Laboratory and Field Testing of Rock Masses for Civil Engineering Infrastructures

José Muralha and Luís Lamas

Abstract Several types of relevant civil engineering infrastructures, such as the foundations of large buildings, bridges and dams, rock slopes, tunnels and caverns, encompass construction of structures on or in rock masses. Rock masses, specifically those within a few hundreds of meters from the surface where civil infrastructures are implanted, being composed of intact rock and discontinuities (e.g., faults, joints, schistosity and bedding planes), often behave as discontinuum media, with the latter determining their behaviour. The assessment of rock mass properties and conditions is crucial for the design of rock engineering structures, and for assuring safety during their life-time exploration. Since the development of rock mechanics as a distinct engineering discipline in the 1950s and early 1960s, the importance of laboratory rock testing emerged. Additionally, the recognition that tests on small size specimens could not be representative of the behaviour of the rock mass led to the emergence and development of specific in situ tests, where comparatively large rock mass volumes are tested in order to estimate engineering properties suitable for design. This chapter presents laboratory and in situ tests currently used to estimate the relevant parameters required to model the behaviour of rock mass—a naturally occurring material with unknown in situ stresses—at a scale compatible with the dimensions of engineering infrastructures.

Keywords Rock mass · Intact rock · Discontinuity · Laboratory testing · In situ testing

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1 Introduction

Rock mechanics is a discipline that applies mechanics principles to rocks and is used to design and monitor structures built on or in rock masses, such as dams, large bridges and buildings, natural and excavated slopes, tunnels, caverns, hydroelectric schemes, nuclear repositories, or mines.

Throughout this chapter, the terms rock mechanics and rock engineering will be used in a sense as defined by the International Society of Rock Mechanics and Rock Engineering (ISRM): "*The field of rock mechanics and rock engineering includes all studies of the physical, mechanical, hydraulic, thermal, chemical and dynamic behaviour of rocks and rock masses, and engineering works in rock masses, using appropriate knowledge of geology*". As a consequence, rock mechanics is generally taken to include rock engineering, though occasionally both terms may be used separately, since rock mechanics is the key for dealing with many problems met in rock engineering projects.

As opposed to common man-made materials used in engineering projects, such as steel or concrete, rocks and rock masses are historical materials that during geological times have gone through quite long history of natural phenomena, being acted on chemically, thermally and mechanically, and undergoing deformation, fracture and weathering. Even at a smaller scale, intact rock is a bonded or cemented aggregate of grains, generally individual crystals or amorphous particles from different minerals, but rarely do not include inter or intragranular cracks. At a rock engineering scale, rock mechanics deals with rock masses, which are media where discontinuities, anisotropy and heterogeneity are nearly always present requiring particular approaches.

Recognition that rock masses are particular media not covered by continuous mechanics led to the seminal reply by Leopold Müller to the question "*Do we know the strength of rock?*". Müller replied: "*For rock (specimens) tested in the laboratory, yes. For a rock mass, no.*" [1]. Though engineering properties of rocks were already being studied all around the world, it commonly acknowledged that Rock Mechanics emerged as an independent discipline at that time [2].

Regrettably, the beginning and the early development of rock mechanics is also related to the occurrence of three catastrophic events: the failure of the foundation of the Malpasset concrete arch dam, in December 1959 (Fig. 1, left) [3, 4], the collapse of the coal mine pillars at Coalbrook, in January 1960 [5], and the landslide of the left bank of Vajont dam reservoir, in October 1963 (Fig. 1, right) [6, 7]. These serious accidents led to understanding that discontinuities, regardless of their origin, play a significant role in the behaviour of rock masses as their reduced shear strength may convert a sound rock masses into a crumbling block system for stresses acting along particular orientations. They also triggered much debate, new research and promoted the development of new tests and methodologies to assess rock mass properties.

In the 1950s, construction of large concrete dams and underground caverns and tunnels for hydroelectric schemes were seeing a notable expansion worldwide.

Fig. 1 Malpasset dam failure (**left**), and Mont Toc (Vajont) landslide (**right**)

Though rock mechanics tests already were an important component in the investigations that supported the design of these structures, the improvement of existent rock testing methods and the development of new experimental testing methods and techniques was also related to the emergence rock mechanics as an autonomous discipline within the geomechanics framework, encompassing a distinct body of knowledge. At that time, novel in situ testing methods started being developed at the Portuguese National Laboratory for Civil Engineering (LNEC), under the leadership of Prof. Manuel Rocha [8]. This chapter will make reference to the authors' experience in this subject, while mentioning relevant technological updates and alluding to other worth mentioning testing techniques.

2 The Relevance of Testing in Rock Mechanics

It is accepted that any structural engineering design comprises some kind of modelling of the physical, mechanical, or hydraulic behaviour of the components involved in the construction. In the design of structures to be built on or in rock masses—a natural, discontinuous, heterogeneous, anisotropic, often highly variable material—the behavioural models depend critically on the input parameters, namely their deformability, strength, permeability and boundary conditions (i.e. natural in situ stresses).

Current developments in computing capabilities, that have allowed the proliferation and availability of numerical analyses, have led to more and more elaborated models, as well as to an increasing demand of a better understanding of the mechanisms occurring in rock masses, once they are disturbed by natural actions or by new man-made structures. These requirements make the need of rock testing an always current topic.

In rock engineering, the behaviour of rocks and rock masses concerns mainly the following properties: deformability and strength, and how they vary with the direction and magnitude of the loads, permeability, susceptibility to weathering, and the natural in situ stresses acting on them before construction starts.

Before the 1960s, researchers and civil and mining engineers working in rock were developing independently their own tests and methods to assess rock mass properties. It is not surprising that early efforts of the ISRM were the establishment of a common to all terminology and the standardization of the different testing techniques and procedures that were used to determine rock and rock mass properties. This second task led to the creation of the Commission on Standardisation of Laboratory and Field Tests (now the ISRM Commission on Testing Methods) that has been working until today ever since 1966 [9]. Though its mandate was to go ahead with the development of test standards, documents published by the commission were not issued as standards but rather as Suggested Methods. It is a term that was carefully chosen, since Suggested Methods do not intend to be testing standards, but documents where practitioners that have not been involved with a particular subject can find guidance, explanations and recommended (not strictly mandatory) procedures [10, 11]. Many ISRM Suggested Methods deal with tests that are not (or were not at the time of publication) available as test standards. Description of rock mechanics tests presented in this chapter derives from ISRM Suggested Methods and other applicable standards, such as ASTM, EN and ISO.

Very often, rock masses include many discontinuities so that they have a blocky structure. The three-dimensional basic elements of these structures are the elementary blocks, without visible macroscopic fractures basically, made of more or less massive, intact rock. Discontinuities are two-dimensional geologic features that occur in rock masses in a large diversity of forms, and their classification is not straightforward. The most conventional differentiation considers simply joints and faults. In general terms, faults are considered to be fractures in rock continuity along which an identifiable shear displacement of the adjacent faces has occurred, usually resulting from rock mass movements occurring over geologic times. Opposed to faults, joints are fractures within the rock that do not exhibit shear displacements between their surfaces. Joints are caused by fractures of the rock body as a result of tensile stresses induced by geologic events such as the folding of rock masses, shrinkage of a rock body due to a temperature decrease or the reduction of stresses caused by the erosion of overbearing rock layers.

Geometrically, both can be considered as approximately plane surfaces, currently defined in Geology by a pair of angles (strike and dip, or dip and dip direction), though some folding often occurs, mainly in the case of larger discontinuities. Usually, joints display a smaller extent or persistence and they occur in an ordered manner: joints with approximately parallel orientations form a joint set. The evaluation of the geometric characteristics of the discontinuities (orientation, intensity, spacing and persistence), and also other descriptive parameters (roughness, aperture, wall strength and filling), is usually performed during geotechnical surveys, and they will not be addressed in this chapter.

Whether they are joints or faults, discontinuities are responsible for not allowing rock masses, at the scale of rock engineering projects, to comply with the basic assumptions of solid mechanics of continuous, homogeneous, isotropic and linear elastic media (known as CHILE media): first of all, they turn rock masses into discontinuous and inhomogeneous media, and additionally their occurrence defines preferential directions that make several characteristics and properties display anisotropic, non-reversible and non-linear elastic behaviours (known as DIANE media).

A basic approach could lead to consider the behaviour of rock masses as the result of some kind of sum of the behaviour of their components: intact rock plus discontinuities. As a consequence, the assessment of rock mass properties could be reached by sampling and testing rock and discontinuities separately in the laboratory and extending the aggregate of the results to the field scale. The other approach would be to evaluate the rock mass properties performing in situ tests involving a tested volume large enough to be considered representative of the rock mass, being the representative elementary volume (REV) the minimum volume of rock mass that encompasses the relevant features of any larger volume. The notion of size effect in the scope of materials testing refers to the variation of a certain property with the size of homothetic samples. In rock mechanics testing, the term "scale effects" is often used in a broader sense denoting not just the difference between sample sizes, but including also the consideration of greater rock volumes that comprise discontinuities and the upscaling to the dimension of the engineering project. Results of laboratory and in situ tests are thus affected by both the chosen testing locations and the volumes representativeness involved in the tests, particularly their relationship with the engineering work that is being considered. Figure 2 shows a schematic representation of scale effects as it is interpreted in rock mechanics.

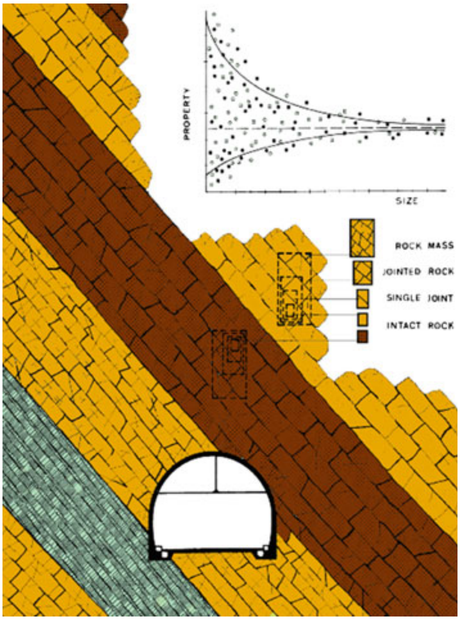

Fig. 2 Schematic representation of scale effects in rock masses [12]

The development of in situ testing methods and techniques specifically dedicated to the geotechnical characterization of rock masses derives from the need to address the issues related with scale effects. In early years, it was also responsible for the recognition of rock mechanics as a distinctive scientific discipline within the geotechnical sphere.

Rock has been used by mankind as a building material and for other purposes since early years. Records of the first mechanical testing of materials are attributed to Leonardo da Vinci ca. 1500, and the first documented rock mechanics experimental study, performed by Gautier around 1770, referred to a testing machine with a lever system that was used to measure the compressive strength of specimens for the pillars of Sainte Genevieve Church in Paris [11].

There are several possible ways to classify the different types of rock tests, none of which being fully satisfactory. In this chapter testing techniques were simply divided into laboratory and field tests. However, even this simple distinction is not undisputable, as several tests can be performed in the field using portable laboratory equipment.

Another informative subdivision is to classify the tests according to their purpose. On the one hand, design tests are those that are used to provide a quantitative measure of given rock or rock mass characteristics, such as the deformability modulus or the shear strength. On the other hand, index tests are simple testing techniques used to give indications about a given characteristic. Since they are generally inexpensive, they can offer important sets of data and thus provide useful estimators for characterization of several physical properties of rock [2]. Another relevant advantage of index tests is that useful correlations have already been established, such as the point load index and the unconfined compressive strength, or they can be specifically defined in the scope of a given project.

Some sandy, clayey, carbonate, or evaporitic geomaterials, referred to as soft rocks, are sensitive to water, and display crumbling, foliated, slaking or expansive characteristics. Additionally, they are difficult to sample, requiring special cutting and drilling techniques, for instance without water, and testing equipment and standard procedures need to be adapted considering limits for specimen deformations. Index tests and correlations may play an important role in overcoming such issues.

In the subsequent sections, the most preponderant tests used for the estimation of rock mass properties usually included in geotechnical characterization for the design of major civil engineering infrastructures are described. Tests used for assessing rock hardness or abrasivity and their interaction with the wear and capabilities of drilling and cutting equipment, tests carried out to characterize rock as a construction material (aggregates or ornamental stones), and tests specially devised for mining and petroleum engineering, are not addressed in the chapter.

3 Laboratory Tests

3.1 Uniaxial Compression

Regarding deformability, many intact rocks show an almost linear elastic behaviour under loadings lesser than 40–50% of their strength, which can be described by two elastic constants, Young's or elasticity modulus E and Poisson ratio ν in the isotropic case, or by five or more depending on the anisotropic degree. These parameters are determined in uniaxial compression tests of cylindrical rock specimens taken from borehole cores or of prismatic or cylindrical specimens cut from rock blocks. The same specimens can be also used to determine the uniaxial or unconfined compressive strength (*UCS*) of the intact rock.

Specimens diameter or side should not be less than 54 mm, or at least greater than 10 times the rock grain size. Specimens should have a height to diameter, or side, ratio of 2.5–3.0. Flat ends and perpendicularity of the specimens should be ensured by an adequate specimen preparation [10, 13, 14].

To determine the elastic constants or simply to control the test, the axial and diametric or lateral strains are measured using strain gauges applied directly on the specimen's faces or displacement transducers coupled to the specimen with specially designed devices (Fig. 3). Standard procedures specify that measuring devices should be placed close to the mid-height of the specimen, and they should average at least two strain measurements. The measuring length of the gauges or devices should be at least ten times the rock grain size. The test is carried out in a loading device to

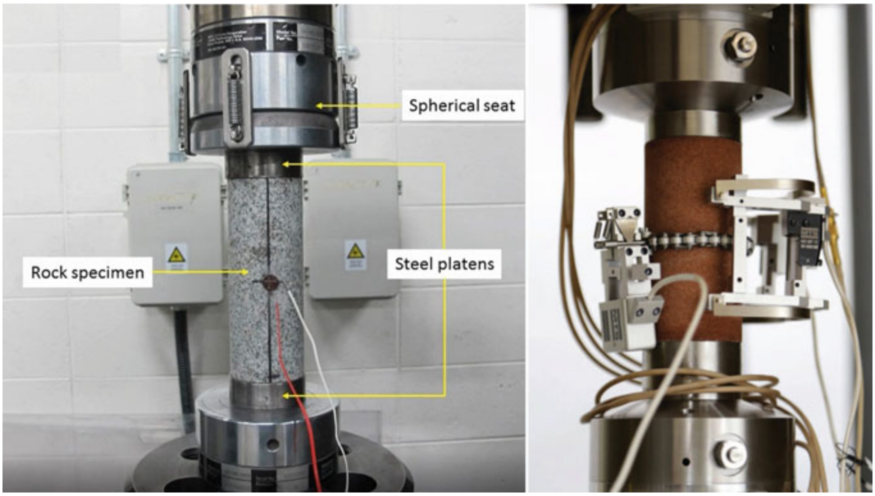

Fig. 3 Uniaxial compression test specimens with electric strains gauges (**left**) and displacement measuring devices (**right**)

consistently apply load at a required stress or strain rate. It is pointed out that stress-controlled tests may lead to explosive failure of the specimens, due to the brittle behavior of hard rocks, and only strain-controlled devices can capture the behavior of the specimens close to and after failure occurs. This requirement leads to the use of stiff servo-controlled testing systems with displacement or strain control to perform these tests [15].

Tests are performed by applying the axial load continuously at a pre-defined stress or strain rate until failure occurs or a predetermined amount of strain is achieved. The stress or strain rates should be selected in order that failure is reached in a test time between 2 and 15 min.

Young's modulus of the specimen, defined as the ratio between a certain axial stress change and the axial strain produced by it, can be calculated using several methods: tangent modulus measured at a fixed percentage of the compressive strength (usually 50%), average modulus of a linear portion of the axial stress axial strain curve, or secant modulus up to a fixed percentage of compressive strength. Figure 4 shows an example of a graph from a uniaxial compression test with the latter calculation.

In some cases, it is preferable to apply two or three loading–unloading cycles at a given stress rate up to an axial stress in accordance with project design requirements, use them to calculate the elastic constants, and then apply a strain-controlled loading cycle until failure.

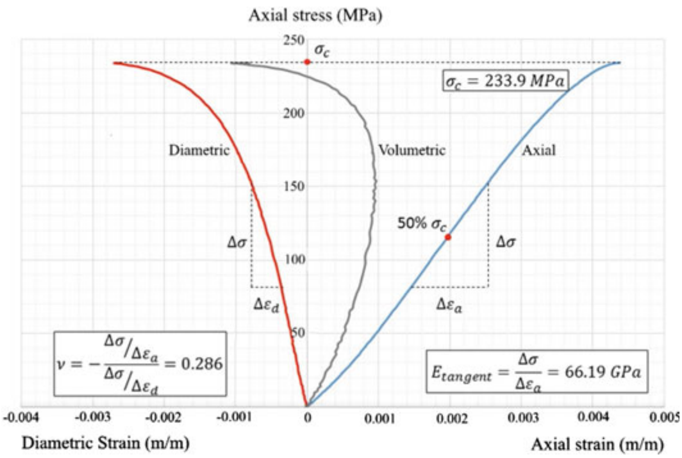

Fig. 4 Graph with the results of a uniaxial compression test

3.2 Triaxial Compression

Assessment of rock strength is necessary for the rational design of underground structures, such as caverns and tunnels. In engineering, the relationship establishing the stress condition by which ultimate strength is reached is referred to as a "failure criterion". They are often expressed as a function of the major principal stresses that rocks can sustain for given values of the other two principal stresses. The Mohr–Coulomb and Hoek–Brown are the most frequently used failure criteria, but both incorporate only the major σ_1 and minor σ_3 principal stresses, and the effect of the intermediate stress is not considered.

Parameters for failure criteria can be determined empirically or from laboratory tests, aiming at characterizing strength and deformation behaviour under stress conditions simulating, as close as possible, those encountered in situ [16]. However, most laboratory tests are conducted on cylindrical specimens subjected to uniform confining pressure, reproducing only a particular field condition where intermediate and minor principal stresses are equal ($\sigma_2 = \sigma_3$). Triaxial tests have been widely used for the study of mechanical characteristics of rocks because of equipment simplicity and convenient specimen preparation and testing procedures.

The main difference between triaxial and uniaxial compression tests lies in the fact that the specimen is inserted in a triaxial cell. Inside this cell, a confining pressure is applied to the specimen by a hydraulic fluid inside the cell, usually oil, that is kept from penetrating into the rock pores by a flexible membrane (Fig. 5, left) [17]. The confining pressure is controlled by a hydraulic system that has to be able to keep it constant during the whole test, taking into account that changes in the specimen's volume resulting from stress changes will affect the oil pressure inside the triaxial cell. The axial stress is applied by a loading device with steel platens of prescribed

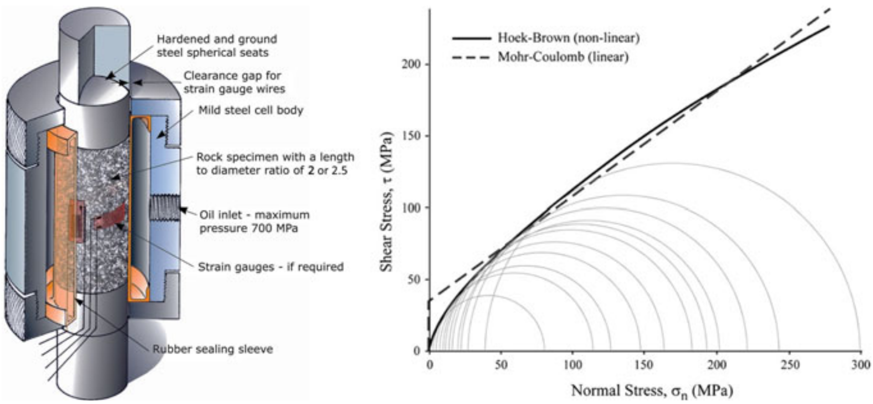

Fig. 5 Cut-away view of a triaxial cell (**left**) [17], and graph with results of a set of triaxial tests and the resulting envelopes for the Mohr–Coulomb and Hoek–Brown failure criteria (**right**) [11]

hardness. It is possible to measure axial and diametric strains using electrical strain gauges applied to the rock surface or displacement transducers inside the cell.

The most frequent test procedure starts with inserting the specimen in the triaxial cell and applying a confining pressure. The cell is then placed in the loading device that will continuously increase the axial load until failure and peak load are obtained. Performing a series of tests with different confining pressures on specimens sampled from the same rock lithology or horizon, test results, σ_1–σ_3 pairs, allow calculating the parameters of the considered failure criteria [18]. Figure 5 (right) shows a graphical representation, in the shear stress—normal stress plane, of the failure envelopes obtained from triaxial tests results.

3.3 Diametral Compression

The diametral compression test, also referred to as Brazilian, or Brazil test or splitting test, is an indirect tensile test intended to estimate the tensile strength of intact rock. It was first developed in 1943, while studying the correlation between compressive strength and flexural tensile strength [19].

By definition, the tensile strength of intact rock should be obtained from the direct tensile test. However, direct tensile test preparation is difficult for routine applications, since it is problematic to attach a cylindrical rock specimen to the jaws of a testing machine. The Brazilian test soon presented itself as an attractive alternative because it is much simpler and inexpensive. Furthermore, rock mechanics design usually deals with complicated stress fields, including various combinations of compressive and tensile stress fields, and testing across different diametrical directions allows determining variations in tensile strength for anisotropic rocks.

This test involves compressing a cylindrical specimen along diametrically opposed longitudinal thin surfaces of a cylindrical specimen using a common load system [10, 18]. Under the action of such load, tensile stresses develop perpendicularly to the loaded diameter and as load is steadily increased the specimen breaks. The load is transmitted to the specimen by steel jaws with cylindrical loading surfaces with larger radius than the specimen's radius until failure. The specimens are right circular cylinders with a height equal to the radius (disks). Figure 6 shows the loading and the stresses occurring along the loaded diameter (left) [20], and a specimen being tested (right) [21].

3.4 Elastic Wave Velocity

The propagation of artificially generated elastic waves through a rock medium can be used to assess the elastic properties of rocks. It is a common non-destructive method that measures the velocities of compressional V_P and shear V_S waves, and, given the

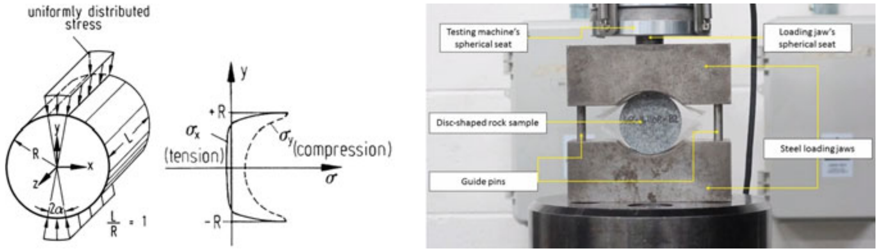

Fig. 6 Schematic representation of the loading and of the stresses along the loaded diameter in a diametrical compression test [20] (**left**), and picture of a specimen being tested [21] (**right**)

bulk density, allows estimating the dynamic Young's modulus and Poisson's ratio of intact rock.

Laboratory wave velocity measurements are usually performed on cylindrical rock specimens prepared for other strength tests, namely uniaxial and triaxial compression tests. The equipment includes an ultrasonic pulse wave generator, a transmitter and a receiver that are coupled to the flat end surfaces of the specimen with a bonding product to improve acoustic transmissivity (Fig. 7). Travelling time of the waves is measured by an oscilloscope, enabling to calculate V_P and V_S, given the length of the rock specimen is also measured [22]. If the mass density of the rock specimen is determined, the Young's modulus and Poisson coefficient can also be calculated. These values are usually referred to as dynamic parameters.

Fig. 7 Ultrasonic velocity test equipment

3.5 Joint Shear

It is common practice to perform laboratory direct shear tests on relatively small discontinuity samples with the objective of estimating the peak and residual or ultimate shear strength of rock discontinuities, as a function of the normal stress applied on the sheared plane [11, 23]. Direct shear tests are mostly conducted with a constant normal load (CNL) applied to the discontinuity plane. This boundary condition is appropriate for a group of engineering problems involving the sliding of rock blocks near the ground surface (e.g., rock slope stability and surface excavation stability). However, when dilation of a discontinuity is constrained during sliding (e.g., around an underground excavation), the normal stress on the sliding surface may change as shear displacement occurs. For this class of problems, constant normal stiffness (CNS) shear tests are more appropriate for determining joint shear strength.

Under CNL conditions, shear strength determination usually includes the application of several different magnitudes of normal stresses on multiple samples from the same joint to determine its shear strength. Alternatively, in cases where it is not possible to sample a representative number of specimens, the same specimen can be tested repeatedly under different constant normal loading conditions. For a single rock joint, at least three, but preferably five, different normal stresses should be used. To minimize the influence of damage and wear, each consecutive shear stage is performed with an increasingly higher normal stress. Usually, multi-stage shear tests are not practical under CNS conditions.

Commonly, direct shear testing machines include a relatively stiff frame against which the loading devices can act, a stiff specimen holder (shear box) in which the two halves of the joint are firmly fastened yet allowing relative and shear displacements, loading devices to apply the normal and shear loads to the specimen, and devices to measure both shear and normal loads and displacements (Fig. 8, left).

The applied normal and shear forces are usually provided by actuators (hydraulic, pneumatic, or gear driven), and cantilever systems can also be used to apply a constant normal load for CNL tests under low normal stresses. Keeping the normal load or stiffness constant during the shear test is very important, and it is usually achieved by servo-controlled close-loop systems (Fig. 8, top right).

Rock joint specimens for direct shear tests are prepared from rock blocks or drilled core samples containing the joint using techniques that minimize disturbance. Usually, specimens are encapsulated with cementitious mortar or similar material, allowing them to be tightly fastened in the shear box (Fig. 8, bottom right).

Specimen sizes depend on the dimension of the shear box, and usually their length along the shear direction ranges between 100 and 200 mm and does not exceed around 400 mm. Length of the specimens should cover the main roughness features of the rock joint, but frequently low frequency waviness is not tested.

Results of rock joint shear tests are presented as plots with the shear stress versus shear displacement graphs. Using these graphs and the records of the measured stresses and displacements, the peak and residual shear strength of each rock joint can be determined. Then, these values are used to calculate the strength parameters of

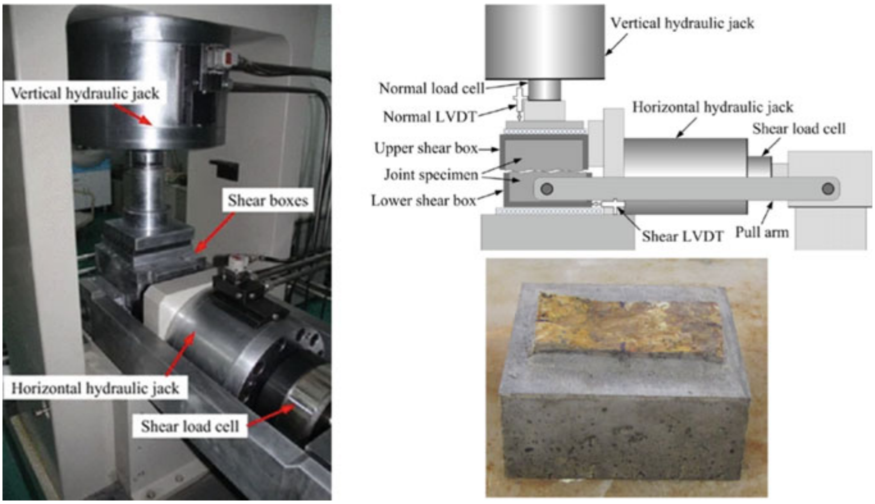

Fig. 8 Rock joint shear test equipment (**left**), schematic representation of the loading frame (**top right**) [24], and encapsulated half of a joint specimen (**bottom right**)

a prescribed failure criterion. Figure 9 shows the plots of the shear stress versus shear displacement graphs of a multi-stage rock joint shear test and the respective peak and residual shear strengths, that allow calculating the parameters of the relevant strength envelope.

Despite the non-linear strength envelope usually obtained for peak shear strength, results of rock joint shear tests are often modelled by the linear Mohr–Coulomb criterion, thus allowing to calculate the friction angle and the apparent cohesion.

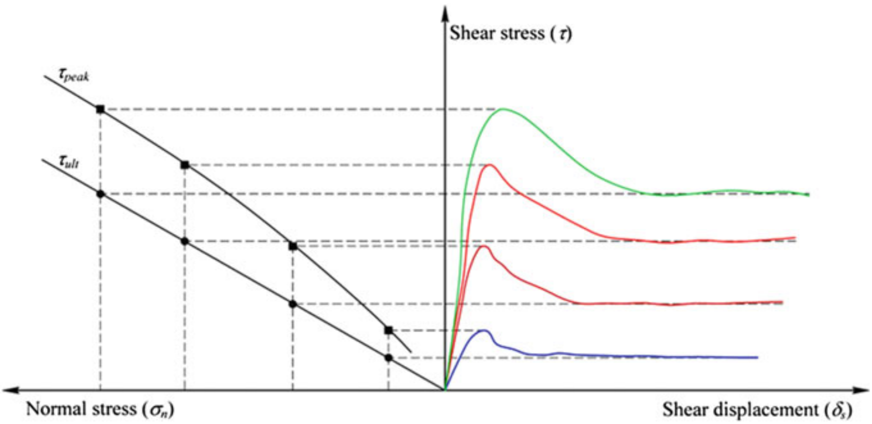

Fig. 9 Plots of the shear stress versus shear displacement graphs of a multi-stage rock joint shear test and the respective peak and residual shear strengths [11]

Particular care should be paid not to extrapolate below the value of the lowest normal stress applied during the test.

In the case of rough or non-planar joints, a non-linear shear strength envelope may be more representative of the test results. In these cases, it is possible to consider other well-established failure criteria, calculate the respective parameters, and deliver them also as results of the tests (e.g., the i value of Patton bilinear criterion [25], or the joint roughness coefficient (JRC), the joint wall compressive strength (JCS) and the residual friction angle (ϕ_r) values of Barton-Bandis criterion [26, 27].

The procedure for joint shear tests described in this section is not intended to cover direct shear tests of intact rock or other types of natural or artificial discontinuities that display tensile strength, such as rock–concrete interfaces or concrete lift joints. However, if the testing equipment holds certain capabilities, namely regarding its loading devices and servo-controlled system, it can be adapted to perform similar tests to determine the shear strength of bonded interfaces.

3.6 Tilt and Pull Tests

Several rock joint shear strength criteria require performing tilt or pull tests to determine some of their intrinsic parameters, being the most prominent the Barton-Bandis model [28, 29]. Tilt tests or pull tests are carried out to assess the basic friction angle (ϕ_b) and the JRC value [30].

Tilt tests are related with the concept of angle of repose of a solid body on an inclined surface. They are carried out by means of simple apparatuses essentially consisting of a rigid plane, which can be rotated around an axis (Fig. 10). A rock joint or a rock surface is placed horizontally on this plane, with the bottom half prevented from moving. The plane is then rotated until the upper part of the joint or surface moves. At this moment, the dip angle of the plane is the friction angle.

In the case of rough rock joints, the tilting angles reach values higher than 70°, generating high stress concentration at the rotating toe of the specimen. To minimize

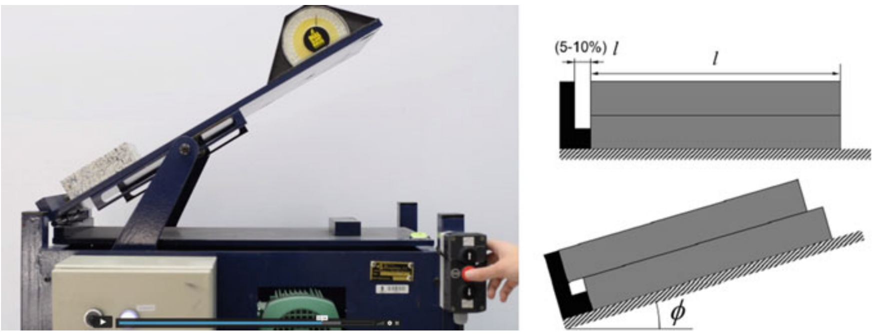

Fig. 10 Tilt test equipment (**left**), and schematic representation (**right**)

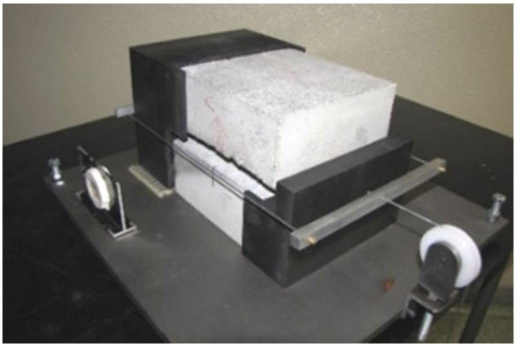

Fig. 11 Pull test equipment

this effect, specimens should have a length to height ratio of the upper block in excess of 4, and pull tests a preferable alternative. Figure 11 shows a pull test apparatus featuring a hard plastic block pulled over roller bearings, that pushes the upper half of the joint sample without any kind of overturning caused by the pull force if it is not parallel to the joint mean surface. The pull force is increased until shear displacement occurs and, given the weight of the upper half of the specimen, the friction angle is easily determined [31].

3.7 Index Tests

Fundamental tests directly measure an intrinsic rock property, such as the compressive strength, while, on the other hand, index tests are simple, cheap and can be performed quickly, but may not determine an intrinsic property. The point load and the Schmidt hammer rebound tests are the best-known examples. Consequently, it is good practice to perform many index tests and calibrate them against fewer fundamental tests, but still with statistical significance according to the property variability.

Point load test. This test method is performed to determine the point load strength index of rock specimens, which is used as an index for strength classification of rock materials or in correlations with the unconfined compressive strength. Since uniaxial compression tests are comparatively more time-consuming and expensive than point load tests, the latter can be used to make timely and more informed decisions during the exploration phases and more efficient and cost-effective selection of samples for more precise and expensive laboratory tests.

Rock specimens for point load tests may be in the form of rock cores (the diametral and axial tests), cut blocks (the block test), or irregular lumps (the irregular lump test), with diameter values D between 35 and 80 mm (Fig. 12). Tests can be performed in either the field or in the laboratory, because the testing machine is portable and little or no specimen preparation is required [10, 32].

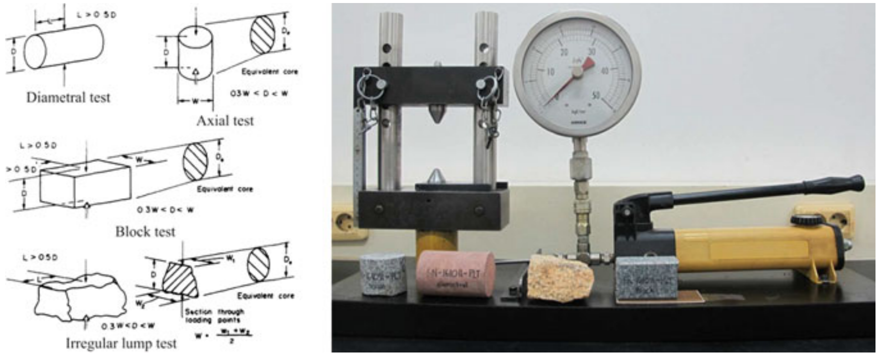

Fig. 12 Point load test equipment with rock specimens and respective size requirements [33]

The result of a single test is the size-corrected Point Load Strength Index $I_{s(50)}$, defined as the value that would be measured in a diametral test with D equal to 50 mm. For a sample of the same rock type several tests should be performed, and the mean $I_{s(50)}$ value is to be calculated after deleting the two highest and the two lowest values as the average of remaining results, for test batches with 10 or more valid tests.

Schmidt hammer rebound. The Schmidt impact hammer is a light, portable apparatus consisting of a spring-loaded piston that transfers its energy as it is released and impacts on a rock surface. Part of this energy is recovered depending on the hardness of the impacted rock. The result of each test is the rebound value R. Though intended to provide a measure of rock hardness, R is most frequently used as an index in rock mechanics practice for estimating rock and joint wall strength, as well as rock excavability and drillability [11].

Though it is a very simple and quick determination, many factors can affect the results. Firstly, as the impact area and released energy are very small, the Schmidt hammer tests only affect a thin band of a few millimetres or centimetres of rock. If the rock specimen is not securely fastened, energy will be dissipated returning a false result. As a consequence, special core specimen holders with V-shaped steel cradles are often used to test cylindrical rock cores, and a large number of impacts should be averaged to render the R value (Fig. 13, left). Moreover, tests should be performed by experienced personnel in order to assure the quality of the result produced by this test method [34].

Schmidt hammer rebound can also be used in the field on rock exposures (Fig. 13, right). As rock faces occur with any given orientation, corrections for reducing the rebound value when the hammer is not used vertically downwards are required.

Fig. 13 Schmidt hammer and core holder (**left**), and used in the field (**right**)

4 Field Tests

4.1 In Situ Stresses

Several authors present descriptions, limitations and fields of application of existing in situ stress measurement methods [35, 36]. For the design of underground structures in civil engineering projects, they are usually classified as methods based on hydraulic fracturing, methods based on complete stress release, and methods based on partial stress release. Methods based on the observation of the rock mass behaviour are less frequently used.

Overcoring and hydraulic fracturing tests are used when the zones of interest can only be reached with boreholes. In most cases, they are performed during the geotechnical survey stage. Flat jack tests require direct access to rock mass surfaces, so they are usually carried out when excavation reaches regions near the underground works. Often their results are used to confirm previous stress field estimates.

Tests for determination of the in situ stresses in rock masses for the design of underground structures are usually scarce in numbers, due to cost and time constraints, they have limitations inherent to their nature, and their results are only valid in the exact locations where they are executed. Owing to these factors, characterization of the in situ stress field in the rock mass at the location of the underground infrastructure often requires a global model for the interpretation of results from all the tests.

Global interpretation methodologies start by establishing a set of assumptions regarding the stress field in the rock mass. For instance, it is common to consider that the vertical and horizontal stresses increase linearly with depth, since the stresses are, in a large proportion, due to the weight of the overlaying ground. Then, three-dimensional numerical models are used to calculate the stresses at the locations where the stress measurements were performed, and an inverse methodology is applied to estimate the in situ stress field that better reproduces the test results [37, 38].

Overcoring tests. Overcoring tests use a complete or partial stress release method allowing to obtain the stress tensor components at a given location in a borehole.

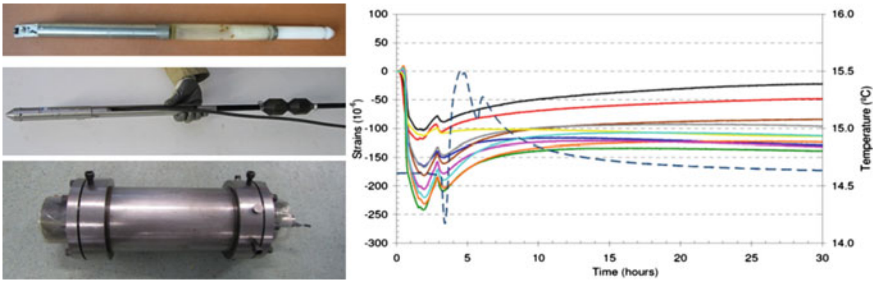

Fig. 14 STT (**top left**), USBM (**middle left**), biaxial test chamber (**bottom left**), and typical strains measured during STT cell overcoring (**right**)

CSIRO and LNEC's STT triaxial cells, and the Borre probe allow determining all six stress components from a single test, while with USBM and doorstopper deformation gauges only the three stress components in a plane can be obtained [10].

STT stress cells are 2-mm-thick epoxy resin hollow cylinders with embedded strain gauges. Test starts by cementing the cell inside a 37-mm-diameter borehole. Then the in situ stresses are released by overcoring with a larger diameter. Strains are measured during overcoring until temperature stabilizes by an in-built data logger. Stresses are calculated using the rock elastic constants obtained in a biaxial test of the recovered core with the cemented cell. Figure 14 presents a STT cell (top left) with the data logger, a biaxial test chamber (bottom left) and a diagram with the typical evolution of the measured strains and temperature during the overcoring process (right).

Flat jack method. The flat jack method is based on partial stress release. LNEC's SFJ (small flat jack) test consists in cutting a 10-mm slot in a rock surface, with a 600-mm- diameter circular disk saw, where a flat jack is inserted. Pressure is applied by the flat jack until deformation caused by opening of the slot is restored. With each flat jack, a single stress component is obtained. Usually, at a given location, several tests in slots with different orientations are performed (Fig. 15) [37, 38].

Hydraulic tests. Two types of hydraulic tests can be performed for the determination of in situ stresses: hydraulic fracturing (HF) and hydraulic tests on pre-existing fractures (HPTF) [10, 39]. HF tests induce a fracture in the rock by applying water pressure in a borehole section isolated by packers, enabling to estimate the minimum horizontal stress. In HTPF tests, water pressure is applied in a borehole comprising an isolated existing fracture whose opening allows to determine the stress component perpendicular to the fracture plane. Figure 16 shows the hydraulic fracturing equipment being inserted in a borehole (top left), an electrical image of a tested fracture (bottom left) and a scheme with the general setup for the hydraulic fracturing tests (right).

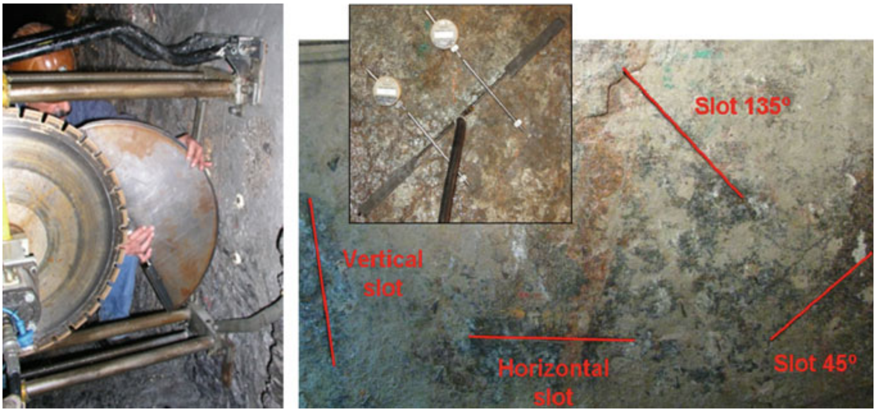

Fig. 15 Flat jack being inserted in the slot, array of slots and instrumentation

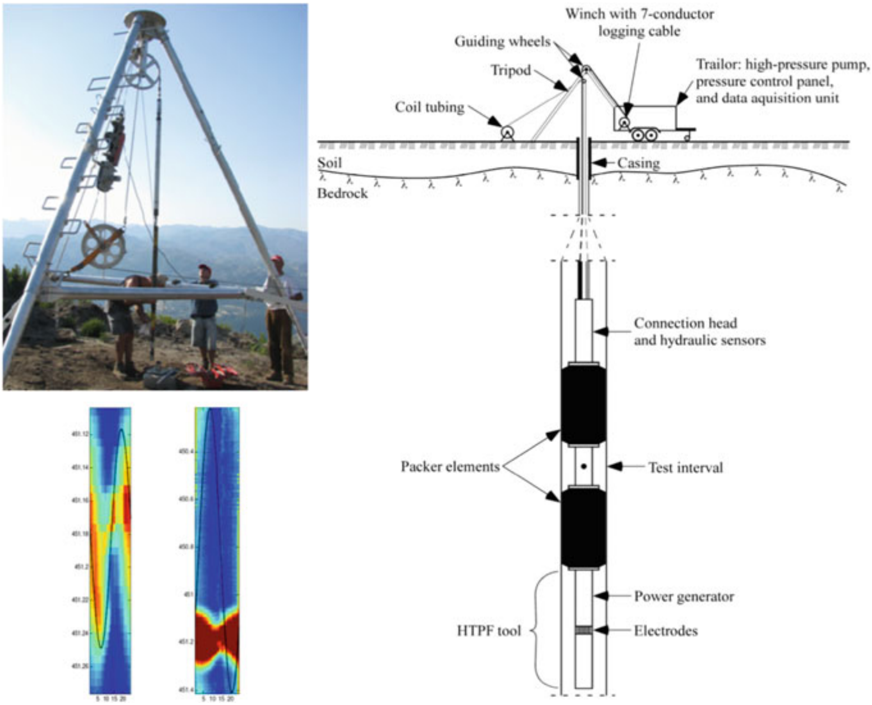

Fig. 16 Hydraulic fracturing equipment (**top left**), electric image of a tested fracture (**bottom left**) and hydraulic fracturing test setup (**right**)

4.2 Permeability

Seepage in rock masses occurs mainly through conductive discontinuities and, for most civil engineering purposes, crystalline rocks can be considered impermeable. This is why reference to permeability tests appears here in the field tests section.

The most commonly used in situ test to estimate permeability in rock engineering works is the Lugeon test, which is also known as “packer test” or “water pressure test”. It was designed by Maurice Lugeon in 1933 as a means of assessing rock mass permeability and the need for grouting at dam sites [40].

The Lugeon test is a stepwise, constant head permeability test performed in a borehole section isolated by one or two packers, whether the isolated section is located at the end of the borehole or not, respectively. Lugeon tests with a single packer are performed as boreholes are being drilled, but double packer tests may be performed after the borehole is concluded (Fig. 17). The injection section length has to be adapted to the jointing of the rock mass, but values of 3 and 5 m are common practice. They are standard tests usually included in geotechnical investigations and in rock mass drainage and grouting curtains in dam foundations.

Test results are expressed as Lugeon units (LU) defined as the loss of one litre of water per minute, per metre of the borehole test section, for an excess injection pressure of 1 MPa measured at the middle of the test section. Estimation of equivalent rock mass permeability from Lugeon tests is controversial, but conversion formulas can be used to calculate the permeability coefficient assuming stationary pressure and flow, and steady-state transmission of water from the borehole to the surrounding medium.

Standard Lugeon tests include several pressure stages, usually five to nine, between a minimum and a maximum pressure. When five pressures are used, the

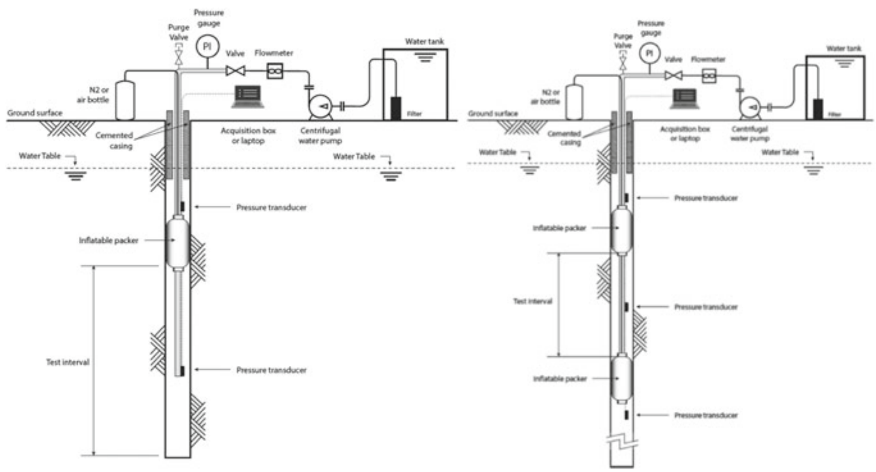

Fig. 17 Scheme of Lugeon tests with a single packer (**left**) and double packers (**right**) [41]

first pressure stage in performed at the minimum pressure, the second at an intermediate value, the third at the maximum pressure, the fourth again at the intermediate value, and the last again at the minimum value. If nine pressure stages are considered, a similar increasing–decreasing sequence is carried out, but with three intermediate pressures. The maximum pressure, which should not exceed 1.0 MPa, is defined taking into account several factors, such as the objective of the test, the depth of the test section and the need to assure that hydraulic fracturing of the rock mass does not occur. After steady flow is reached, each pressure stage lasts 10 min.

A Lugeon value is calculated for each one of these pressure stages, and test interpretation follows from the analysis of the LU values versus pressure plots. Different evolution trends of these graphs during the increasing–decreasing pressure allow to define if flow in the injected rock mass section can be considered laminar or turbulent, or if wash-out or void filling occurred, or even if hydraulic fracturing was reached.

Particular projects may require the execution of particular permeability tests, such as pressure drop test, in which water is injected into a borehole section up until a given pressure is reached and then water injection is stopped and pressure drop (or build-up) is measured, or the constant head Lefranc-type tests used in the case of high permeability environments.

4.3 Deformability

Rock mass deformability plays an important role in the design of several types of structures, because their behaviour depends on the displacements undergone by the rock mass. This is the case of concrete dams, large bridge foundations, underground caverns and tunnel linings. For the design of these important types of structures, it is not adequate to characterize the rock mass deformability by only using laboratory tests on intact rock specimens and extrapolating their results to the rock mass based on geomechanical classifications. For these structures, in situ deformability tests such as borehole expansion tests, plate loading tests or flat jack tests, are required.

Borehole expansion tests. Several types of borehole expansion tests are available to evaluate rock mass deformability, but they involve relatively small rock mass volumes around 0.1 m^3, which are seldom a representative elementary volume (REV). A major advantage is that they are not expensive, as they are performed in boreholes that are generally used for other purposes in the scope of geotechnical investigation of the rock masses, and it is possible to carry out a significant number of tests and use these results for zoning the rock mass deformability at a given site.

Borehole expansion tests can be performed with borehole jacks, also known as stiff dilatometers, that apply a unidirectional pressure over two diametrically opposed sectors of a borehole wall. As an alternative, dilatometers are probes that apply a uniform radial pressure via a flexible rubber membrane pressed against the borehole walls by a fluid. Some of this second type of apparatuses, derived from soil pressuremeters, measure the rock mass deformation indirectly by recording the volume change of the probe, while others, like LNEC's BHD dilatometer, measure directly the diametric displacements with displacement transducers contacting the borehole wall [10, 42].

Dilatometer tests are carried out after the probe is installed at the desired borehole depth and an initial low pressure is applied so that the flexible membrane expands and contacts the walls. Tests usually follow a loading programme including several loading–unloading pressure cycles with increasing peak values and prescribed pressure stages at which pressure is maintained for 1–2 min, displacements stabilize and data (pressure and displacements or probe volume) are measured. Gradual pressure increase and cautious monitoring of the displacements is required, since the applied radial pressure induce tensile stresses that, if in excess, may cause rock fracturing [43, 44].

Test results are plotted as stress versus displacement curves and the deformation modulus can be calculated assuming that the rock mass is isotropic, elastic and linear-elastic. In Fig. 18 a BHD dilatometer probe (left), and the full standard equipment (probe, winch, positioning rod, water pump and read-out unit) (right) are displayed.

Plate loading tests. Plate loading tests are widespread in situ deformability tests, but in some cases they do not provide satisfactory results, because the rock mass in the tested zone is often disturbed by the excavation. They consist of applying pressure via steel loading plates, about 1 m in diameter, to a rock surface in an exploratory adit or test chamber, and calculating the rock mass deformation modulus from the measured deformation [10, 42].

Most frequently double tests are performed on opposite walls at the same location, as one surface is used as reaction for the other. Accordingly, the loaded surfaces have to be coplanar, and any unevenness has to be compensated with cement mortar. The loads are applied by hydraulic jacks, and displacements are usually measured at the

Fig. 18 BHD dilatometer probe (**left**) and full equipment (**right**)

Fig. 19 Plate loading tests in an adit (**left**) [45], and in a tunnel (**right**) [46]

steel plates and at the rock surface around it with displacement transducers, and on occasions also inside the tested rock with extensometer rods.

Relations between the pressure changes and induced displacements of the rock mass allow calculating an equivalent rock mass deformation modulus. Figure 19 shows two plate loading test set-ups, a vertical test in an exploratory adit (left) and a slightly inclined test in a tunnel (right).

On occasions when rock masses are relatively competent, high pressures are required to produce appraisable displacements, which may be hazardous given the precarious stability conditions of the set-ups. In other cases, if the load surfaces are not adequately chosen and prepared, loads may be applied to disturbed rock mass in the excavation damaged zone.

Large flat jack tests. To avoid the shortcomings of plate load tests, large flat jack tests (LFJ) are preferably used, as they also allow testing relatively large volumes of rock mass, of a few cubic meters, while determining the deformability in less disturbed zones of the rock mass [10, 47].

LFJ tests consist in cutting a thin slot in the rock mass, by means of a disk saw, and inserting a flat jack that is then pressurized in order to load the slot walls while measuring the rock mass deformation with several displacement transducers. In order to obtain a mean value of the modulus of deformability in large rock volumes, as well as information about the rock mass heterogeneity, a group of two co-planar contiguous slots is usually cut for each test.

The equipment for cutting the slots includes a machine, with a 1000 mm diameter diamond disk saw mounted at the end of a rig that houses the system that transmits the rotating movement to the disk. A central 168 mm diameter hole with a depth of 1.10 m is previously drilled by the same machine, in order to allow the introduction

Fig. 20 Plate loading tests in an adit (**left**), and in a tunnel (**right**) [47]

of the disk supporting column. The disk saw cuts 1.50 m deep slots (Fig. 20). Once a slot is cut, a flat jack is introduced and, after the central hole is filled with cement mortar, the jack is ready to be filled with hydraulic oil and pressurized. Usually, as tests are carried out with two flat jacks side by side, this procedure is repeated for the second jack. Each flat jack consists of two steel sheets less than 1 mm-thick, welded around the edges. Inside the flat jack, four transducers measure the opening and closure displacements of the slot. The flat jacks are then inflated to adjust to the surface of the slots and a low initial pressure, usually of about 0.05 MPa, is applied (Fig. 21).

A LFJ test comprises at least three loading and unloading cycles reaching increasing maximum pressures. Displacements are measured by the four transducers in each flat jack and, in some cases, by transducers mounted on the rock surface across the slot. The raw test results are the pressure versus displacement curves obtained in the test.

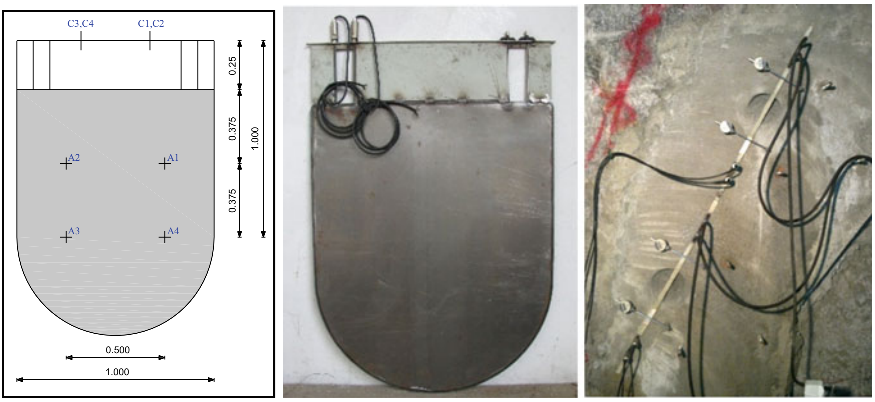

Fig. 21 Schematic representation of a large flat jack (**left**), a large flat jack (**centre**), and s LFJ test set-up with two jacks on a vertical wall of an adit (**right**) [47]

Pressure applied to the rock mass by the flat jacks in the cut slots causes tensile stresses to develop at the edge of the cut slot. As a test is carried out, pressures and stresses increase and if the combination of rock mass tensile strength and in situ stresses is exceeded, which is common, a tension crack will develop around the slot. Though it might not be visible at the surface, it will be noticed in the pressure versus displacement curves as they will show a decrease in the deformability. This conclusion is used to outline the continuation of the test and establish the maximum pressures of the following cycles. It is also used in the model for interpretation of LFJ test results to calculate the rock mass deformability modulus, which is based on the theory of elasticity for homogeneous, isotropic and linear elastic bodies, and takes into account the possible development of the tension crack.

4.4 Shear Strength

Best shear strength estimates are obtained from in situ direct shear tests as they inherently account for any possible scale effect. However, due to the duration and high cost of such tests, they are solely performed in special cases, to assess the shear strength of particular interfaces in the rock mass relevant for design, such faults and joints with thick fillings, veins or weathered bands, bedding or interlayer planes, and concrete-rock contacts.

In situ direct shear tests can be performed underground in exploratory adits and test chambers on discontinuities with any orientation, or at the surface. The walls and roof of the adit or tests chambers provide the reactions for the normal and shear forces, often reaching 4 MN.

Preparation for an in situ shear test is very complex and time consuming. First, after defining the test location and the shear direction, a rock block with the discontinuity, around 1 m^2 in area and 0.5 m in height, is cut using disk saws or drilling overlapping boreholes. Then, the block is encapsulated with reinforced concrete or a steel frame. Concrete blocks are built on the roof or sidewalls of the adit for reaction of the normal and shear forces. All these operations have to be executed ensuring that the discontinuity does not move and that all filling materials are not disturbed (Fig. 22 left). In situ direct shear tests performed at the ground surface, anchored concrete and steel structures are required to provide reaction blocks for both normal and shear forces (Fig. 22 right).

Sometimes, the direction of the shear jacks is inclined in relation to the discontinuity plane, but acting through its centroid. The forces are applied using hydraulic jacks, either cylinders or flat jacks. Load cells can be used, but usually stresses are calculated from the pressure of the jacks, considering the area of the discontinuity and its inclination: Transducers are used to measure normal and shear displacements and if environmental conditions at the testing site allow, a data acquisition system may be used.

Owing to the high costs of these tests, they are typically performed as multi-stage shear tests under several, usually five, increasing normal stresses. Tests start

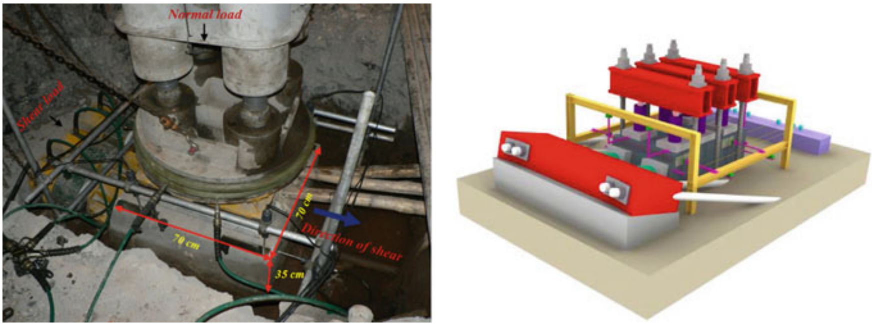

Fig. 22 Example of in situ direct shear tests on the floor of an adit (**left**) [48], and schematic representation of a test at the surface (**right**) [49]

by applying the lowest normal stress until stabilization of the normal displacements is reached. Normal stress should be applied at a slow rate in order to allow excess pore pressures in the filling material to dissipate. Then, shearing at a constant shear displacement rate (0.1–0.5 mm/min) is initiated and continues until the shear displacement progresses under an approximately constant shear stress. If the shear force is inclined, as it is increased, it produces an increase of the normal load that needs to be continuously compensated as shear displacement goes on. After this first stage, the shear stress is slightly decreased and the normal stress is increased to the value established for the second stage, and a similar sequence follows. Completion of the test happens after several stages under increasing normal stresses are performed.

Results of a test are plotted as shear stress versus shear displacement curve that allows defining the shear strength for each normal stress, and subsequently enables plotting these values and the calculation of the strength parameters of the tested discontinuity, for instance the friction angle and the apparent cohesion.

5 Concluding Remarks

In the scope of large civil engineering projects, the laboratory and field tests carried out for the characterization of rock masses can be seen as small pieces of a large puzzle aimed at providing fundamental elements about the rock mass for the design. The values of the parameters that characterize the rock mass properties, which will be used in design, shall result from an expert and cautious judgement of the whole range of values obtained from the testing program. Once integrated in a safety verification procedure with adequate safety requirements, they will provide an important basis for assuring safety during the construction and exploration stages of the project.

The laboratory and field tests presented in this chapter were considered as the most relevant and commonly performed for the characterization of rock masses. It has to be recognized that even a simple enumeration of all currently available methods and techniques is an unfeasible task, and that the biased selection that was inevitably necessary reflects the experience of the authors. Furthermore, all descriptions of testing equipment, methods and procedures included in this chapter had to be seriously shortened to a minimum, but still allowing to fully comprehend the underlying basic principles of the tests, their objectives and results, and their benefits and shortcomings. Detailed description of equipment, in particular of the measuring devices, was intentionally excluded given their continuous advancements.

Referencing had to be considerably abbreviated also. References in this chapter have to be understood as starting points for wider searches. It was sought to provide the source references for each test and they often comprise the ISRM Suggested method and the corresponding ASTM standard.

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