim 10^4\,s^{-1}$ is the Hopkinson pressure bar tests for the measurement of rock mechanical properties (Kumar, 1968; Li *et al.*, 2000; Frew, Forrestal and Chen, 2001; Li, Lok and Zhao, 2005; Cai *et al.*, 2007; Xia *et al.*, 2008; Dai *et al.*, 2010). At higher strain rates (i.e. exceeding $10^3\,s^{-1}$), light gas guns have been successfully deployed to test the mechanical properties of rock materials (Shockey *et al.*, 1974; Shang, Shen and Zhao, 2000).

### 2.3.2 Loading rate effects on rock material strengths

Changes of rock strength with loading rate are primarily reported through laboratory tests. There have been many attempts to derive empirical equations to express the relationship between loading rate (or strain rate) and rock material strength.

Based on uniaxial compression tests with strain rate of $10^{-6}$–$10^4\,s^{-1}$ on limestone, Lankford (1981) proposed that:

$$\sigma_{dc} \propto \begin{cases} \dot{\varepsilon}^{1/(1+n_c)} & \dot{\varepsilon} < 10^2\,s^{-1} \\ \dot{\varepsilon}^{1/n} & \dot{\varepsilon} > 10^2\,s^{-1} \end{cases} \tag{2.3}$$

where $\sigma_{dc}$ is the uniaxial dynamic compression strength, $\dot{\varepsilon}$ is the strain rate, $n$ and $n_c$ are material constants, and are equal to 0.3 and 130, respectively in his experiments. Lankford concluded that there exists a critical strain rate for a certain material. When the strain rate is smaller than the critical value, the compressive strength slightly increases with the strain rate. However, when the strain rate is larger than the critical value, the compressive strength switches to rapidly increase with the strain rate.

Olsson (1991) studied the uniaxial compressive strength of a tuff with a strain rate in the range $10^{-6}$ to $10^3\,s^{-1}$. In his experiment, he also found a critical strain rate of $76\,s^{-1}$, and gave the similar relationship,

$$\sigma_{dc} \propto \begin{cases} \dot{\varepsilon}^{0.007} & \dot{\varepsilon} < 76\,s^{-1} \\ \dot{\varepsilon}^{0.35} & \dot{\varepsilon} > 76\,s^{-1} \end{cases} \tag{2.4}$$

In addition, similar conclusions are drawn by Chong and Boresi (1990), and Lajtai, Duncan and Carter (1991).

Based on tests on a granite at strain rate of $10^{-4}$ to $10^0\,s^{-1}$, Masuda, Mizutani and Yamada (1987) noted that the dynamic compressive strength increases with the strain rate, following the relationship given as:

$$\sigma_{dc} = C \log(\dot{\varepsilon}) + \sigma_c \tag{2.5}$$

where $\sigma_c$ is the static uniaxial compressive strength, and $C$ is a constant for the rock material.

Based on tests on a granite with strain rate between $10^{-4}$ and $10^{0}$ s$^{-1}$, Zhao (2000) suggested the relationship can be unified and expressed as:

$$\sigma_{dc} = RSC_d \log(\dot{\sigma}_{dc}/\dot{\sigma}_{sc}) + \sigma_{sc} \tag{2.6}$$

where $\dot{\sigma}_{dc}$ is the dynamic loading rate; $\dot{\sigma}_{sc}$ is the quasi-static loading rate, $\sigma_{sc}$ is the uniaxial compressive strength at quasi-static loading rate (0.5∼1 MPa/s according to ISRM suggested methods), and $RSC_d$ is the dynamic rock strength constant for the rock material.

Logan and Handin (1970) conducted quasi-dynamic triaxial compression tests of the Westerly granite at confining pressures up to 700 MPa, and found the failure strength increases proportionally with increasing loading rate. The rate of increase rises with increasing confining pressure. Green and Perkins (1968) and Masuda, Mizutani and Yamada (1987) also found that at a low confining pressure the effect of loading rate on the strength of a granite is smaller than that at a high confining pressure. However, Yang and Li (1994) reported that the loading rate sensitivity seems to decrease with increasing confining pressure on a marble. Dynamic triaxial compression tests on a granite (Li, Zhao and Li, 1999) showed that the increments of compressive strength with increasing loading rate are different under various confining pressures. The maximum rising rate is 86%, with the strain rates increasing from $10^{-4}$ to $10^{0}$ s$^{-1}$ under the confining pressure of 20 MPa. Zhao (2000) suggested the confining pressure effects can generally account for the effect on strength following the Hoek-Brown strength criterion.

Changes of dynamic tensile strength of rock materials with loading rate have also been reported extensively, mostly with the Brazilian tests (e.g. Price and Knill, 1966; Zhao and Li, 2000; Wang, Li and Song, 2006; Cai *et al.*, 2007; Dai and Xia, 2010; Chen *et al.*, 2009; Cho, Ogata and Kaneko, 2003; Erzar and Forquin, 2010; Asprone *et al.*, 2009; Cadoni, 2010; Huang, Chen and Xia, 2010a). Results all showed that tensile strength increases with loading rate, with similar equations to those of compressive strength proposed based on the experiments.

Dynamic shear tests on rock materials done by Zhao, Li and Zhao (1998) and Fukui, Okubo and Ogawa (2004) concluded that rock material shear strength is also rate-dependent. When the loading rate increases by one order of magnitude, the shear strength increases by approximately 10%. Zhao (2000) further suggested that, based on the results of compression, tension and shear tests, the change of shear strength with loading rate is primarily the change of the cohesion but not the friction angle.

### 2.3.3 Fracture dynamics and strain rate mechanisms

Efforts have been made to study the mechanism governing the rate-dependent behaviour of rock materials (e.g. Kumar, 1968; Qi, Wang and Qian, 2009; Chong *et al.*, 1980; Blanton, 1981; Chong and Boresi, 1990; Morozov and Petrov, 2000 and Ou, Duan and Huang, 2010).

Rock is typically a brittle and inhomogeneous material, containing initial defects such as grain boundaries, micro-cracks and pores. There have recently been increasing studies of inhomogeneity effects on the failure mechanism of rock materials. Some researchers (e.g. Cho, Ogata and Kaneko, 2003; Cho and Kaneko, 2004; Zhu and Tang, 2006; Zhou and Hao, 2008; Zhu, 2008) incorporated the rock inhomogeneity into numerical methods, and successfully simulated progressive failure of rock materials under both static and dynamic loading conditions. These analyses revealed that the differences are due to the stress concentrations and redistribution mechanisms in the rock. The rock inhomogeneity also contributes to the difference between the dynamic and static tensile strengths. In addition, Cho and Kaneko (2004) used the same method to investigate the influence of applied pressure waveforms on dynamic fracture processes in rocks.

Observation from the experiments showed that at high loading rates, rock materials fail with more fractures and fragments are of smaller size. This observation is often related to the strength increase. Since more fractures are generated at high loading rates, more energy is consumed hence leading to higher loads and higher strengths. There is certainly a connection between high density of fracturing and high strength. However, the reasons for more fracturing are still under investigation, and are believed to be micromechanics based (Kazerani and Zhao, 2010).

Micromechanics-based crack models have been investigated (e.g. Zhang, Wong and Davis, 1990; Wong, 1990; Wong *et al.*, 2006; Brace and Bombolakis, 1963; Nemat-Nasser and Horii, 1982; Ashby and Hallam, 1986; Deng and Nemat-Nasser, 1992, 1994; Nemat-Nasser and Deng, 1994; Ravichandran and Subhash, 1995; Huang, Subhash and Vitton, 2002; Huang and Subhash, 2003; Zhou *et al.*, 2004; Zhou and Yang, 2007; Li, Zhao and Li, 2000; Xie and Sanderson, 1995; Alves, 2005; Saksala, 2010; Wang, Sluys and de Borst, 1997; Ambrosio and Tortorelli, 1990; Bourdin, Larsen and Richardson, 2010; Larsen, Ortner and Süli, 2010). Paliwal and Ramesh (2008) developed an interacting micro-crack damage model based on sliding of pre-existing cracks for the estimation of the strain rate dependent constitutive behaviour of brittle materials, which shows a good agreement with experiments (Paliwal and Ramesh, 2008; Kimberley, Ramesh and Barnouin, 2010). In order to evaluate the variability of the mesoscale strain rate dependent constitutive behaviour in brittle materials, Graham-Brady (2010) improved on the interacting micro-crack damage model by incorporating statistical characterization of mesoscale random cracks.

Zuo *et al.* (2006) presented a rate-dependent damage model, the Dominant Crack Algorithm (DCA), for the damage of brittle materials based on the dominant crack. Zuo, Disilvestro and Richter (2010) recently proposed a rate-dependent crack mechanics based model by incorporating plastic deformation into the DCA model for damage and plasticity of brittle materials under dynamic loading.

Kazerani (2011) and Kazerani and Zhao (2010, 2011) studied rock fracturing with microscopic discrete element modelling and revealed that the rate dependency observed in the experiments may be due to several causes: the intrinsic rate-dependent properties of the microstructure, the structural rate dependent properties of the rock material composition, and the testing conditions. The structural rate dependent properties of the rock material composition are related to the mineral grain structure and homogeneity. The intrinsic rate dependent properties of the microstructure are on the cohesion of the bond between microelements. Testing conditions, such as end frictions at loading and supporting points/planes also contribute to the rate effects.

### 2.3.4 Rock dynamic strength criteria

Based on dynamic experimental data of the Bukit Timah granite (Li, Zhao and Li, 1999; Zhao *et al.*, 1999a; Zhao and Li, 2000), Zhao (2000) examined the applicability of the Mohr-Coulomb and the Hoek-Brown criteria to rock material strength in the dynamic range.

The Mohr-Coulomb criterion is only applicable to dynamic triaxial strength in the low confining pressure range. It appears that the change in the strength with loading rate is primarily due to the change of cohesion, and the internal friction angle seems unaffected by loading rate. The dynamic triaxial strength can be estimated as

$$c_d = \sigma_{dc}(1 - \sin\phi)/2\cos\phi \tag{2.7}$$

$$\sigma_{d1} = \sigma_{dc} + \sigma_3(1 + \sin\phi)/(1 - \sin\phi) \tag{2.8}$$

where $c_d$ is the dynamic cohesion, and $\phi$ is the friction angle.

The dynamic triaxial strength can be represented by the Hoek-Brown criterion at low and high confining pressure ranges for the loading rate range examined. It may be assumed that the parameter $m$ (a constant in the Hoek-Brown criterion) is not affected by the loading rate. Hence, the dynamic triaxial strength can be estimated from

$$\sigma_{d1} = \sigma_3 + \sigma_{dc}(m\sigma_3/\sigma_{dc} + 1.0)^{0.5} \tag{2.9}$$

Additional testing data will provide further verification of the above conclusions for other rocks and for the wide range of loading rates.

## 2.4 NUMERICAL MODELLING OF ROCK DYNAMIC FRACTURING

Dynamic fracturing of rock governs the strength and failure mode, and is one of the most important research issues in rock dynamics. However, the real mechanism of the rate-dependency for fracturing pattern and mechanical properties of rock under dynamic loading is still not clear. Facing this problem, both the experimental method and the analytical method are limited. With the rapid advancement of computing technology, numerical methods provide powerful tools. The combination of numerical and physical modelling methods can be the best applicable solution to provide the insight of rock fracturing dynamics. In this section, numerical methods used for rock fracturing dynamics are briefly reviewed. Detailed reviews on the corresponding classical numerical methods and the newly developed numerical methods can be found in Chapters 13 and 14 of this book.

### 2.4.1 Numerical methods for fracturing modelling

Generally, numerical methods used in rock mechanics are classified into continuum based method, discontinuum based method and coupled continuum/discontinuum method (Jing, 2003). The continuum based methods are methods which based on continuum assumption, examples are the Finite Element Method (FEM) (Clough, 1960), the Finite Difference Method (FDM) (Malvern, 1969), and the Smoothed Particle Hydrodynamics (SPH) (Monaghan, 1988). The merits of continuum methods are directly inputting macro mechanical parameters which can be obtained from experiments and precisely modelling the stress state of pre-failure stage. Moreover, computer codes for continuum-based methods are also relatively mature, e.g., LS-DYNA (LSTC, 2010), ABQUS (SIMULIA, 2010), FLAC (ITASCACG, 2010) and RFPA (MECHSOFT, 2010) are commercial computer codes which can be used to model dynamic fracturing problems. However, the continuum assumptions in these continuum-based methods make them unsuitable for dealing with complete detachment and large-scale fracture opening problems. It is also difficult to apply continuum-based methods to solve problems which involve complex discontinuity, such as jointed rock masses and rock in post-failure state.

Discontinuum-based methods treat rock material or rock mass as an assembled model of blocks, particles or bars, e.g., the Distinct Element Method (DEM) (Cundall, 1971), Discontinuous Deformation Analysis (DDA) (Shi, 1988) and Distinct Lattice Spring Model (DLSM) (Zhao, 2010). In these methods, the fracturing process of rock is represented by the breakage of inter-block contacts or inter-particle bonds. Discontinuum-based methods can reproduce realistic rock failure processes especially the post failure stage. However, they are not best suited for stress state analysis of pre-failure rock. Available commercial computer codes based on DEM are UDEC/3DEC and PFC (ITASC, 2010) and DDA (Shi, 1988). There also exist some research codes, for example, DLSM (Zhao, 2010).

In order to overcome the limitations of both continuum and discrete methods, coupled methods have been developed in recent years. For example, the Numerical Manifold Method (NMM) (Shi, 1991) was developed to integrate DDA and FEM, the FEM/DEM method (Munjiza, 2004) is designed to couple FEM with DEM, and the Particle-based Manifold Method (PMM) (Zhao, 2009) was proposed to combine DLSM and NMM. The coupled method is capable of capturing both the pre-failure and the post-failure behaviour of rock materials. However, its implementation is difficult and no commercial codes are available now. There only exist some research codes, e.g., NMM (Shi, 1991), Y2D (Munjiza, 2004), m-DLSM (Zhao, 2010). In Table 2.5, a summary on these numerical methods and corresponding computer codes are listed.

*Table 2.5* Numerical methods for rock dynamic problems.

| *Numerical Methods* | *Typical Software/Code* | *General Applicability* |
|---|---|---|
| Continuum based: FEM, FDM, BEM, SPH | LS-DYNA, ABQUS, FLAC, RFPA | Displacement without element detachment |
| Discontinuum based: DEM, DLSM | UDEC/3DEC, PFC, DDA, DLSM | Element detachment, rock fracturing, rock block movement |
| Coupled/hybrid based: combined methods | FEMDEM, NMM, Y2D, m-DLSM, PMM | Multiscale, displacement, fracturing, and block movement combined |

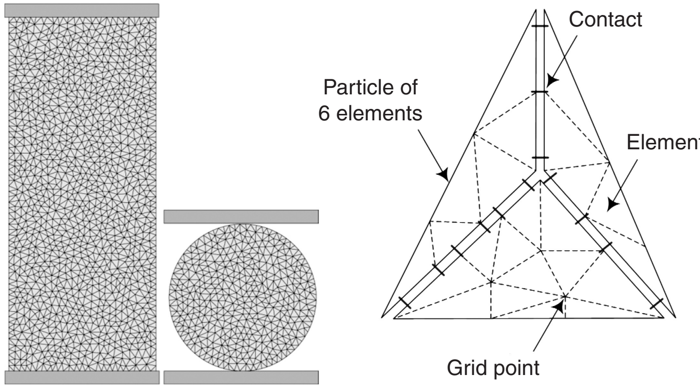


*Figure 2.3* UDEC model for dynamic fracturing simulation (left), and basic unit (right).

### 2.4.2 Micromechanics modelling of rock dynamic fracturing using UDEC

Dynamic fracturing of heterogeneous materials such as rock and concrete cannot be modelled realistically without appealing to their microstructures. This requires that a successful numerical method must be capable of considering the formulation and evolution of micro discontinuities. Recently, the micro dynamic fracturing of rock is modeled by using UDEC through implementing a triangulation pre-processor and a rate-dependent cohesive law (Kazerani and Zhao, 2010). The basic scheme is shown in Figure 2.3, in which the material is represented as an assembly of distinct particles/bodies interacting at their boundaries. The interface between these particles is viewed as a contact which in fact represents grain-interface or grain cementation properties for igneous or sedimentary rocks, respectively. In order to model the dynamic fracturing of rock materials, a full rate-dependent cohesive law was proposed (Kazerani, 2011; Kazerani and Zhao, 2010). The model was used to model the tensile and compressive failure of rock materials, and compared well with experimental results (Kazerani and Zhao, 2011). It is also used for simulating the dynamic fracture toughness test of rock materials, dynamic crack propagation of PMMA plate (Kazerani and Zhao, 2011) and dynamic failure of joints under shear force (Kazerani, Zhao and Yang, 2010).

### 2.4.3 Particle-based Manifold Method (PMM) for multiscale rock dynamics modelling

Particle-based Manifold Method (PMM) is a new particle-based multi-scale numerical method and corresponding computer code, currently under development by EPFL-LMR (Zhao, 2009; Sun, Zhao and Zhao, 2011). PMM introduces the microscopic particle concept into the numerical manifold method (NMM) and rebuilds a particle manifold method (Fig. 2.4). It unifies continuum-discontinuum models at micro scale.

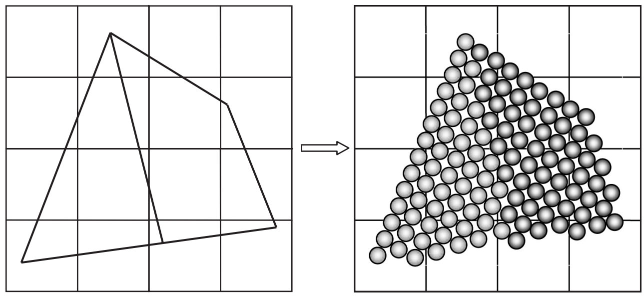

*Figure 2.4* NMM model (left) of two blocks with rectangular mathematical meshes, and PMM model (right), where the blocks are approximately replaced by particles and the mathematical meshes remain.

Further, PMM can be incorporated with NMM for multi-scale modelling. In summary, PMM provides the following new features:

i) PMM is a dynamic model. Motion of discontinuum can be accurately described by inertial equations. Static simulation is also available when the velocity is ignored.
ii) PMM is a fully implicit model. All unknowns are solved by a global mathematical equation. The contact behaviours are described by the penalty method and the open-close iteration is inherited from manifold method to make contact state convergent.
iii) PMM extends NMM to micro scale simulation. By importing proper failure mechanisms, PMM could simulate explicit processes with implicit modelling.
iv) PMM is capable of presenting material nonlinearity and inhomogeneity. The separation of mathematical mesh and material mesh frees the description of physical domain without the limitation of drawing meshes. Inhomogeity is described at the micro scale.
v) An analytical sphere simplex integration is given to guarantee the accuracy of integration on physical domain.
vi) PMM has mobility of contact mechanism and failure model. PMM overcomes the difficulty of 3D implementation of NMM by replacing the polyhedron-to-polyhedron contact by the sphere-to-sphere contact.

The advantages of PMM, including unified implicit computational format, accurate dynamic simulation, and microscale and manifold features, make the model a suitable tool for analysing rock dynamics, especially when dealing with dynamic fracturing.

Multi-scale modelling is regarded as an exciting and promising methodology due to its ability to solve problems which cannot be handled directly by microscopic methods due to the limitation of computing capacitance (Guidault *et al.*, 2007; Hettich, Hund and Ramm, 2008; Xiao and Belytschko, 2003). The most direct way to build a

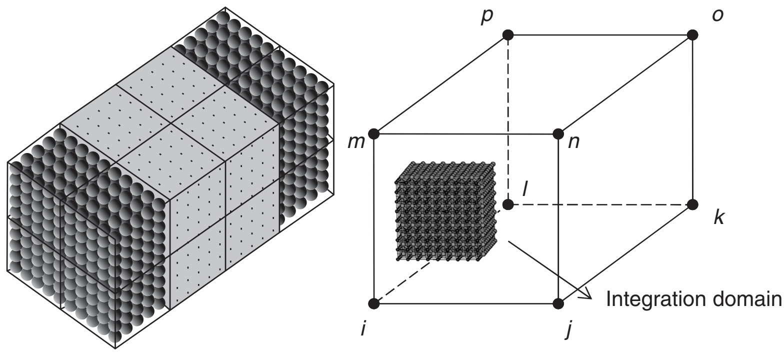


*Figure 2.5* Particle based Manifold Method (PMM) model (left) and basic element (right).

multi-scale numerical model is to combine two different scale methods. This methodology has been widely used, for example, in the coupling of MD with continuum mechanics models (Mullins and Dokainish, 1982; Hasnaoui, van Swygenhoven and Derlet, 2003). The PMM is to couple with the DLSM (Distinct Lattice Spring Model) (Zhao, 2010; Zhao, Fang and Zhao, 2010) and the NMM. The computational model of PMM is shown in Figure 2.5. The PMM element is realized by replacing the physical domain of the manifold element in NMM by the particle-based DLSM model. The implementation details of this method are given by Zhao (2010). As a newly developed numerical method, only a few examples are given (e.g., Zhao, 2010; Zhu *et al.*, 2011; Kazerani, Zhao and Zhao, 2010). The implicit PMM and GPU based high performance PMM code is still under the development (Sun, Zhao and Zhao, 2011). The new computer code will provide useful solvers for rock dynamic problems at multiscale.

## 2.5 PROSPECTS OF ROCK DYNAMICS RESEARCH

Rock dynamics research is not limited to the aspects discussed in the previous sections. It has a much wider scope, with topics ranging from wave propagation, to response of rock material and rock mass, to engineering applications, dealing with microscopic fracturing of rock material to dynamic behaviour of rock masses (Zhao *et al.*, 2006b). There are indeed many issues yet to be covered in rock dynamics. Some of the important aspects requiring investigations are discussed below.

### a) Wave propagation in rock joints

Further studies in this field need to be focused on the coupling of wave attenuation and joint geometrical properties, such as spacing, frequency, aperture, roughness and filling. Typical information on rock joints includes orientation, aperture and filling, surface roughness, spacing and frequency, which can be generally measured. Spacing, frequency and orientation can remain as geometrical parameters and can be the input for either analytical solutions or numerical modelling. Aperture and roughness can be correlated to mechanical properties such as joint normal stiffness and shear strength.

Therefore, it is possible to incorporate those rock joint parameters in the wave propagation analysis, particularly in numerical modelling, to estimate wave attenuation in the jointed rock masses.

To deal with filled joints, mechanical properties of filling materials (e.g., sand or clay) can be determined and incorporated into the wave propagation analytical solutions by treating the filling as a viscous material.

#### b) Wave propagation in rock masses

Studies along this line are to develop equivalent medium wave propagation parameters for jointed rock masses, by incorporating rock mass parameters. Statistic approached may be adopted to represent the geometrical distribution of joints and of the joint properties for rock masses. This can be achieved by performing a parametrical study using numerical modelling to generate a representative rock mass and then to obtain a wave attenuation coefficient for that rock mass.

#### c) Interaction of wave transmission and joint damage

Joint damage associates with energy consumption, and complicates the wave propagation equation. For analytical solutions, one must consider the energy balance at the failure of the joint surface asperities under compression and shearing.

There are possibilities for exploring the interaction between wave transmission and joint damage by physical and numerical modelling. For numerical modelling, the challenges will be the simulation of rock joint surface damage under dynamic loads. Micromechanical discrete element modelling is likely to be required in such cases in order to model the fracture and failure of rock joint surfaces.

#### d) Rock fracture induced seismic energy and wave

When a highly stressed (or strained) rock (material or joint) fractures, the stored strain energy is released at the facture plane. If the energy released is sufficiently large, it can cause induced seismic events. Physical experiment may offer direct observation on energy release patterns (amplitude and form), with good monitoring devices. Chapter 15 addresses this issue.

Numerical modelling, particularly micromechanics-based discrete element methods, will be good tools to capture the phases of statically-strained rock materials, sudden fracturing, and released and propagation of dynamic stress.

#### e) Mechanics of rock fracturing and rate effects

While it is clear that at high loading rate, rock material strengths increase and rock material fails with more fractures, it is not clear yet what is the cause of high density of fracturing. There are indeed many opportunities within this field to explore the mechanical and physical cause of rate effects on rock strength and failure pattern. For example, rate effects on fracture branching, rate effects on multiple fracture initiation, and rate effects on crack propagation velocity.

Further study also needs to be conducted on the shear strength of rock joints under dynamic loads, to understand the rate effects on shear strength and dilation.

#### f) Micromechanics modelling of rock fracturing and failure

As already mentioned in (e), numerical modelling of rock fracture and failure need to be micromechanics- and discrete-based. There are two aspects which need to be addressed.

One is to incorporate micromechanical constitutive laws and input parameters into the existing codes, such as UDEC and DDA. The second is to develop new microscale numerical codes with specific focus on modelling fracture initiation, propagation and branching. The need for correlation with physical modelling will also advocate experimental progress in terms of high-speed, high-resolution micromechanics monitoring and observation.

The other question that micromechanics modelling should address is the effect of element size. It is argued that if the elements are sufficiently small, the contact force between the elements will be sufficiently simple and non-rate dependent (Zhao, Wang and Tang, 2008). It needs to be verified and also determined with the element size.

#### g) Static-dynamic interaction

Rock dynamics also covers the dynamic failure processes under existing static loading conditions, as reflected by rock burst and spalling. Rock burst mechanism, failure pattern, energy release, fracture propagation velocity and distance are likely to be affected by static strain energy (in situ stress) and triggering mechanism (e.g. stress re-distribution due to excavation). The process may involve static-dynamic transition and interaction. Such a study will require multiscale and multimechanics approaches.

Other areas which can be explored are the interaction between rock fracturing and groundwater, gas and pore pressure and temperature, which may extend the approaches to multiphysics.

#### h) Rock and earthquake engineering applications

In parallel with the fundamental studies outlined above, rock dynamics will be continuously applied to engineering and construction. Stability of slopes and tunnels under various dynamic conditions (earthquake and explosion), reinforcement and support of rock slope and tunnels for dynamic loads, use of explosives and blast damage control, seismic and vibration hazard control, are some typical examples of engineering applications needing to be addressed.

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Chapter 3

# Split Hopkinson pressure bar tests of rocks: Advances in experimental techniques and applications to rock strength and fracture

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## 3.1 INTRODUCTION

The accurate measurement of rock dynamic mechanical properties has always been a very important task for a variety of rock engineering and geophysical applications, which include quarrying, drilling, rock bursts, blasts, earthquakes, and projectile penetrations. In these applications, the rock materials are subjected to dynamic loading over a wide range of loading rates. Therefore, accurate determination of dynamic strength and toughness properties of rocks over a wide range of loading rates is crucial. However, in sharp contrast to the static rock testing methods, no recommended methods have been suggested by the International Society of Rock Mechanics (ISRM). In addition, the existing dynamic testing results with different methods and instrumentations are so scattered that cross-referencing of others' results is unfeasible. It is thus necessary and urgent for the rock mechanics community to develop reliable suggested methods to standardize the mechanical testing of rocks under high loading rates.

To test dynamic mechanical properties of rocks, we need a reliable testing device. For testing rock materials under high strain rates ($10^2 \sim 10^3$ s$^{-1}$), split Hopkinson pressure bar (SHPB) is an ideal dynamic testing machine. As a widely used device to quantify the dynamic compressive response of various metallic materials at high loading or strain rates, SHPB was invented in 1949 by Kolsky (Kolsky, 1949; Kolsky, 1953). Shortly after that, SHPB was attempted by researchers to test brittle materials such as concretes (Ross, Thompson and Tedesco, 1989; Ross, Tedesco and Kuennen, 1995), ceramics (Chen and Ravichandran, 1996; Chen and Ravichandran, 2000) and rocks (Christensen, Swanson and Brown, 1972; Dai, Xia and Tang, 2010). However, some major limitations of using SHPB for brittle materials were not fully explored until two decades ago (Subhash, Ravichandran and Gray, 2000).

Unlike ductile metals, brittle materials have small failure strains (<1%) and hence if the loading is too fast, as in a conventional SHPB test, the specimen may fail in a non-uniform manner (i.e., the front portion of the sample may be shattered while the back portion of the sample remains intact.). To achieve accurate measurements in SHPB tests, one has to make sure that the dynamic loading is slow enough so that the specimen is experiencing an essentially quasi-static load, and thus the deformation of the specimen is uniform. As a rule of thumb, it takes the loading stress wave to travel in the specimen 3–4 rounds for the stress to achieve such an equilibrium state. The pulse-shaping technique was proposed to slow down the loading rate and thus to minimize the so-called inertial effect associated with the stress wave loading (Frew, Forrestal and Chen, 2001). Another problem in conventional SHPB tests is that the specimen is subjected to multiple loading due to the reflection of the wave at the impact end of the incident bar. A momentum-trap technique was proposed to ensure single pulse loading and thus enable valid post-mortem analysis of the recovered specimen (Nemat-Nasser, Isaacs and Starrett, 1991). Other advancements in SHPB can be found in a recent review (Field *et al.*, 2004).

Using these new techniques in SHPB, we systematically measured the dynamic mechanical properties of rocks. A few new testing methods were developed to accurately measure the dynamic compressive strength and response, the dynamic tensile strength, and dynamic fracture parameters of rocks. For all these tests, we used core-based rock specimens to facilitate sample preparation. In the rock dynamic compression, we addressed the issue of the length to diameter ratio of the cylindrical rock specimen. In the static uniaxial compressive strength (UCS) tests, the length to diameter ratio is required to be 2 or more to minimize the end frictional effect; in SHPB tests, the friction is dynamic and thus the frictional effect is presumably smaller. Shorter specimen favors dynamic stress equilibrium but has worse frictional effect. An optimal length to diameter ratio was sought. The dynamic tensile strength measurements using SHPB were conducted using the Brazilian disc (BD) method. This method was fully validated on the dynamic force balance and quasi-static data reduction with the aid of high speed photography. We proposed the fracture onset detection to determine the correct value of the far-field load at failure for calculating the rock tensile strength. There are two methods used to measure the dynamic fracture toughness of rocks: the notched semi-circular bend (SCB) method and the cracked chevron-notched Brazilian disc (CCNBD) method. Using a special optical technique to monitor the crack surface opening distance (CSOD), we observed the stable fracture to unstable fracture transition in dynamic CCNBD tests. We also showed that using our optical device, the dynamic fracture energy and fracture velocity of rocks can be estimated.

The chapter is organized as follows. The principles of SHPB and the new testing techniques are covered in Section 2. The application of SHPB to dynamic compressive tests, dynamic tensile tests and dynamic fracture tests of rocks are discussed in Section 3, Section 4, and Section 5 respectively. Section 6 concludes the materials presented in the entire chapter.

## 3.2 PRINCIPLES OF SPLIT HOPKINSON PRESSURE BAR AND NEW TECHNIQUES

### 3.2.1 The split Hopkinson pressure bar system

SHPB is composed of three bars: a striker bar, an incident bar, and a transmitted bar (Gray, 2000). The impact of the striker bar on the free end of the incident bar induces a longitudinal compressive wave propagating in both directions. The left-propagating wave is fully released at the free end of the striker bar and forms the trailing end of the incident compressive pulse $-\varepsilon_i$ (Fig. 3.1). Upon reaching the bar-specimen interface, part of the incident wave is reflected as the reflected wave $-\varepsilon_r$ and the remainder passes through the specimen to the transmitted bar as the transmitted wave $-\varepsilon_t$.