Mervyn S. Paterson

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Experimental Rock Deformation – The Brittle Field

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Second, Completely Revised and Updated Edition

With 87 Figures

Springer

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The first edition of this book was published by Springer-Verlag in 1977 within the book series 'Minerals and Rocks' (ISBN 3-540-08835-0).

Library of Congress Control Number: 2004117900

ISBN-10 3-540-24023-3 **Springer Berlin Heidelberg New York**
ISBN-13 978-3-540-24023-5 **Springer Berlin Heidelberg New York**

Springer is a part of Springer Science+Business Media
springeronline.com

Printed in The Netherlands

Cover design: Erich Kirchner, Heidelberg
Typesetting: Büro Stasch (stasch@stasch.com), Klaus Häringer, Bayreuth
Production: Luisa Tonarelli
Printing: Krips bv, Meppel
Binding: Litges & Dopf, Heppenheim

Printed on acid-free paper 32/2132/LT – 5 4 3 2 1 0

Preface

This monograph deals with the part of the field of experimental rock deformation that is dominated by the phenomena of brittle fracture on one scale or another. Thus a distinction has been drawn between the fields of brittle and ductile behaviour in rock, corresponding more or less to a distinction between the phenomena of fracture and flow. The last chapter deals with the transition between the two fields. In this new edition an attempt has been made to take into account new developments of the last two and a half decades. To assist in this project, the original author greatly appreciates being joined by the second author.

The scope of the monograph *is* limited to the mechanical properties of rock viewed as a material on the laboratory scale. Thus, the topic and approach is of a "materials science" kind rather than of a "structures" kind. We are dealing with only one part of the wider field of rock mechanics, a field which also includes structural or boundary value problems, for example, those of the stability of slopes, the collapse of mine openings, earthquakes, the folding of stratified rock, and the convective motion of the Earth's mantle. One topic thus excluded is the role of jointing, which it is commonly necessary to take into account in applications in engineering and mining, and probably often in geology too. Shock phenomena have also not been covered.

We attempt to bring out the principal aspects of brittle behaviour, with special attention to the fundamental physical aspects. We hope, in doing so, to provide also a useful introduction to the basics of rock properties for engineering and earth science applications. We have tried to present the material in a way that will enable advanced students and others not specialized in this field to grasp the context, or to guide them to where relevant background material is to be found (for example, many references have been made to appropriate sections in Jaeger and Cook (1979)). Thus *we* hope that the monograph will be useful to students and non-specialists as well as to those actively engaged in research in this field.

The primary aim of the book is to set down the state of current knowledge of the *brittle* mechanical properties of rock as determined in laboratory experiments, covering also the history of the developments leading to this state and giving a fairly comprehensive listing of the published material. The presentation has therefore something of the character of a guided tour of the literature, and it is hoped that it will serve enduringly as a reference source.

For the new edition one entirely new chapter (Chapt. 6) has been added to deal with theoretical developments that mainly post-date the first edition. Extensive revision has also been done in Chapters 5, 8 and 9, while the remaining chapters have been revised in minor ways , incorporating new references. The first edition covered the period up to about 1977 with an attempt to give a comprehensive coverage of the literature on the main topics. In view of the exponentially expanding publication rate, it has not been feasible to attempt anything approaching complete coverage of the literature since 1977 in the new edition. However, we have attempted to include what have appeared to us to be the main contributions that have appeared in the interim. We apologize for omissions or important papers that we have overlooked.

As far as possible, we have referred to publications that are generally available in libraries and have avoided reference to theses and reports not having wide distribution. We have also, on the whole, avoided reference to abstracts of papers presented to meetings but we have made a general exception in the case of EOS (Transactions of the American Geophysical Union) where the material is of particular interest and we have been unable to trace its publication in full.

The S. I. system of units is used and conversion has been made to these units where necessary in presenting data. The quantity most often quoted is stress, for which the most convenient unit is the meganewton per square metre, called the megapascal (MPa). One megapascal is equal to 10 bars, so to convert from MPa to bars simply add one zero. Useful conversions are:

1 MPa = 10 bar = 145 lb in^{-2}

100 MPa = 1 kbar = 6.47 imperial ton in^{-2}

4.18 kJ mol^{-1} = 1 kcal mol^{-1} = 0.0434 eV atom^{-1}

1 J m^{-2} = 1000 erg cm^{-2}

The sign convention adopted for stress and strain is that compressive stress is positive and shortening strain is positive. Thus, if l is the length and Δl is a small increase in the length, we define the infinitesimal strain as $\varepsilon = -\Delta l/l$. Similarly if v is the volume and Δv a small increase in volume, we define the volumetric strain ε_v as $\varepsilon_v = -\Delta v/v$. By adopting these conventions in both stress and strain, the elastic constants and moduli remain positive quantities.

We are grateful to Drs. W. F. Brace, E. T. Brown, M. Friedman, R. C. Liebermann, S. A. F. Murrell, and E. H. Rutter for critical reading of the original individual chapters and to Drs. N. Beeler, B. Evans, P. Meredith and J. Rudnicki for reviews of the new material. We are also grateful to our colleagues for many useful discussions and criticisms. Finally special thanks go to our wives for their understanding, forbearance and support during the rather long gestation of this revision.

Canberra and Stony Brook, September 2004 *M. S. Paterson*
Teng-fong Wong

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Chapter 1

Introduction

A knowledge of the mechanical properties of rocks is fundamental to several branches of both Earth science and engineering. However, depending on the environmental conditions and on the scale in space and time, many different aspects of mechanical behaviour may be involved. This diversity ranges from effects associated with the very small, short-term, essentially elastic strains in earthquake waves to the large, slow, irreversible deformations of the Earth's crust and mantle that are involved in the gross tectonic processes of ocean-floor spreading and continental evolution.

Because of the variety of viewpoints, research on the mechanical behaviour of rocks has tended to develop somewhat independently in several directions not closely related to each other. In particular, there has tended to be a separation between studies undertaken because of interest in civil engineering and mining and studies aimed at geologic understanding. There are also important aspects of rock behaviour that involve porosity of the rock and the movement of fluids through the pore structure, with applications to fields such as petroleum engineering, nuclear waste storage and environmental pollution problems (Committee on Fracture Characterization and Fluid Flow 1996).

In the engineering fields the problems are commonly concerned with possible failure to support a load, e.g., in a mine opening or slope, or with the avoidance of excessive displacement, as in the case of a dam foundation. In either case, it is rare that large strains are involved, but a knowledge of fracture properties is usually vital. Thus, engineering rock mechanics tends to be preoccupied with brittle behaviour.

In the Earth sciences, brittle behaviour is also of interest, as in faulting and the mechanics of earthquakes. However, now there is a wide range of phenomena, extending from the small-scale penetrative deformation studied by the geologist in the hand specimen to the large-scale deformations involved in major tectonic processes, which depend on the ability of rock to undergo large plastic deformations. So the rock mechanics problems in the Earth sciences often also involve those of ductile behaviour. Thus, the fields of engineering and geologic rock mechanics are not exclusively distinguished by their respective concerns with brittle and ductile behaviour, and any research on mechanical properties may be potentially of interest in either field.

The task of establishing the mechanical properties of rocks is undertaken mainly in the laboratory. It constitutes an important branch of materials science, both for the potential practical applications and for the intrinsic scientific interest of exploring other classes of materials, especially where silicon-oxygen bonding or pore structure is involved. However, attention should be drawn to an important constraint of scale in applying laboratory results to practical situations.

In most bodies of rock there are structural features, notably joints, on a coarser scale than the scale of specimens used in laboratory tests, and these features often have an important influence on the mechanical behaviour of the body of rock. In the extreme, the strength of the rock mass may be determined almost entirely by the properties of the joints, the blocks between the joints behaving largely as rigid elements. This circumstance leads to a distinction being made between the behaviour of the "rock material" and the behaviour of the "rock mass" (cf. Jaeger and Cook 1979). The laboratory tests normally give information about the properties of the rock material itself but other steps may have to be taken to determine the properties of the rock en masse. Whether such a distinction between rock material and rock mass need be made in particular cases in practice has to be decided in the individual situation; it may not always be of significance, especially where the processes involved are pervasive down to a fine scale, as with microcracking or crystal plasticity. The effects of scale tend, in general, to be of greater importance in brittle behaviour than in ductile but must always be borne in mind in extrapolating to practical situations.

In treating the laboratory study of inelastic behaviour of rocks, the major division of the subject is naturally that into the brittle and ductile fields. The distinction between brittle and ductile behaviour is in the first place a macroscopic one, depending on whether or not the rock is capable of undergoing a substantial permanent strain without macroscopic fracture. The distinction is not in all respects a sharp one, especially when microscopic processes are also taken into account, but it is nevertheless a very useful one. At the same time, it is important to bear in mind that brittleness or ductility depends in a vital way on the environmental conditions, such as pressure, temperature, and strain rate; the same rock may be brittle under some circumstances and ductile under others.

The present volume deals only with the field of brittle behaviour. The main emphasis will be on the results of laboratory and related theoretical investigations on the mechanical properties of rock under the conditions in which it behaves as a brittle material, although the factors leading to the brittle-ductile transition are also considered in the concluding chapter. In brief, the aim is to review here what we know about rock as a relatively brittle material. In many cases, a broad distinction will be made between compact and porous rocks. The former are often of igneous or metamorphic origin, although some well-cemented sediments also fall into this category, while the latter are often sedimentary rocks. *Compact rocks* are those of very small porosity, less than a few percent, whose properties tend largely to depend on the crack-like nature of their pore space. *Porous rocks* are those with a substantial amount of pore space of more or less equant dimensions, especially as intergranular space; their properties often involve a role of the pore space in the destruction of cohesion between grains or in compaction.

After a preliminary chapter on apparatus, there are two chapters dealing with the stress at which brittle fracture occurs. These chapters treat what may be described as the classical approach to brittle fracture, which regards it as a discrete event and records only the peak stress. The next two chapters deal with the development of events that precede the peak in the load-displacement curve and continue into the post-peak region, and with the theoretical treatment of the brittle behaviour at the micromechanical level. The role of pore fluid pressure on brittle behaviour is then dealt with, followed by a chapter on friction, which is of importance wherever there is sliding on fracture surfaces. Finally, the transition to ductile behaviour with increase in confining pressure and temperature is discussed.

Chapter 2

Experimental Procedures

In the mechanical testing of metals, the applied stress is usually uniaxial, two of the principal stresses remaining zero during a test. However, in the case of rocks, as also in the case of soils, experiments on the mechanical properties are commonly made with all three principal stresses non-zero, and usually compressive. To some extent, this practice has arisen because, with triaxial stress, practical situations involving overburden are more effectively simulated. But, more fundamentally, it reflects the fact that the failure criteria for rock and soil depend strongly on both the normal and shear components of stress, at least where brittle or cataclastic behaviour is concerned. This situation is in contrast to the situation with metals where the yielding depends very little on the mean normal stress. In this chapter, we review briefly the various experimental arrangements that have been used for studies on the brittle behaviour of rocks, mostly at room temperature.

2.1

The Triaxial Test

2.1.1

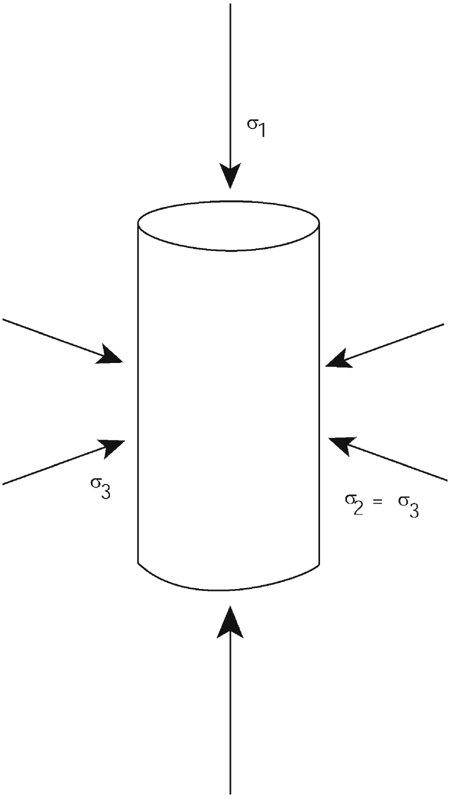
Principle and Terminology

The simplest and most commonly used procedure for achieving a triaxial state of stress in a laboratory test is to superimpose a hydrostatic pressure and a uniaxial stress, that is, in effect to carry out a uniaxial test in an environment of hydrostatic pressure. Such a test is usually referred to simply as a "triaxial test", following the nomenclature of soil mechanics, and we follow this convention here. Of course, this test, more accurately described as an axisymmetric triaxial test, involves only a special case of triaxial stress in which the intermediate principal stress is always equal to one or other of the extreme principal stresses, but the term is widely used without qualification to refer to these conditions. As a result, the term has to be qualified when a general triaxial stress is involved, that is, when all three principal stresses are varied independently; such a general triaxial test is often referred to as a "true triaxial test".

At this point, it is useful to define some further terms and concepts relating to experimental conditions, particularly to triaxial tests. Since, in these tests, cylindrical specimens are subjected to simple compression or extension in the presence of a superposed hydrostatic pressure, the state of stress in the specimen ideally consists of two equal principal stresses normal to the compression or extension axis and a third principal stress parallel to it. Adopting the convention common in geologic and rock mechanics literature that compressive stress is positive, and designating the greatest, intermediate, and least principal stresses as σ_1, σ_2, σ_3, respectively, the stress state in a triaxial compression test is therefore normally assumed to be that shown in Fig. 1. The two equal principal stresses have the same magnitude as the superposed hydrostatic pressure, while the third principal stress departs from it. The superposed hydrostatic pressure is called the *confining pressure* since it is the pressure in the medium surrounding the specimen (note that this is not the same as the mean stress in the specimen, which, in the triaxial compression test, is $(\sigma_1 + 2\sigma_3)/3$).

The amount by which the axial stress component departs from the confining pressure in a triaxial test is commonly called the *"differential stress"*. This jargon term is, in origin, peculiar to the rock mechanics literature, although it has also become widely used in geological literature. It should not be confused with the deviatoric stress or stress deviator, as defined in mechanics (Fung 1965, Ch. 3; Engelder 1994). In the ideal triaxial situation assumed above, the differential stress is identical with the *"stress difference"*, which is the classic term for the difference between the greatest and least principal stresses, $\sigma_1 - \sigma_3$, that is, twice the maximum shear stress (Love 1910). How-

Fig. 1.
System of stresses in the conventional triaxial compression test; σ_3 is the confining pressure



ever, in practice, due to factors such as anisotropy or misalignment of the specimen, the axial stress component may not exactly represent the maximum principal stress. Therefore, we retain the term "differential stress" for use in the direct reporting of triaxial test results but suggest that the term "stress difference" be used when referring to the principal stresses.

The force applied by the loading piston to the specimen in the triaxial test can be resolved into two components, one corresponding to the confining pressure itself and one equal to an additional or *net axial force* (we now prefer the term *"net axial force"* to the term *"differential load"* used in the first edition). The net axial force is the force measured by an internal load cell.

2.1.2

Triaxial Testing Apparatus

The essential elements of a triaxial testing apparatus are shown schematically in Fig. 2. The cylindrical specimen is surrounded by a fluid, commonly liquid in the present applications, which is raised to a high pressure to provide the confining pressure. The confining pressure is normally held constant during the experiment. The piston can be advanced to apply the net axial force to the specimen. The confining pressure and axial force can normally be controlled independently of each other, and means are provided for measuring the axial force and the displacement during the test.

Many practical designs for triaxial testing apparatus have been described in the literature. No attempt is made to review them comprehensively here but several will

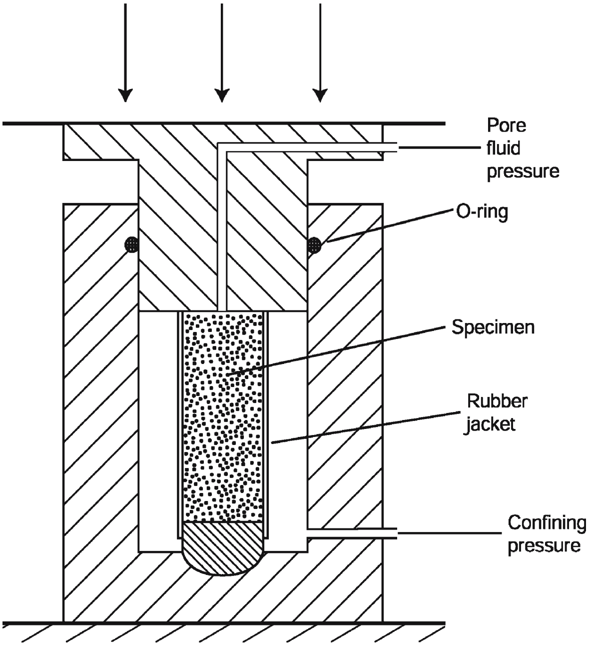
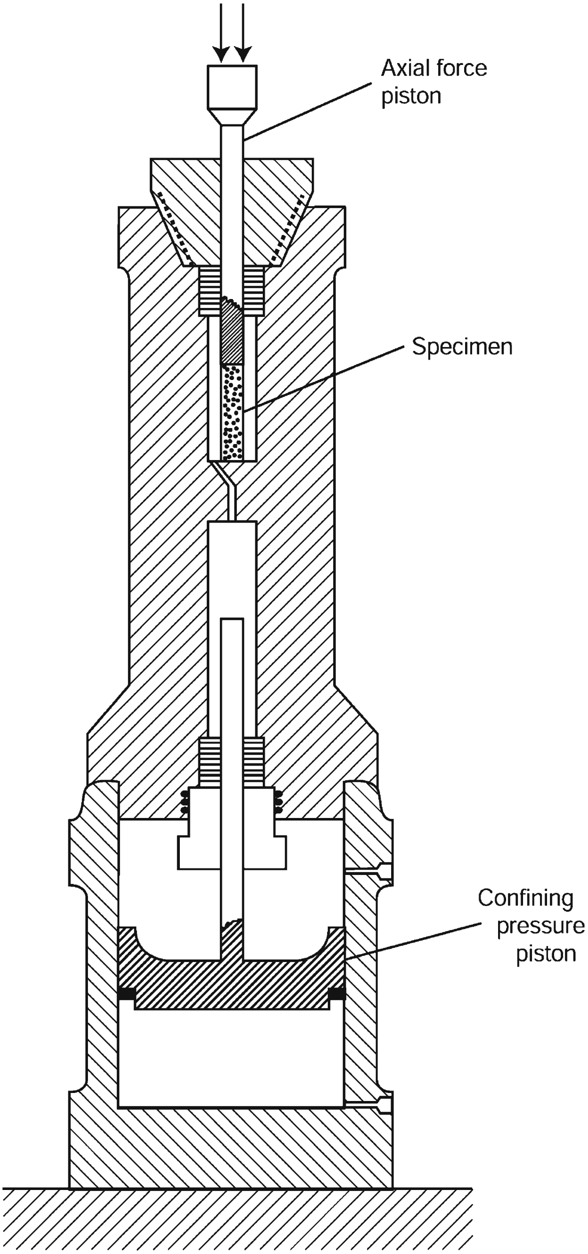


Fig. 2.
Schematic representation of the elements of a conventional triaxial compression testing machine, including optional arrangements for pore fluid pressure

Fig. 3.
Triaxial testing apparatus of von Kármán (1911), slightly simplified



be mentioned to illustrate particular developments. The pioneering apparatus of von Kármán (1911) already contained all the essentials just listed and served as a prototype for later apparatus (Fig. 3). Simple examples are those of the U.S. Bureau of

Reclamation (Balmer 1953), Donath (1966, 1970a), and Hoek and Franklin (1968) (see also Robertson 1955; Brace 1964a; Murrell 1965; Smith et al. 1969).

The first apparatus of Griggs (1936; see also Griggs and Miller 1951) introduced the innovation of using two pistons of equal diameter, connected by a yoke so as to undergo identical movements. Only one of the pistons contacts the specimen and applies force to it while the other performs two compensating functions: (*1*) it maintains constant volume (apart from a small elastic distortion effect deriving from the net axial force) and hence avoids change in confining pressure due to piston movement during straining; (*2*) it counterbalances the piston load due to the confining pressure alone, so that the external loading device only has to apply the net axial force itself plus the friction on the two pistons, which is a much smaller net force than would otherwise be the case when the confining pressure is high. The same arrangement has been used by Paterson (1964b, 1970), and an equivalent arrangement has been used by Donath (1970a). An alternative way of achieving the same end is, using appropriate seals, to connect the pressure fluid to the annular area on a stepped single piston to provide the compensating force (Murrell and Ismail 1976a; Tullis and Tullis 1986; Paterson 1990a), as shown in Fig. 4.

Nearly all triaxial testing apparatus for rock mechanics operate within the 1000 MPa pressure range and, even within this range, most have been restricted to pressures of up to 300–500 MPa. Although pressures higher than 1000 MPa and at least up to 3000 MPa have been used to some extent in triaxial testing in metallurgy (see e.g., Bridgman 1952; Ryabinin et al. 1958; Brandes 1970), the use of such pressures is unusual in rock mechanics at room temperature. Exceptions include the 2000 MPa apparatus of Bergues et al. (1974), an apparatus designed by Heard and used by him and co-workers up to 2200 MPa (Schock, Heard and Stephens 1973), and the cubic press of Shimada (Shimada 1981; Shimada and Yukutake 1982; Shimada 2000).

Carrying out extension tests in triaxial testing apparatus does not generally present special difficulties provided that there are means of making positive attachments to each end of the specimen assembly. Gripping the specimen is very simple if it is jacketed and if the stress difference is numerically smaller than the confining pressure. The tensile net axial force can then in effect be applied to the ends of the specimen by suction relative to the confining pressure when the piston is allowed to move out of the cylinder. All three principal stresses are, of course, still compressive in this case. If net tensile stress is to be applied or if end effects from friction are to be minimized, grips can be attached to suitably shaped specimens such as the "dogbone" specimens used by Brace (1964a; see also Mogi 1966b).

The adaptation of triaxial testing apparatus for use at elevated temperatures is relevant mainly to studies in the ductile field but the influence of temperature on brittle behaviour has been studied to some extent. In brief, for moderate temperatures, up to around 800 K, an external furnace can be used to heat the whole pressure vessel and contents, while for higher temperatures and for greater flexibility in operation the furnace is placed inside the pressure vessel and an inert gas becomes essential as a pressure fluid. For examples of apparatus in these categories, see Griggs et al. (1951), Handin (1953), Griggs, Turner and Heard (1960), Heard (1963), Heard and Carter (1968), Paterson (1970), Goetze (1971), Heard and Duba (1978), Tulls and Tullis (1986), Kohlstedt and Chopra (1987), Durham (1990) and Paterson (1990a).

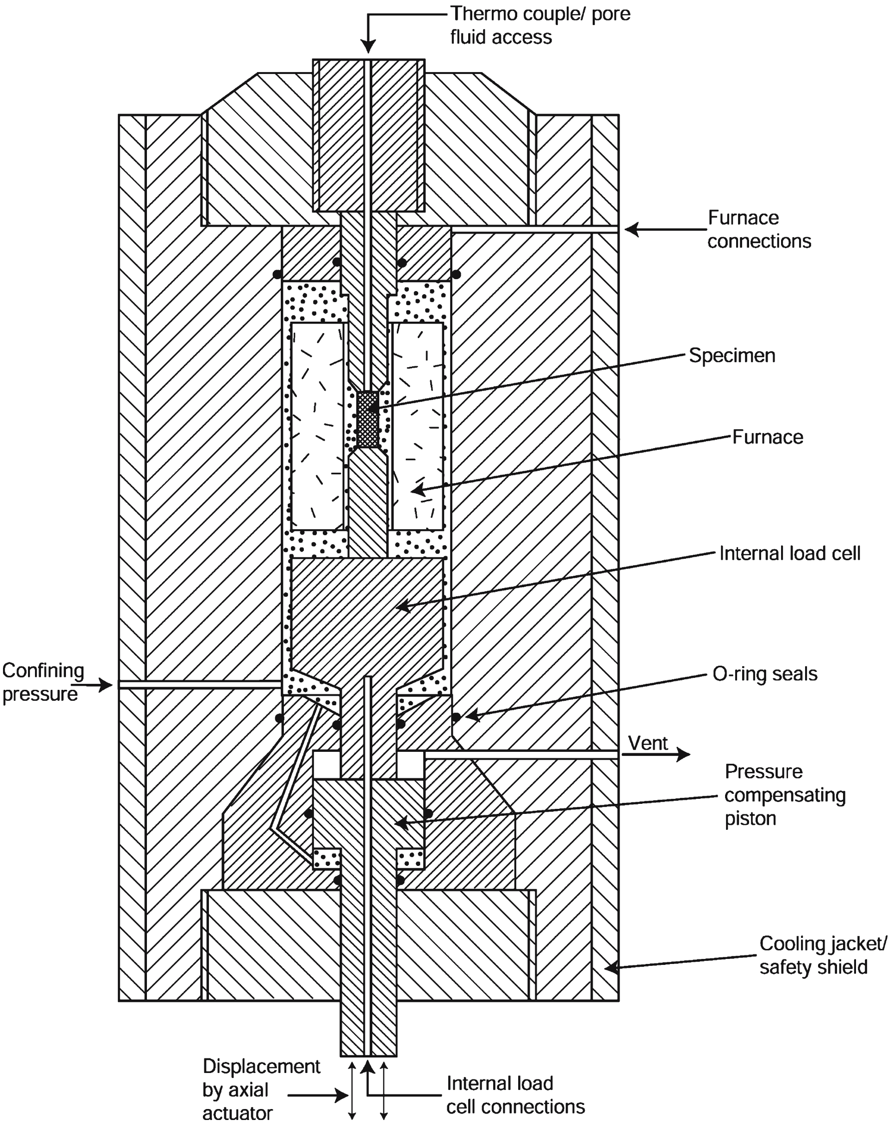


Fig. 4. Schematic representation of a high-pressure high-temperature triaxial testing apparatus, incorporating compensating piston and showing specimen set-up for a compression test. Details of the internal load cell and displacement-measuring arrangements not shown (after Paterson 1990a)

Pore fluid pressure is often an important experimental variable in rock mechanics. Pore fluid pressure can be independently controlled in a jacketed specimen attached to the loading piston if the piston is drilled so that the pore fluid, often water, can be introduced inside the jacket from an independent external pressure generator. It may be desirable to interpose a porous pad between the specimen and the piston to give a better distributed access of the pore fluid to the specimen while preventing extrusion of the specimen, but in the case of strong rocks it is important to use material for the pad that has, at the same time, adequate strength and adequate permeability. Brace and co-workers have used zirconium carbide powder for this purpose (Brace, Orange and Madden 1965; Brace, Walsh and Frangos 1968) but suitable porous ceramic discs are now available commercially. Another problem, discussed in Chapt. 7, is that a long period of time is required for achieving pressure equilibrium within the pore space in a rock of very low permeability. In some experimental situations, a hollow piston open to atmosphere is useful for ensuring that the pore pressure is zero; this expedient can be especially useful in cases where there is a question of whether effective jacket sealing is being achieved.

There are many other refinements or modifications in triaxial testing apparatus that have been introduced for special purposes and which will be dealt with where appropriate in later chapters. These innovations include devices for measuring dilatancy or velocity of elastic waves and the use of servo-controlled or especially stiff machines. Another optional feature, common even in the simplest apparatus, is the use of a spherical seat in compression testing to help ensure better initial concentricity of loading. Safety in operation should also not be overlooked and adequate shielding of highly stressed parts is important. Even with liquids, quite high stored energy is involved at a few hundred megapascals and failure in hardened pistons or tubing connections can give rise to dangerous missiles. A code of safe practice has been published by the High Pressure Technology Association (Cox and Saville 1975).

The design of pressure vessels has been the subject of considerable study, although some of the problems relate to larger vessels than are normally used for laboratory triaxial apparatus; see, e.g., Davidson and Kendall (1970), Turner (1974), Mraz and Nisbett (1980), Bednar (1985), Bonn and Haupt (1995) and the proceedings of two conferences organized by the Institution of Mechanical Engineers (1968, 1977). References to the problem of fatigue life of pressure vessels will also be found in the latter proceedings.

A pressure fluid commonly used at room temperature is kerosene but many other low viscosity liquids are also suitable. The viscosity of kerosene becomes very high as 1000 MPa is approached, and if low viscosity is important lower hydrocarbon mixtures such as petroleum ether or even pure isopentane can then be used. Tables of the influence of pressure on viscosity of lubricating fluids published by the American Society of Mechanical Engineers (1953) may be used in selecting pressure fluids of suitable viscosity. Some liquids can also be used at moderately high temperatures, especially under high pressure; thus, Handin (1953), Yukutake and Shimada (1995) and Ohnaka et al. (1997) used silicone fluid and perfluoro-polyether oil up to 773 K.

Many types of commercially available pumps are suitable for generating pressures of up to the order of 100 MPa or so. Compressed air-driven pumps can be especially convenient and versions of these are available with which much higher pressures can be reached. However, intensifiers have usually been used for the highest pressures, backed up by a suitable priming pump. For pressure measurement, Bourdon gauges or pressure gauges based on sensing the distortion of a container by means of strain gauges or displacement transducers are available commercially for pressures up to at least 700 MPa. However, manganin resistance gauges are convenient and still widely used over the whole pressure range (for use of manganin and other gauges, see Babb 1970; Peggs 1983).

Conventional seals of many types from commercial hydraulic practice can be used at lower pressures, at least below 100 MPa. For the higher pressures, the "unsupported area" seal of Bridgman (1949a) has been widely used in the past for both closure and piston seals and various modifications of it have also been used (e.g., Heard 1963). Lower friction can be achieved with the "controlled clearance" seal (Handin 1953; Johnson and Newhall 1953; Newhall 1957) and with O-ring seals. Conventional O-ring seals are especially convenient to use both for static and piston seals but anti-extrusion rings are needed at high pressures. Examples of reliable O-ring piston seal design are shown in Fig. 5. The choice of an O-ring compound with good low temperature properties helps to avoid brittle failure of the O-ring above the glass transition (Paterson 1964a).

2.1.3
Preparation and Jacketing of Specimens

Specimens are usually obtained by diamond core drilling, the thin-walled glass-working drills being especially convenient and economical for this purpose. A drilling machine with minimal vibration is desirable for reducing breakage of cores. The ends of the cores can be squared off with abrasive paper or by diamond grinding, as appropriate, using a V-block or similar jig to aid in getting the ends square to the specimen axis. It is important that the ends be parallel within a small tolerance, say, 0.01 mm or less. In geotechnical applications, it is often important to preserve the original water content, which may require special procedures (Brown 1981). In geophysical applications, it is more often important to test under controlled moisture conditions, such as

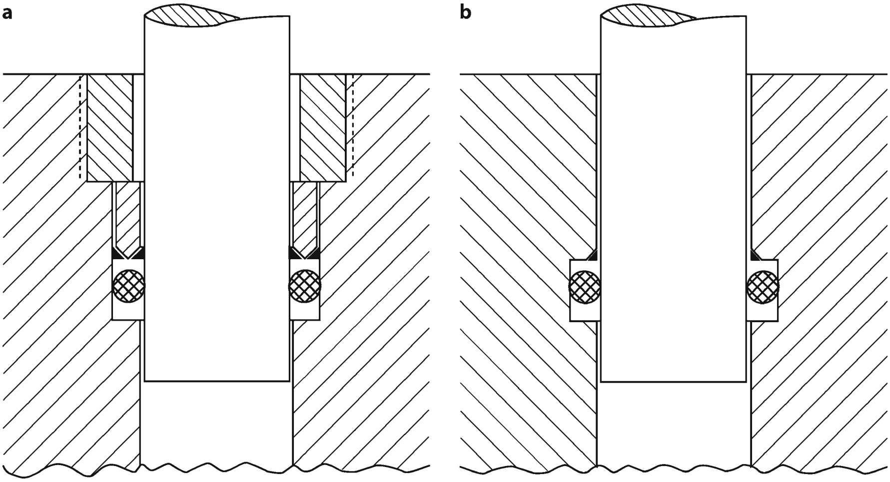


Fig. 5. Examples of O-ring seals with anti-extrusion rings, suitable for piston seals. **a** With separate retaining sleeve, after Paterson (1962); **b** simpler version, suitable when the diameter is large enough that the anti-extrusion ring can be flexed into position

saturated or dry. Although air drying is often used, a more thorough or controlled drying, such as in a vacuum oven, is good practice. For a study of the importance of the various factors in specimen preparation, see Podnieks et al. (1972).

The jacketing of the specimen in an impervious jacket is a very important factor in the experimental conditions in triaxial testing of rocks. The jacket prevents the pressure fluid from entering pores or cracks and its use greatly influences the strength. In general terms, its action is to enable the confining pressure effectively to inhibit the opening of cracks or sliding on existing crack surfaces. At room temperature, tubing of rubber, polyurethane, or similar materials is widely used for jacketing, the ends being sealed by devices such as a hose-clip or a stretched ring of the same material, or "heat-shrink" tubing can be used. Although these types of jackets are very convenient, it should be realized that elastomers may become quite strong and apply appreciable constraint to the specimens if the glass transition pressure (commonly 200–500 MPa) is exceeded (Paterson 1964a; Weaver and Paterson 1969a,b). Therefore, even at room temperature, jacketing is often better done with thin-walled tubing of metal such as copper or indium, sealing the ends with rubber tubing or an O-ring (Donath 1966, 1970a). At elevated temperatures, metal jackets can be sealed with press-fitted rings of the same material as the pistons (Handin 1953), or a long jacket covering also the loading pistons can be used, sealed with O-rings at the cool ends of the pistons (Paterson, Chopra and Horwood 1982).

2.1.4
Measurements, Control, and Data Reduction

The two primary measurements to be made during a triaxial test at constant confining pressure are those of net axial force and axial displacement, from which a plot of differential stress versus axial strain can be derived. In some applications, the change of volume of the specimen is also measured.

For measurement of the force externally to the pressure vessel a suitable load cell, chosen from the wide range commercially available, can be used, or the force can be obtained by measuring the oil pressure in the loading jack. These approaches are, however, only satisfactory if the net axial force is large compared with the piston friction, for which correction must be made. Otherwise, an internal load cell is desirable since friction can be notoriously variable or unpredictable and the corrections correspondingly inaccurate.

A suitable internal load cell can be made from an elastic element to which electric resistance strain gauges are attached, as first used by Haasen and Lawson (1958; see also Pugh and Gunn 1964; Mogi 1965; Davis and Gordon 1968; Paterson 1970), although Bridgman (1952) measured the change in resistance of the element itself. Alternatively, some form of displacement transducer can be used to measure the distortion of the elastic element (Robertson 1955; Heard 1963; Gelles 1968; Paterson 1990a), although some of these arrangements have the disadvantage of being fairly sensitive to changes in confining pressure as well as to the net axial force. In the case of the first type, attachment of electric resistance strain gauges with epoxy resin tends to be short-lived at the higher pressures because of the difference in compressibility of the resin and the metal substrate; this difficulty can be reduced by the use of ceramic strain gauge cements or sprayed coatings normally intended for high-temperature use. The chief problem in achieving high precision with internal load cells is to calibrate them under pressure in order to allow for any small effects of the confining pressure that there may be on their sensitivity – for some methods, see Pugh and Gunn (1964), Davis and Gordon (1968), Gelles (1968), Paterson (1970, 1990a), and Fung (1975).

The axial strain in the specimen is commonly obtained from measurements of piston displacement made outside the pressure vessel and corrected for apparatus distortion. The linear variable differential transformer (LVDT or DCDT) is very convenient for this purpose. Electric resistance strain gauges can be used on the specimen itself (see, e.g., Brace 1964a,b; Schock and Duba 1973) or other types of strain transducers devised (Serdengecti and Boozer 1961; Smith et al. 1969; Hardy and Kim 1970; Fung 1975; Tullis and Tullis 1986; Brown and Heuze 1979; Spetzler 1987).

Measurement of volume change that occurs in response to the application of both hydrostatic and deviatoric stress is also often of interest, especially in porous rocks. Procedures for measurement of volume changes are discussed in Chapt. 5.

The simplest and commonest way of carrying out a test is to operate the machine at a constant displacement rate (or constant drive-motor speed), except in simple creep tests where constant force is maintained after the initial loading. However, servo-control loading systems were introduced in the 1970's (Houpert 1970; Rummel and Fairhurst 1970; Hudson, Brown and Fairhurst 1971; Hardy, Stefanko and Kimble 1971; Rummel 1975) and are now in general usage. In these systems, the property of specimen behaviour to be controlled, for example, the axial strain, is measured by a suitable transducer, whose signal is monitored by the control system; the control system, in turn, adjusts the driving device in such a way as to make the signal follow a programmed course. This course may be a set strain rate or loading rate, a constant force or strain, or any other desired course such as a cycling force. The application of such systems to the study of the later stages of fracture development is discussed in Chapt. 5.

The axial force and displacement data are now commonly acquired and reduced digitally, but a visual output on a strip-chart recorder may also be desirable for following the progress of the test. For early application of computers, see Donath and Güven (1971), Güven and Donath (1971) and Hardy et al. (1971).

In reducing the data the following steps are usually involved:

- *Strain*: The measured piston displacements are corrected for apparatus distortion, which is normally proportional to the force, although non-linearity may have to be taken into account at small forces. The strain is then commonly calculated as $\varepsilon = \Delta l / l_0$ where l_0 is its initial length and Δl is the decrease in length of the specimen (under the convention of shortening strain being positive). If the strain is large, the "natural strain" $\varepsilon = \ln(l_0/l)$ is often used, where l is the current length.
- *Stress*: If an external load cell is used, the friction correction has to subtracted from the recorded force; this correction is usually obtained from observations made during piston advance before the specimen is contacted, neglecting further friction associated with the axial force. Any force known to be supported by the jacket must also be subtracted from the measured force. At small strains, the differential stress is calculated from the corrected force, using the initial cross-sectional area of the specimen; a "true stress" is more appropriately calculated using the current cross-sectional area (usually assuming that the specimen is deforming at constant volume and remaining cylindrical).

2.2
Other Types of Tests

The *uniaxial compression test* is widely used in engineering practice and is also often used in research. It has the advantage of being simple in principle, and of requiring minimal sophistication in equipment. A further advantage is that it most readily allows the carrying out of auxiliary measurements, for example, of lateral strain, dilatancy, or velocity of elastic waves. New potentialities in the use of the uniaxial compression test have also been opened up recently with the advent of servo-control (Sect. 2.1.4 and 5.2.1). For a comprehensive review of the uniaxial compression test, see Hawkes and Mellor (1970) and Vutukuri et al. (1974).

The *uniaxial tension test*, the most common test in metallurgy, is used only minimally for rocks. A major reason for its avoidance lies in the difficulty of gripping relatively short specimens in such a way that reproducible behaviour can be achieved. For methods of overcoming this difficulty, see Fairhurst (1961), Hardy and Jayaraman (1970), Hawkes et al. (1973), Barla and Goffi (1974), and Okubo and Fukui (1996). More commonly, indirect testing methods involving inhomogeneous stress states are applied in attempting to characterize the tensile strength of rock in a reproducible way.

The main use of inhomogeneous stress tests is, in fact, that of obtaining data on the tensile strength of rocks. There are many such methods based variously on bending, torsion, indentation, diametral compression, hoop-stress loading from internal pressure in hollow cylinders, etc., often involving application of line loads or point loads. For an introduction to these test methods and for further references on them, see Jaeger (1965, 1967), Jaeger and Cook (1979, p. 160 et seq.), Hardy and Jayaraman (1970, 1972), Mellor and Hawkes (1971), Broch and Franklin (1972), Barla and Innaurato (1973), Lawn and Wilshaw (1975a), Cox and Scholz (1988), Santarelli and Brown (1989), Ewy and Cook (1990a,b), and Suárez-Rivera et al. (1990).

The *shear test* is sometimes used in rock mechanics in a form similar to that common in soil mechanics (Lambe and Whitman 1969); the machine of Obert et al. (1976) is such an example but with arrangements for varying its normal stiffness. However, the main application of the shear test has been for friction or joint studies (e.g., Goodman and Ohnishi 1973; Ross-Brown and Walton 1975; Dieterich 1972a; Dieterich 1981; Linker and Dieterich 1992; Cox 1990). The disadvantage of inhomogeneous stress in the simple shear test can be reduced by the use of torsion of hollow cylinders (Christensen, Swanson and Brown 1974; Durand and Comes 1974; Kutter 1974; Durand 1975; Tullis and Weeks 1986; Biegel et al. 1992; Beeler et al. 1996). For an analysis of the stress conditions and other aspects of the torsion test, see Paterson and Olgaard (2000).

For "true" triaxial testing, independent loading of the three pairs of opposite faces of a cubic specimen is usually used. Severe difficulties are encountered in reducing friction at the loaded faces to obtain a good approximation to homogeneous stress, but considerable progress has been made. References are listed in the further discussion of such tests in Sect. 3.6.

There are also various tests in which the specimen's dimensions are constrained in some degree. The simplest of these is the *uniaxial strain test* in which the specimen is constrained to retain the same diameter during deformation with the intention of simulating certain situations in the Earth (for examples of studies in uniaxial strain, see Swanson and Brown 1971b; Brace and Riley 1972; Walsh and Brace 1972; Schock, Heard and Stephens 1973; Spiers et al. 1990; Teufel, Rhett and Farrell 1991). It follows that the volume changes significantly during such a test. Another case is that of the *biaxial strain test* in which the specimen is constrained to undergo plane strain (for example, Maso and Lerau 1980; Ord, Vardoulakis and Kajewski 1991; Labuz, Dai and Papamichos 1996; Yumlu and Ozbay 1995).

Chapter 3

Experimental Studies on the Brittle Fracture Stress

3.1
Introduction

In the simplest view, the brittle fracture of a rock is a discrete event in which the failure of the rock occurs, without significant prior deformation and without warning, at a particular stress. In later chapters it will be shown that this view is oversimplified and is, in particular, inadequate for an understanding of the physical mechanisms of fracture. However, it is often sufficient from a phenomenological point of view, and it is a view that has underlain a great deal of research on the factors that affect the gross brittle behaviour of rocks. The present chapter and the following one deal with these studies, in which, apart from noting the gross nature of the fracture, the only quantity of interest is the peak stress ("brittle fracture stress"). Such studies, often essentially empirical in approach, provide much of the basis for applied rock mechanics in engineering and mining, as well as for the analysis of geologic faulting.

This chapter deals with experimental studies on the various factors (except pore pressure) that affect the brittle fracture stress and the gross nature of the fracture. Fracture is described as "brittle fracture" when it is not preceded by any appreciable amount of permanent deformation, although the microscopic and other pre-failure observations described in Chapt. 7 show that the behaviour is by no means purely elastic prior to the macroscopic fracture. There is, indeed, some latitude in defining what is a brittle fracture. Sometimes it may refer to failure following amounts of inelastic strain that are small compared with the elastic range; in other cases, fracture following several percent of inelastic strain is still taken as brittle (e.g., Heard (1960) defined fractures as brittle if the strain did not exceed 3% prior to the fracture).

Many types of tests have been used in the experimental study of brittle fracture in rocks. We shall consider mainly the results from the uniaxial compression test and the conventional triaxial test, described in Chapt. 2. These tests have the advantage of being the simplest to interpret because, at least in principle, they involve only homogeneous stress. Tests such as torsion, bending, and diametral compression ("Brazilian") tests involve inhomogeneous stress distributions, and their interpretation then tends to rest on the generally dubious assumption that the stress distribution prior to fracture corresponds to that of the purely elastic state. Some of the latter tests have advantages of practical convenience, as in assessing the relative tensile strengths of different rocks at atmospheric pressure, but we shall not consider them in detail here (for references, see Sect. 2.2).

3.2
Types of Fractures

It is useful, following Griggs and Handin (1960), to distinguish two principal modes of brittle failure (Fig. 6):

1. Shear fracture, in which the relative displacement is parallel to the fracture surface, the latter being generally inclined at an angle of less than 45° to the maximum compressive principal stress.
2. Extension fracture, characterized by separation normal to the failure surface, which is generally oriented normal to the least principal stress (compression positive).

The *shear fracture* is the dominant mode of macroscopic brittle failure in triaxial compression tests at all but the lowest confining pressures. Shear fracture also predominates in triaxial extension tests above a higher threshold of confining pressure. In general, the fractures in both cases occur at a fairly well-defined angle to the direction of the greatest compressive principal stress, usually between 20° and 30°. However, there is often a tendency for the angle to increase slightly with increase in confining pressure (e.g., Paterson 1958). Also, in extension tests it has been shown that there tends to be a gradual change in orientation of the failure surface in the transition from extension fracture to shear fracture with increasing confining pressure in the transition range (Brace 1964a; Hoskins 1969b).

As the brittle-ductile transition is approached with increasing confining pressure, the shear failure tends to become a zone of intense deformation and fine-scale frac-

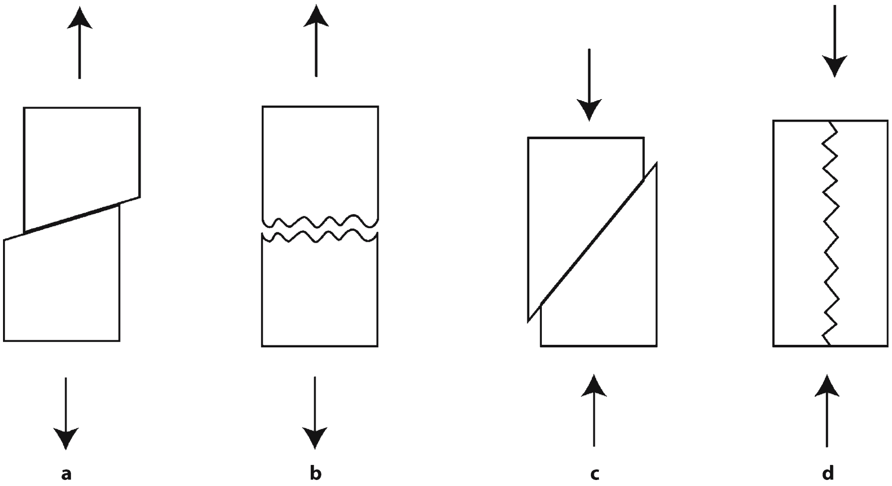


Fig. 6. Types of fractures. **a** Shear fracture in extension test (least principal stress vertical, compression positive); **b** extension fracture in extension test; **c** shear fracture in compression test (greatest principal stress vertical); **d** extension or "axial splitting" fracture in compression test

turing, rather than a discrete shear fracture, and separation may not then take place readily at the failure zone after the test. The term "fault" is sometimes applied to shear failures to cover both the discrete shear fracture and such a shear zone of concentrated cataclasis, but Jaeger and Cook (1979, p. 92) have suggested that this term be restricted to geologic use. At confining pressures near the brittle-ductile transition, conjugate shear failures sometimes occur, but this effect seems to depend on there being constraint on rotation of the ends of the specimen; thus, single shears have been observed when a spherical seat was used and two conjugate shear, or more, with rigid end plattens under otherwise identical conditions (Paterson 1958; Jaeger 1960a).

More detailed discussion on the genesis and development of shear failures will be given in Chapts. 5 and 9. The remainder of this section will deal only with the topic, sometimes rather enigmatic, of extension fractures.

The *extension fracture* is best known from its occurrence in the uniaxial tension test as a parting of the specimen normal to the direction of the tensile stress. However, extension fractures can also occur under predominantly compressive conditions, as is observed in several types of tests. In the classic "embracing" or "pinching-off" experiment of Föppl (1900) and Bridgman (1912, 1952), a cylindrical rod of brittle material is loaded by fluid pressure on the cylindrical surface while the free ends project from the pressure vessel and are restrained only by the friction at the packings; the failure is usually by extensile fracturing normal to the axis of the rod (see also Gurney and Rowe 1948a, 1948b; Jaeger and Cook 1963; Secor and Montenyohl 1972). Extensile fractures are also commonly observed in triaxial tests in extension when all the macroscopic principal stresses are compressive (Handin 1953; Heard 1960; other references given by Griggs and Handin 1960), and they have been observed in confined Brazilian tests under similar circumstances (Jaeger and Hoskins 1966a). Further, in compression tests at or near atmospheric pressure, there frequently occur extension fractures in the form of splitting parallel to the compression axis, analogous to the fractures observed in biaxial compression by Föppl (1900).

The apparent paradox of extension fractures occurring in the absence of macroscopic tensile stress has given rise to a good deal of speculative discussion. One view is that the fracture occurs when a certain critical extensile strain is exceeded (maximum extensile strain criterion of failure, e.g., Bridgman 1938). However, this is not a very satisfying explanation from a physical point of view, although Bridgman has attempted to defend it against such criticism. In terms of physical processes, an extension fracture presumably must involve failure of bonds under local tensile stress, even if this only occurs at the molecular or atomic scale (cf. Gurney and Rowe 1948a,b). It seems most likely that the fractures originate in response to local tensile stresses around flaws or cracks on a microscopic scale, arising as described below in relation to axial splitting (see also Scholz, Boitnott and Nemat-Nasser 1986).

The nature and magnitude of local stresses around microscopic flaws will be strongly affected by whether fluid or other material under pressure has access to them. This consideration leads to the distinction, emphasized by Brace (1964a), between *intrusion fractures* and *internal fractures*. Intrusion fractures are said to occur when the surrounding material (pressure medium, jacket material, or even a wedge-shaped fragment of the specimen itself) penetrates cracks that are open to the surface. Such fractures can therefore occur when all the macroscopic principal stresses are compressive, and they probably account for many of the extensile failures observed under these conditions. In contrast, internal fractures can only be expected when there is a net tensile component of stress in the region through which the crack is propagating, arising independently of the injection of material. The essential difference between intrusion and internal fractures, therefore, lies in whether or not normal forces act on the crack faces after separation.

The "axial splitting" or "axial cleavage" fracture is common in uniaxial testing but it is suppressed by the addition of relatively low confining pressures, even by a few megapascals in some cases (Griggs 1936; Paterson 1958; Heard 1960; Jaeger 1960a; Wawersik and Fairhurst 1970; Wawersik and Brace 1971). The axial splitting probably normally originates from local transverse tensile stresses at flaws or heterogeneities in the material. Thus, in uniaxial compression tests, Wawersik and Fairhurst (1970) have noted a reduced tendency to axial splitting in the more nearly homogeneous or finer-grained materials such as lithographic limestone.

The manner in which local transverse tensile stresses can arise at flaws is illustrated in the two-dimensional analysis of stresses around a circular hole in an elastic medium (Jaeger and Cook 1979, pp. 249–252). This analysis (Fig. 7a) shows that, under a uniform uniaxial loading applied at a large distance from the hole, there is not only an intensification of the axial compressive stress component σ_x in the neighbourhood of the end of the transverse diameter of the hole but there arises also a transverse tensile stress σ_y, of peak magnitude equal to that of the mean compressive applied stress, in the neighbourhood of the end of the diameter parallel to the direction of loading. A qualitatively similar situation arises at an elliptical hole or crack with the major diameter parallel to the uniaxial compressive loading direction (Jaeger and Cook 1979, pp. 266–277). This transverse local tensile stress component is reduced or completely suppressed by the addition of a compressive transverse loading of the body (e.g., by

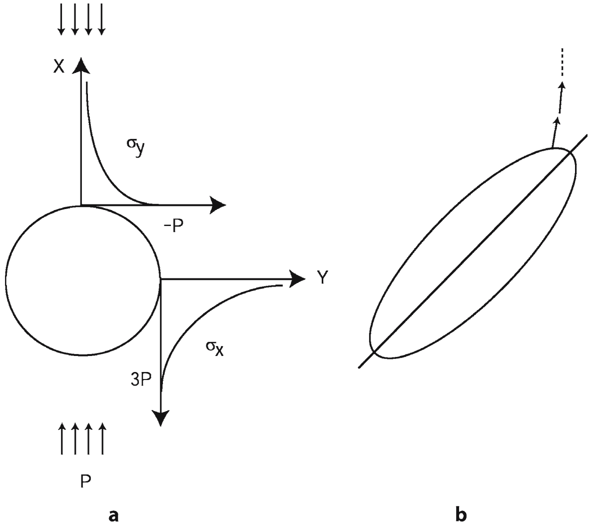


Fig. 7.
a Distribution of stresses around a hole in uniaxial compressive loading (two-dimensional case); σ_y is the lateral stress at points along the x axis, σ_x the longitudinal stress along the y axis, and P the average applied compressive stress. **b** Direction of propagation of a crack from an inclined flaw under compressive loading applied vertically

applying a confining pressure). Thus, qualitatively we can expect local transverse tensile stresses to arise at the ends of any axially oriented flaws in brittle rock specimens if the confining pressure is not too high, thus favouring propagation of a crack parallel to the specimen axis from the flaw. When the flaws are inclined to the specimen axis, the maximum tensile stress component no longer occurs at the end of the major diameter but at a point where the normal to the flaw surface is more nearly parallel to the specimen axis. Then the flaw will tend not to propagate in its own plane but to swing into an orientation more nearly parallel to the axis of loading, as shown in Fig. 7b (Brace and Bombolakis 1963; Bombolakis 1964; Hoek and Bieniawski 1965). More detailed analysis in the context of the wing crack model and linear elastic fracture mechanics will be discussed in Chapt. 6 and Appendix 1.

The local, induced tensile stresses of the type just described have therefore been invoked to explain why cracks often propagate parallel to the direction of compressive loading, for example, by Gramberg (1965, 1970), Fairhurst and Cook (1966) and Holzhausen and Johnson (1979). Somewhat related notions are found also in the explanations of Ros and Eichinger (1949), Trollope and Brown (1966), Brown and Trollope (1967), Trollope (1968) and Karihaloo (1979). A microdynamical theory of axial splitting has been given by Nemat-Nasser and Horii (1982) and by Horii and Nemat-Nasser (1985, 1986); see Chapt. 6. An energy-based model of compressive splitting was also proposed by Bhattacharya (1998).

3.3
Observed Stresses at Brittle Failure

There is no universal simple law governing the level of stress at which a given rock fractures because, (*1*) this level involves all principal components of the stress and, (*2*) it probably depends on the mode of fracture. To express the failure conditions in the most general way, it is usual to seek an appropriate function of the three principal stresses σ_1, σ_2, σ_3, which takes on the same characteristic value for any combination of the principal stresses at which fracture occurs. This condition, often for convenience written in the form

$$\sigma_1 = f(\sigma_2, \sigma_3)$$

is known as a *criterion of failure*. The function f includes at least one parameter, the value of which is characteristic of the particular material. However, it cannot be assumed that a given simple form of the function f will necessarily apply to more than one particular mode of fracture unless it is shown that the underlying physical mechanisms are the same; in particular, it is possible that extension and shear fractures may be controlled by different criteria of failure.

The question of establishing criteria of failure for rocks under general conditions of stress will be taken up later in this chapter. Here, we shall only discuss the stresses at macroscopic failure in the situations most commonly dealt with experimentally. These involve extension fracturing and shear fracturing in the conventional uniaxial and triaxial tests, and so the immediate interest centres on the uniaxial strengths and on their modification by the addition of the confining pressure.

It is not feasible to present here a compendium of all existing room temperature data from uniaxial and triaxial tests, so we shall only present a broad summary of the trends. Important compilations of data have been given by Hirschwald (1912, pp. 75–82), Ros and Eichinger (1949), Balmer (1953), Wuerker (1956, 1959), Handin (1966, esp. Tables 11-3 to 11-5), Lama and Vutukuri (1978), Hoek and Brown (1980), and Singh (1981). Useful summary plots including some of the same data have been given by Hoek and Bieniawski (1965); see also Jaeger and Cook, (1979, p. 154), Mogi (1966a), Ohnaka (1973b), Hoek and Brown (1980), and Lockner (1995). Measurements on sedimentary rocks (mostly arenaceous and argillaceous) has been published by Handin and Hager (1957, 1958), Hoshino et al. (1972), Hoshino (1974), and Vernik et al. (1993); see also Jones and Preston (1987). Very broadly, the compressive fracture stresses for various rocks at room temperature can be categorized as:

1. *Igneous and high-grade metamorphic rocks*: uniaxial compressive strengths generally in the range 100–200 MPa, sometimes higher, especially in fine-grained rocks; at a confining pressure of 100 MPa, differential stresses at fracture often around 500–800 MPa, sometimes higher; at a confining pressure of 500 MPa, differential stresses at fracture usually above 1000 MPa and sometimes above 2000 MPa.
2. *Low-porosity sedimentary and low to medium-grade metamorphic rocks* (including calcite limestones and marbles): uniaxial compressive strengths in the range 50–100 MPa; at a confining pressure of 100 MPa, differential stresses at fracture of 200–300 MPa, if still brittle.
3. *High-porosity sedimentary and some low-grade metamorphic rocks*: uniaxial compressive strengths of 10–50 MPa.
4. *Low-porosity dolomites and quartzites*: very high uniaxial compressive strengths, up to 300 MPa or more; at 100 MPa confining pressure, differential stress at fracture in range 500–1000 MPa, or even higher in quartzite.

The uniaxial tensile strengths tend to be of the order of one-tenth of the uniaxial compressive strengths, but in triaxial tests the differential stresses for shear fracturing in extension are of a similar order of magnitude to those in compression.

The strength variations above reflect many factors that cannot be analysed in detail in a general treatment. These factors include mineral composition, porosity, state of alteration or weathering, and prior history affecting microstructural details such as the density and distribution of microcracks. For systematic studies on the influence of porosity, see Dunn, La Fountain, and Jackson (1973), Hoshino (1974), Friedman (1976), Logan (1987), Scott and Nielsen (1991), Vernik, Bruno and Bovberg (1993), Hatzor, Zur and Mimran (1997) and Wong, David and Zhu (1997).

We shall now summarize the broad trends that have been deduced for the macroscopic fracture stress as influenced by variation in the confining pressure, a variation that corresponds to variation in the ratio of the extreme principal stresses. We distinguish between the extension and shear modes of fracturing and, for the present, only isotropic behaviour is considered.

Extension fracturing is often said to occur when the least principal stress (compressive stresses reckoned positive) is equal to the uniaxial tensile strength (e.g., Murrell 1967a; Hoek 1968). The most important data for rocks in support of this con-