

required mitigation and prevention and the approach to assessing the hazards was the same.

For engineering geologists working in environments where mass movements of any type are possible it is beholden on them to ensure all the hazards are identified. The methods of investigation for all forms of mass movement are essentially the same and require the creation of an adequate ground model that will enable the nature and scale of any hazards and risks to be identified and quantified. The ground model can then be used in the design of any measures needed to mitigate the risk, whether this is to existing or proposed infrastructure development. Note that the ground model is not a simple definitive construct as it will develop and improve when new data become available during investigations and any subsequent construction. These developments of the ground model must be incorporated into any design process.

There is a wealth of literature and on-going research into the nature and causes of mass movement. Nevertheless, it is still not possible to state with any certainty where or when a slope failure will occur and what size it will be nor the extent of the runout. Although snow avalanche research has probably advanced further in establishing these issues than landslide studies the Galtür disaster in Austria in 1999 remains as a salutary lesson showing there are still many unknowns. Until these questions can be answered mass movement research must continue to ensure there are no more disasters like the 1963 Vaiont Reservoir landslide.

Cross-References

- [Avalanche](#)
- [Climate Change](#)
- [Engineering Geomorphological Mapping](#)
- [Factor of Safety](#)
- [Glacier Environments](#)
- [Hazard](#)
- [Hazard Assessment](#)
- [Landslide](#)
- [Mountain Environments](#)
- [Risk Assessment](#)
- [Shear Strength](#)
- [Shear Stress](#)
- [Site Investigation](#)
- [Stabilization](#)

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Mechanical Properties

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Definition

Physical properties that determine the behavior of ground that is under mechanical stress.

Mechanical properties of ground determine the behavior of the ground under stress in a mechanical way; for example, settlement of ground under a foundation, subsidence due to underground excavation or extraction of gases and fluids, tunnel and slope stability, and “breaking” of rock or cemented soils. Deformation is the change in volume or shape of ground under stress in which rheology and viscosity may be important. Strength commonly denotes a stress condition at which the ground fails (breaks) when a threshold in stress conditions is exceeded. Constitutive models are the aggregate term for relations that describe the chemical-physical-mechanical behavior of ground.

Ground materials are diverse and may be gases, fluids, solids (i.e., minerals, grains, and aggregates of grains or minerals) and any mixture of these and also include man-made ground, such as fills and waste dump materials. Ground is commonly differentiated between soil and rock, soil being an aggregate of loose or weakly bonded particles and rock consisting of particles cemented or locked together, giving rock a tensile strength. Soil and rock are, by some, differentiated based on a compressive strength difference with soil being weaker than 1 MPa and rock being stronger. A differentiation is made between “intact” and “discontinuous” ground, that is, ground without and with, respectively, distinct planes of mechanical weakness (discontinuities) such as faults, joints, bedding planes, fractures, schistosity. A groundmass consists of (blocks of) intact ground with discontinuities, if present.

normal stress minus the hydrostatic stress, which is the average of all normal stresses working on a body of ground, is the deviatoric stress.

Total and Effective Stress, and Gas and Fluid Pressure

A porous ground consists of a skeleton of solid particles with pores in-between. The pores may contain gases and fluids. When a load is applied on a body of ground, part of the load is taken by the skeleton resulting in stress in the skeleton, and part by the gas and fluid. The skeleton, gas and fluid will deform. Many mechanical characteristics of the ground depend on the stress between particles; therefore, normally the load is divided into “effective stress,” or the stress in the skeleton, and the stress (pressure) in the gas and fluid. In the case of pores filled by water, this is the “porewater pressure.” Effective stress together with the gas and fluid pressure is the “total stress” which equals the stress from the outside on the body of ground. In a porous ground without any gas or fluid and in a nonporous ground, the effective stress equals the total stress. Total stress may be applicable in situations where the pore gas and fluid pressure cannot or dissipates too slowly, for example, during fast loading of a low-permeable clay (the “undrained” situation). In most cases, the presence of gas is neglected under engineering conditions, but may be important in, for instance, subsidence due to gas and oil exploitation.

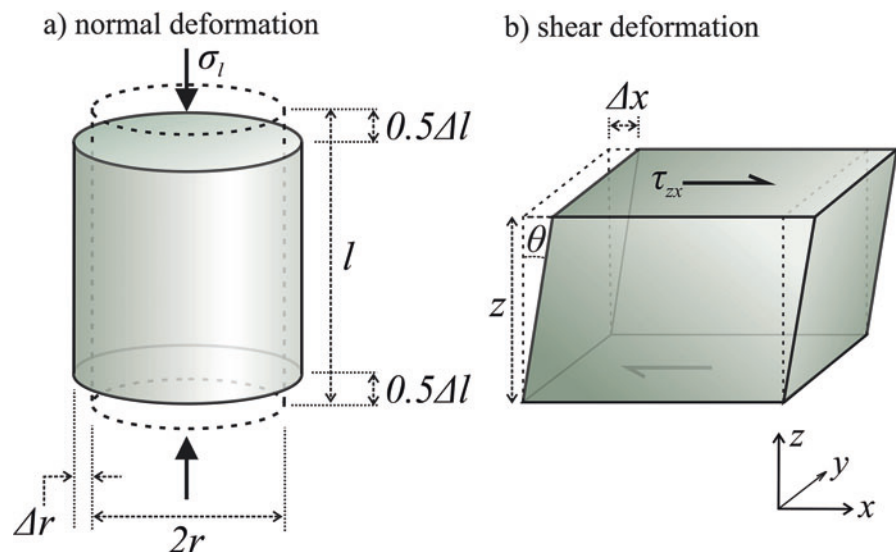
Stress, Deviatoric Stress, Deformation, and Strain

If a normal (σ) or shear (τ) stress is applied on a body of ground, the ground will deform, that is, will be strained, respectively, and change in shape by an angle rotation. The

Elastic Deformation

The relation between normal and shear stress and strain for an intact, homogeneous, isotropic, and ideal-elastic body of intact ground is formulated in Eqs. 1 and 2 (Fig. 1). In elastic

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Fig. 1 Deformation, stress and strain



deformation, stress and strain are coupled properties; there is no strain without stress and vice versa. Under the influence of a normal stress (σ) in a particular direction, the material becomes shorter in that direction and wider perpendicular to the stress direction. The amount of shortening in relation to the stress is expressed by the elastic deformation or Young's modulus (E). The amount of widening in one direction related to the shortening in the other direction is expressed by the Poisson's ratio (ν).

$$E_l = \frac{\sigma_l}{\varepsilon_l} \quad \varepsilon_l = \frac{\Delta l}{l} \quad \varepsilon_r = \frac{\Delta r}{r} \quad \nu_{lr} = \frac{\varepsilon_r}{\varepsilon_l} \quad (1)$$

E_l = elastic Young's modulus of deformation in l direction [Pa]

ν_{lr} = Poisson's ratio of expanding in r direction due to stress in l direction

σ_l = stress in l direction [Pa]

l, r = length respectively radius of body [m]

Δ_l, Δ_r = deformation in l respectively r direction [m]

$\varepsilon_l, \varepsilon_r$ = strain in l respectively r direction

Shear stress (τ) causes a deformation in shape governed by the shear modulus (or modulus of rigidity). The shear modulus is defined as follows:

$$G_{zx} = \frac{\tau_{zx}}{\gamma_{zx}} \quad \gamma_{zx} = \frac{\Delta x}{z} = \tan \theta \quad (2)$$

G_{zx} = shear modulus in z direction due to shear stress in x direction [Pa]

τ_{zx} = shear stress in x direction on plane with normal in z direction [Pa]

γ_{zx} = shear strain

l, r = length respectively radius of body [m]

Δx = shear deformation in x direction [m]

z = height of body [m]

For an intact, homogeneous, isotropic, and ideal-elastic solid, the Young's modulus, Poisson's ratio, and shear modulus are related following:

$$2 G(1 + \nu) = E \quad (3)$$

Deformation characteristics are often expressed in terms of Lamé parameters (also named "constants" or "coefficients") λ and μ :

$$\lambda = \frac{\nu E}{(1 + \nu)(1 - 2\nu)} \quad \mu = G = \frac{E}{2(1 + \nu)} \quad (4)$$

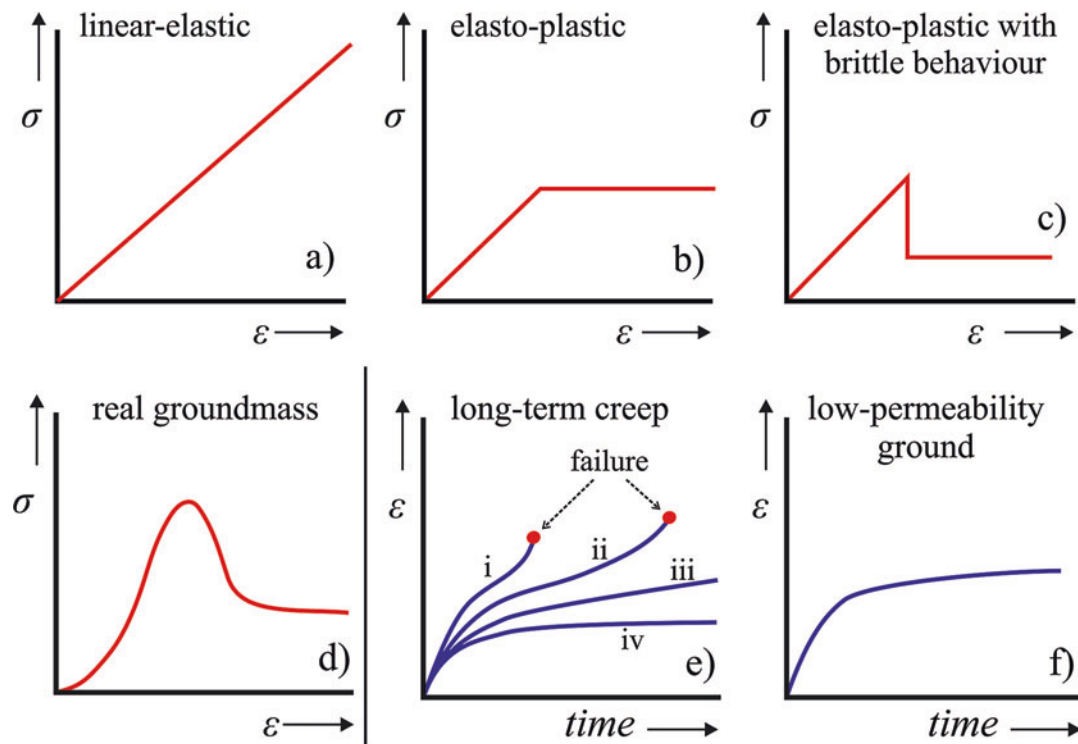
Moduli and Poisson's ratio are anisotropic for most intact ground and most groundmasses, that is, the values vary with direction.

Non-elastic Intact Ground and Groundmass

Most intact ground and virtually all groundmasses do not deform in an ideally elastic manner, and the normal and shear deformation moduli are not elastic or only partially elastic (Fig. 2). Ground may deform as a combination of plastic, elastic, and brittleness, properties which may also depend on factors such as time, temperature, confining stress, presence of gases and fluids, and nuclear radiation. Brittleness means that the intact ground fails, that is, breaks or fractures. Figure 2a shows a linear-elastic deformation; on release of the stress, the sample will return to its original volume and shape. In Fig. 2b the material behaves elasto-plastically; the first part is elastic deformation, whereas in the second (plastic) part the deformation increases under constant stress; the ground does not return to its original volume and shape when the stress is released; Fig. 2c is similar to Fig. 2b, but at the boundary between elastic and plastic, the material fails (brittleness). Figure 2d shows the deformation behavior of most real ground which is a combination of elastic, plastic, and brittle deformation. Intact ground, in particular rock, may deform more-or-less elastically for stresses up to 50–80% of the unconfined compressive strength (UCS) value (see below). Groundmasses are seldom intact but mostly contain discontinuities. Discontinuous groundmasses virtually never behave elastically, but mostly deform permanently, thus plastically, with shear displacements along discontinuities. Under higher confining pressure or temperature, most materials show a more gradual deformation without brittleness, such materials are said to undergo ductile deformation. As most intact ground and groundmasses do not or not fully behave elastically, many engineers prefer the letter " D " to denote the deformation modulus rather than " E ." Table 1 lists values for D -moduli and Poisson's ratios for various intact grounds and groundmasses. Note the enormous variation in values even for geologically or lithologically similar ground.

Consolidation and Compaction

Under influence of stress, and over time, ground changes; gases and fluids may be expelled, grains and particles re-arrange, and the constituents of the ground may change structurally and chemically. Generally, this results in a decrease of volume. The expressions "consolidation" and "compaction" are used interchangeably, but consolidation is more often used for soil materials with low permeability and compaction for permeable granular soil, for rock, and for soil becoming rock. In soil mechanics, it is customary to differentiate between primary and secondary consolidation. The first is mainly related to expelling gases and fluids from pores in the ground, whereas the second is more related to



Mechanical Properties, Fig. 2 Various stress–strain (deformation) behavior; σ = stress; ε = strain; (e) Modified after Dusseault and Fordham (1993). Note that in figures (a–d) the horizontal axis is strain while in (e) and (f) it is time

re-arrangement of grains and changes in material, the latter particularly in organic soils, such as peat (Fig. 2f). Mathematically this is characterized by the “coefficient of consolidation” (c_v – a smaller value indicates that more time is required for consolidation) for short-term primary deformation and by the “coefficient of secondary consolidation” for long-term deformation (C_α – a larger value indicates more consolidation in a given time span) (Bodó and Jones 2013). Short and long-term deformation under near-surface conditions may be centimeters to many decimeters per year per meter thickness of ground, the latter especially for ground such as peat and household waste.

Compaction of granular soil-type material under stress involves re-arrangement of the grains resulting in a smaller volume. Compaction of rock and soil becoming rock involves expelling of gases and fluids, re-arrangement, and structural and chemical changes of particles, grains, and minerals. Compaction rates for ground under near-surface conditions are very variable ranging from centimeters per year to millimeters per millions of years or longer per meter thickness of ground.

Time Effects, Creep, and Temperature

Deformation of intact ground and groundmasses is time dependent. There are several reasons that cause this

dependency, which can be divided into four phases with increasing timespan: (1) No strain can be instant after applying a stress. Instant strain would require an infinite velocity of the material particles, which is impossible. The particles have a certain mass, and displacement requires a certain timespan. If stress is applied, shock waves of stress-strain will travel through the ground. It will take some time, albeit in engineering terms very little, mostly in the order of microseconds to minutes, before stress and strain are again in equilibrium throughout the ground. Under slow application of stress, the shock wave effect may be minimal, but it will still take time before equilibrium between stress and strain is established. (2) All ground shows some (sometimes limited) effects of longer-term deformation. This is called “creep.” Figure 2e shows various options for strain versus time for long-term deformation under constant stress. Some materials deform with an increasing deformation rate leading to rapid failure (curve i in Fig. 2e). Others deform very slowly, and deformation rates may attenuate (curve iv), be steady state (curve iii), or re-accelerate after a long steady state period resulting in failure (curve ii). Creep is responsible for the delay in, for example, collapse of underground excavations. When a groundmass is loaded with a new stress environment due to excavation, the stress levels may not exceed the strength of the groundmass. Hence, the excavation will not fail. However, when stresses are near to the maximum stresses the

Mechanical Properties, Table 1 Example of deformation values

Material		Deformation modulus (D) (GPa) ¹	Poisson's ratio (ν) ¹
Soil			
Doha marine loose sand ^{2,a}		0.02	
Sand (Amsterdam) ^b		0.035–0.04	0.2
Residual soil & fill ^{b,c}		0.01–0.04	0.15
London clay (drained; depending on depth and direction) ^{3,d}		0.007–0.2	0.125
Aeschertunnel glacial till ^{4,e}		0.08	0.2
Clay (Amsterdam) ^b		0.01	0.15
Peat (Amsterdam) ^b		0.002	0.15
Frozen dense sand (artificially frozen, $T \sim -10^\circ\text{C}$) (short/long-term) ^{5,f}		0.75/0.33	0.3 ^g
Frozen stiff clay (artificially frozen, $T \sim -10^\circ\text{C}$) (short/long-term) ^{5,f}		0.3/0.12	0.006–0.13 ^h
Ice (natural fresh water ice; $T \sim -5^\circ\text{C}$) ^{5,i}		10	0.33
Man-made material			
Concrete (regular commercial, Portland cement, 28 days cured) ^{6,j}		27–35	0.2
Iron/steel ^k		200	0.3
Intact rock			
Hawkesbury sandstone ^l		6–14	0.15
Falset carboniferous sandstone ^m		35–60	0.1–0.2
Vinalmont limestone ⁿ		70	0.31
Sibbe limestone ⁿ		1.2	0.25
Königshain granite ^{7,o}	Slightly weathered	50	
	Moderately weathered	25	
	Highly weathered	15	
Äspö slightly fractured diorite and granite ^p		69–79	0.21–0.28
Basalt ^{8,q}		78	0.25
Gorleben salts (uniaxial/triaxial short-term laboratory tests; average of different formations) ^r		25/33	0.25–0.32
Rock mass			
Hawkesbury sandstone ^{9,1}		0.05–2.5	
Sheared flysch ^{9,v}		0.433	
Marly shale (standard zone) ^w		1.8	
Mu-cha tunnel fault (sheared sandstone and shale in clay matrix) ^{9,x}		0.2	0.3
Sydney-Gunnedah Basin coal ^{9,y}		2.5	0.24
Äspö slightly fractured diorite and granite ^{9,p}		55	0.26
Basalt ^{8,9,q}		10–40	0.3
Gorleben salts (short-term dilatometer tests; average of different formations) ^{10,r}		19	

Notes: Values reported are for normal (near-) surface engineering conditions. ¹“D” is the deformation modulus and “ ν ” the Poisson's ratio; values for 50% of the failure strength if reported. ²“Loose to dense” refer to the packing of the particles. ³Undrained and drained refer to the dissipation of pore gas and fluid pressures during loading; generally undrained applies to fast loading situations and drained for slow. ⁴Glacier deposit: clayey sand and silt, with gravel and isolated boulders. ⁵Values indicative only; strongly dependent on test conditions, deformation rate, compaction, temperature, structure, and quantity. ⁶Indicative only depending on type of concrete, aggregate, and cement type. ⁷Weathering description follow BS 5930: 1999 (1999). ⁸Summary literature typical values based on different basalts. ⁹Properties determined by rock mass classification and/or back analyses from tunnel construction. ¹⁰Dilatometer tests. Data from: ^aChen (2010), ^bBosch and Broere (2009), ^cChan and Stone (2005), ^dKarakus and Fowell (2005), ^eCoulter and Martin (2004), ^fJessberger et al. (2003), ^gKirsch and Richter (2009), ^hLee et al. (2002), ⁱSchulson (1999), ^jBamforth et al. (2008), ^kAshby and Jones (2012), ^lPells (2004), ^mKouokam (1993), ⁿSwart (1987), ^oThuro and Scholz (2004), ^pAndersson (2010), ^qSchultz (1995), ^rBräuer et al. (2011), ^sMarinos et al. (2009), ^tAlejano et al. (2008), ^xYu (1998), ^ySainsbury (2008)

groundmass can sustain, small microcracks may develop with time. The number of cracks will increase over time, and grow together, until, after some time, the groundmass fails. Similarly, in slope stability, loading a discontinuity with a shear stress may cause asperities on the plane to be stressed such that no immediate failure occurs, but with time microcracks form in the asperities which break after some time. The length of the period between stress loading and failure may range

from seconds to many years. (3) All grounds show a very long-term creep effect that is often also strongly dependent on temperature and confining stress. The ground deforms and after some time the ground may fail even if the ground is stressed well below the maximum sustainable stresses. The mechanisms for this effect are largely unknown, but it is thought that re-crystallization under stress and weathering may play a role. Long-term creep is likely responsible for

some collapses of excavations after long-time spans, sometimes after many hundreds of years. (4) On geological timespans, ground may show viscous flow.

Volumetric (Hydrostatic) Deformation

A body of ground will first compress due to setting and closure of fissures (small cracks) if it is under equal stress from all directions (Fig. 3), then a second phase of elastic deformation occurs for the particle skeleton. This is followed by a collapse of the pore structure in porous ground which effectively also destroys the skeleton structure. With continuing increase in stress, the particles in the ground will interlock and the amount of volume change with stress increase strongly reduces, resulting in a strong increase of the volumetric deformation modulus. Further increase in stress will cause the structures of particles and minerals to be destroyed and, at even higher stress levels, molecular and atomic structural changes. This high level of stress does not normally occur in engineering but may occur during underground nuclear tests. The bulk modulus (K) of volumetric deformation of ground is defined as:

$$\text{bulk modulus [Pa]} = K = \frac{\sigma_{\text{hydrostatic}}}{\Delta V/V} \quad (5)$$

$\sigma_{\text{hydrostatic}} = \text{hydrostatic stress [Pa]}$

$\Delta V = \text{change in volume [m}^3\text{]} \quad V = \text{volume [m}^3\text{]}$

For an intact, homogeneous, isotropic, and ideal-elastic solid, the bulk modulus is related to Young's and shear moduli as follows:

$$K = \frac{EG}{3(3G + E)} \quad (6)$$

During compression, gases and fluids are expelled from the ground if possible. If not, part of the stress is taken up by

the gases and fluids which will increase the overall deformation moduli and may prevent pore collapse. Note that a ground under equal stress is not really failing in the sense that a failure plane and a strength can be defined as discussed below.

Unconfined and Confined Compressive Strength

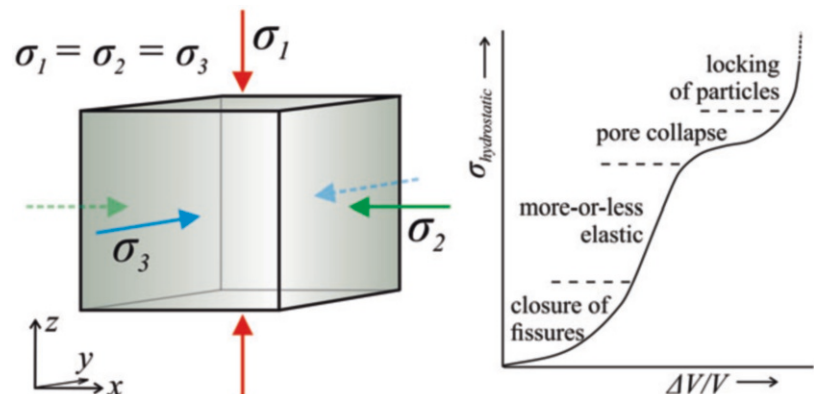
The compressive strength is the compressive stress at failure on a sample under a deviatoric normal compressive stress (σ_1). Compressive strength of ground material can be tested under different stress configurations. Depending on the type of test undertaken the compressive strength is denoted as Unconfined Compressive Strength (UCS), (normal) triaxial compressive strength, or true-triaxial compressive strength. Note that the tests are measuring the compressive strength but that the failure mode is actually due to stresses in the sample exceeding the shear or tensile (the latter sometimes referred to as splitting or bending) strength. Normally also the change in dimensions of the sample are measured during the test to obtain deformation characteristics. Triaxial and true-triaxial tests are mostly completed with pore pressure transducers allowing measurement of gas and fluid pressure in the sample during the test.

Unconfined Compressive Strength (UCS)

The Unconfined Compressive Strength (UCS) (also uniaxial strength) is the compressive stress (σ_1) measured at failure on a sample of ground under the condition that the confining pressure is zero ($\sigma_2 = \sigma_3 = 0$) (Fig. 4). The test is normally done on a cylinder sample, but can also be done on a cubic sample. Soil samples should have some form of attraction or gluing effect between particles; otherwise, the sample falls apart under its own weight. Alternatively, a sleeve or jacket of, for example, rubber is used to maintain sample integrity. The approximate failure plane is indicated in Fig. 4d; the

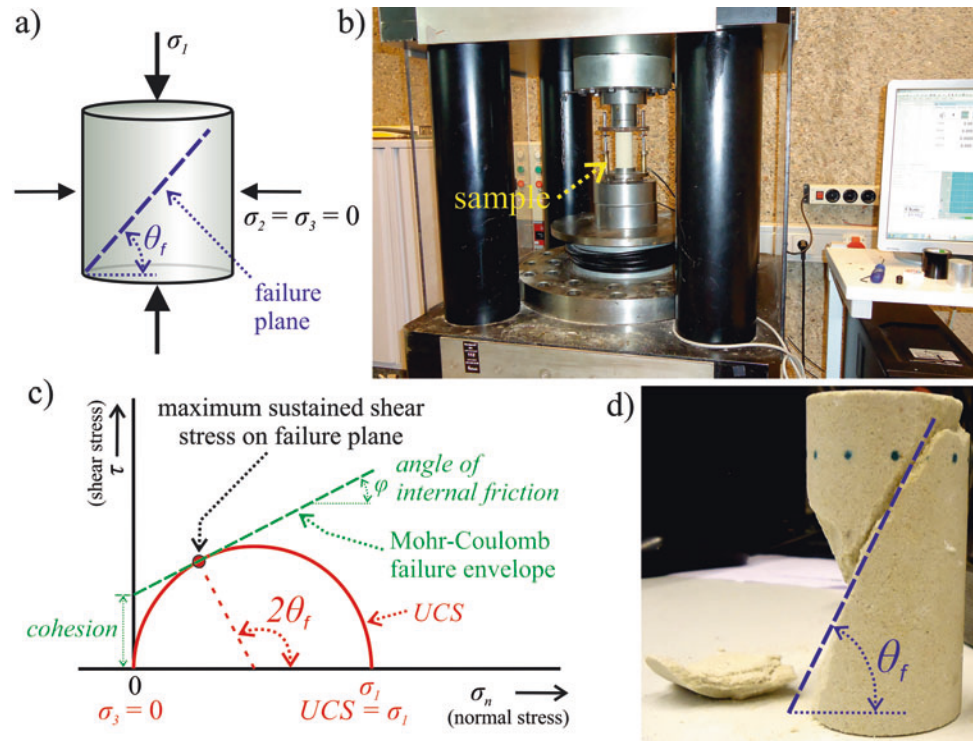
Mechanical Properties,

Fig. 3 Volumetric stress-strain of porous ground



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Fig. 4 Unconfined Compressive Strength (UCS) test on a cylinder sample; (a) sample and stress configuration; (b) test equipment; (c) stress configuration in a Mohr-circle diagram; (d) test result (Photos courtesy W. Verwaal, TU Delft, 2017)



angle may be slightly different from the failure plane angle indicated in Fig. 4c due to changes in failure plane area during the test. Figure 4c shows the failure state in the Mohr-circle diagram with the Mohr-Coulomb failure envelope. Examples of UCS values are listed in Table 2. Test standards and procedures are given in ASTM D7012-10 (2008) and Ulusay and Hudson (2007).

Triaxial Compressive Strength

In a triaxial test, the compressive stress (σ_1) is measured at the point failure of a sample that is under confining pressure (Fig. 5). The test is normally done on a cylinder sample in a sleeve of a foil of rubber (for soil) or metal, such as, copper or steel (for rock). The sample with sleeve is positioned in a pressure cell in which water or oil gives the confining pressure on the sleeve (Fig. 5b, c, e). The test is denoted a “normal triaxial test” but more commonly just “triaxial test”. Example values in terms of the Mohr-Coulomb failure envelope parameters are listed in Table 2. Test standards are given in ASTM D7012-10 (2008) and Ulusay and Hudson (2007). The confining pressure cell is made of glass or another transparent material for low-pressure tests, for example, on soil and very weak rock, and of steel for high-pressure tests on rock.

True-Triaxial Compressive Strength

The compressive stress (σ_1) is measured at the point of failure of a sample that is under confining pressure. In a true-triaxial

test, the confining pressure is not equal in the x and y directions ($\sigma_1 \neq \sigma_2 \neq \sigma_3$) (Fig. 6). The test is done to investigate the influence of the intermediate principal stress (σ_2). A true-triaxial test apparatus is technically highly complicated. The test is normally done on cube or rectangular prism samples, where platens compress the sample from three perpendicular directions independently. The whole test setup can be placed in a pressure cell to allow for pore pressures. Various solutions are used to solve technical problems that arise from the reduction in size or change in shape of the deforming sample, whereas the test platens cannot easily change size or shape.

Tensile Strength; Direct Tensile Strength (DTS) and Brazilian Tensile Strength (BTS)

Intact ground with some attraction or gluing between grains or particles has a tensile strength. The tensile strength can be established by a confined or unconfined Direct Tensile Strength (DTS) test (Fig. 7a, d) or by an indirect tensile strength test, including the Brazilian (or indirect or splitting) Tensile Strength (BTS) test (Fig. 7b, e). The DTS can be done in a confining pressure cell. Standards and test procedures are given in ASTM D3967-08 (2008), ASTM D2936-08 (2008), and Ulusay and Hudson (2007). The tensile strength plots negative in the Mohr’s circle diagram (Fig. 7c). The BTS is established by compressing a disk of the ground. The disk will fail by induced internal tensile stress. The Brazilian Tensile Strength (BTS) is then:

$$\sigma_{t \text{ Brazilian}} = \frac{2P}{\pi Dt} \quad (7)$$

$\sigma_{t \text{ Brazilian}}$ = Brazilian Tensile Strength (BTS) [Pa]
 P = load on sample at failure [N]
 D = diameter [m] t = thickness [m]

Estimation of the tensile strength of ground is notoriously unreliable. Small inhomogeneities or small cracks, which are often invisible, may decrease the tensile strength considerably.

Additionally, failure in a tensile stress environment is a mechanism propelling itself. If failure starts, the tensile force must be taken by the remaining not yet failed part, increasing the tensile stress in that volume. This volume is stressed even more and fails faster. Tensile failure is a (very) rapid process and happens normally with little warning. For these reasons, the tensile strength of intact rock is not or only partially considered in design in rock mechanics. Examples of intact tensile strength are listed in Table 2.

Mechanical Properties, Table 2 Examples of strength values

Name	UCS (MPa)	TS (MPa)	ϕ' (degrees)	cohesion' (MPa)	range of confining pressure (MPa)
Soil					
Sand (rounded particles) (loose to dense) ^{1,a}			27–35	0	<0.5
Sand (angular particles) (loose to dense) ^{1,a}			33–45	0	
Sandy gravel (loose to dense) ^{1,a}			35–50	0	
Residual soil & fill ^{b,c}			25	0	<0.1
Clay (undrained) (very soft to hard) ^{2,3,d}			0	0.01–0.3	<0.5
Clay (drained) ^{3,a,c}			20–35	0–0.15	
Peat (drained) ^{3,c}			20	0.006	<0.15
Frozen dense sand (artificially frozen, T~−10°C) (short/long-term) ^{4,e,f}	7/4	20–50% of UCS	38/22	2/1.4	<1.4
Frozen stiff clay (artificially frozen, T~−10°C) (short/long-term) ^{4,e,f}	6/1.5		1.5/7.5	0.8/0.6	
Ice (natural fresh water ice) ^{4,5,g,h}	1–18	1.3	25–48 ⁱ	0.25 ⁱ	<0.2 ⁱ
Man-made material					
Concrete (regular commercial, normal strength, Portland, 28 days cured) ^{6,j}	15–40	2–5	43 ^{7,k,l}	5.5 ^{7,k,l}	<17
Iron (yield strength) ^m	50	200			
Steel (low-alloy) (yield strength) ^m	500–1,980	680–2,400			
Intact rock					
Vindhyan sandstone ⁿ	102	6.9	37	33	<15
Hawkesbury sandstone ^o	21–60	3.5	47	4.2	<20
Eagle Ford Shale ^p	2.1	0.93	24	0.41	<3
Yucca Mountain-Topopah Spring Tuff ^q					
(TSw2/Tptpmn)	187	11.6	48	40	<15
(TSw3/Tptpv)	16	4.0	47	3.5	<10
Vinalmont limestone ^r	190	7			
Sibbe limestone ^r	3.5	0.38			
Basalt ^{8,s}	266	14.5	31	66	3.4–34.5
Königshain Granite ^{9,t}					
Slightly weathered	185	18			
Moderately weathered	38	3.5			
Highly weathered	13	1			
Completely weathered			25	0.015	<0.4
Residual soil			35	0.025	
Gorleben salts (short-term laboratory tests; average of different formations) ^u	26	1.6			
Rock mass					
Sheared flysch ^{10,v}	4.5		16	0.073	~ 2.5
Mu-Cha Tunnel Fault (sheared sandstone & shale in clay matrix) ^{10,w}	2.6		28	0.1	~ 3.5
Basalt ^{8,10,s}				0.6–6	3.4 – 34.5
Deriner Granodiorite ^{10,x}	80	16	40	0.35	<1.5

(continued)

Mechanical Properties, Table 2 (continued)

Name	UCS (MPa)	TS (MPa)	ϕ' (degrees)	cohesion' (MPa)	range of confining pressure (MPa)
Falset Granodiorite ^{9,11,y}					<0.6
Fresh (zone 1)	175	10.2	47	0.017	
Slightly weathered (zone 1–2)	110	4.1	46	0.016	
Moderately weathered (zone 3)	80	2.7	38	0.014	
Highly weathered (zone 4)	3		17	0.008	
Completely weathered (zone 5)	0.5		6	0.003	
Falset Lower Muschelkalk Limestone ^{11,12,y}					
Large blocky	80	8 ^z	62	0.027	
Small blocky	70	8 ^z	18	0.007	
Sydney-Gunnedah Basin coal ^{10,aa}		0.8	38	1.9	~ 12.5

Notes: Note the large variation between different materials, the wide variation within the same material due to different states of weathering, and the large influence of block size on mass properties while the intact material strengths (*UCS* and *TS*) are about the same. Values reported are for normal (near) surface engineering conditions. For groundmasses, the *UCS* and *TS* are of the intact ground. ϕ' and *cohesion'* are effective Mohr-Coulomb failure envelope parameters with the range of confining pressures as the properties are validated within the given range only. Not all literature reports whether effective or total ϕ and *cohesion* are reported, but the test conditions imply effective, except where otherwise indicated. ¹“Loose to dense” refers to the packing of the particles (following BS5930), which influences the ϕ' values. ²“very soft to hard” refers to the consistency of the clay BS 5930: 1999 (1999). ³“Undrained” and “drained” refer to the dissipation of pore gas and fluid pressures during loading; generally, undrained applies to fast loading situations and drained for slow. Values reported for undrained cohesion are “*Su*” (or “*cu*”) and ϕ and *cohesion* are total stress Mohr-Coulomb failure envelope parameters (i.e., without accent). ⁴Values indicative only; strongly dependent on test conditions, deformation rate, compaction, temperature, structure, and quantity. Strength is the highest stress sustained in the test; highest values for fast loading. ⁵*UCS* and *TS* at temperature -1°C to -16°C ; shear properties at -2°C ; shear samples contain ground fragments and air bubbles. ⁶Concrete strength values are indications for “regular” commercially available concrete without additives or reinforcement. *UCS* of commercially available “high-strength” concrete is 40–150 MPa, up to 200 MPa with *TS* up to 9 MPa for “ultra-high strength” concrete (UHC 2006). *UCS* for special purposes concrete (e.g., military) may be over 800 MPa. ⁷*Cohesion'* and ϕ' depend on the strength combination of aggregate and cement matrix; i.e., does the material shear through low-shear strength aggregate or are high-shear strength aggregate particles overridden. The values in the table are therefore only indicative of a particular combination of aggregate and cement matrix. The values are based on combined regression of data from various authors. ⁸Summary literature for typical values based on different basalts. ⁹Weathering classifiers indicating an increasing grade of weathering from fresh to slightly weathered, etc., follow ISO 14689-1: 2003 (2003). The zones follow BS 5930: 1999 (1999) and not the replacement standard ISO 14689-1: 2003 (2003) as the replacement is at present considered by some to be inferior to the BS 5930 (Price et al. 2009). ¹⁰ ϕ' and *cohesion'* determined by rock mass classification and/or back analyses from tunnel construction. ¹¹ ϕ' and *cohesion'* back analyzed from slope engineering. ¹²“Large blocky” implies that most of the blocks in the rock mass are about equi-dimensional with sides between 0.6 and 2 m, while “small blocky” implies equi-dimensional with sides between 6 and 20 cm (ISO 14689-1: 2003 2003). Data from: ^aCraig (2004), ^bChan et al. (2005), ^cBosch and Broere (2009), ^dBS 5930: 1999 (1999), ^eJessberger et al. (2003), ^fZhang et al. (2007), ^gSchulson (1999), ^hGagnon and Gammon (1995), ⁱArenson et al. (2003), ^jBamforth et al. (2008), ^kSonnenberg et al. (2003), ^lWong et al. (2007), ^mAshby and Jones (2012), ⁿDubey (2006), ^oPells (2004), ^pHsu and Nelson (2002), ^qCiancia and Heiken (2006), ^rSwart (1987), ^sSchultz (1995), ^tThuro and Scholz (2004), ^uBräuer et al. (2011), ^vMarinos et al. (2009), ^wYu (1998), ^xCekerevac et al. (2009), ^yHack (1998), ^zKouokam (1993), ^{aa}Sainsbury (2008)

Point Load Strength (PLS)

Point Load Strength (*PLS*) tests have been developed to get an idea about the strength of a piece of rock with little effort. The test can be done on lumps of rock or borehole cores with a size of about 50 mm. The sample is placed between two standardized steel “points” (Fig. 8). The two points are moved together by mechanical or hydraulic loading until the sample breaks. The maximum force (*P*) at failure divided by D^2 is “*Is*.” The *Is* value has to be corrected if the sample size differs much from 50 mm. The procedure can be found in ASTM D5731-08 (2008) and Ulusay and Hudson (2007). Normally the result is denoted “*Is50*” or “*PLS*.” The *PLS* test is not intended as a replacement for Unconfined Compressive Strength (*UCS*) testing. The test is very sensitive for irregularities and inhomogeneities in the sample, such as discontinuities. In particular, if these are present near one of the points, the *PLS* value is significantly reduced. Notwithstanding this, various

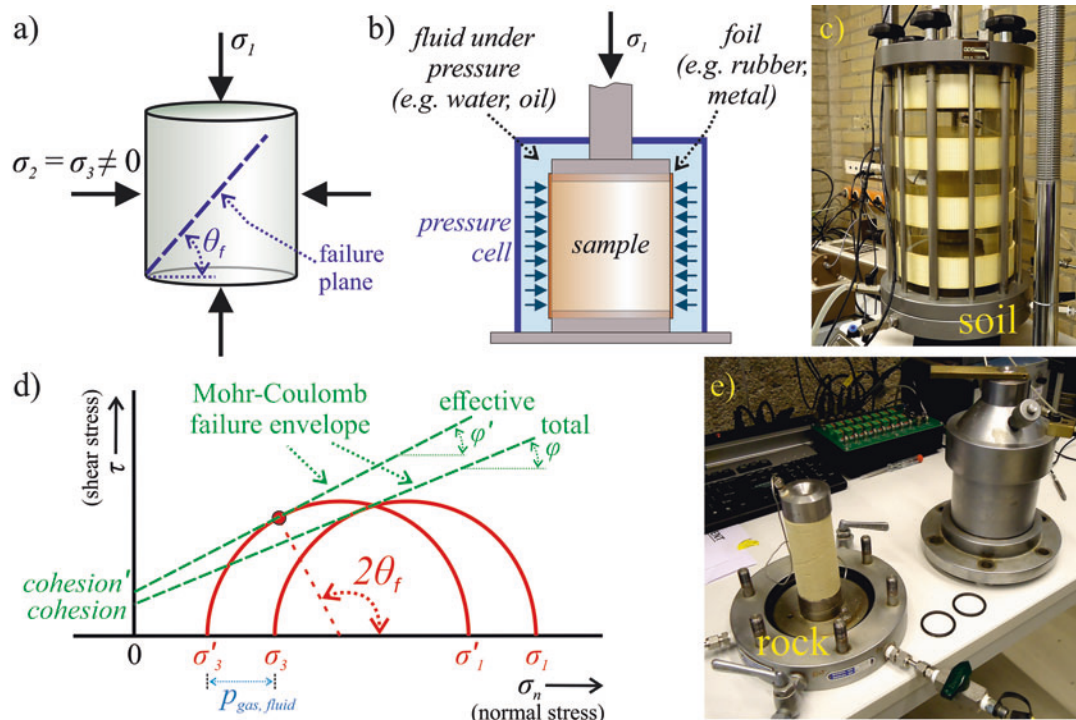
relations have been proposed between *PLS* and *UCS* (Rusnak and Mark 2000); a commonly used, but in the opinion of the author unreliable, relation is (Bieniawski 1975; Broch and Franklin 1972):

$$UCS = 24 \times Is_{50} \quad (8)$$

The author does not regard the *PLS* test as particularly useful, and not much if any better than the faster, and simpler to execute, rebound and “Simple Means” tests below.

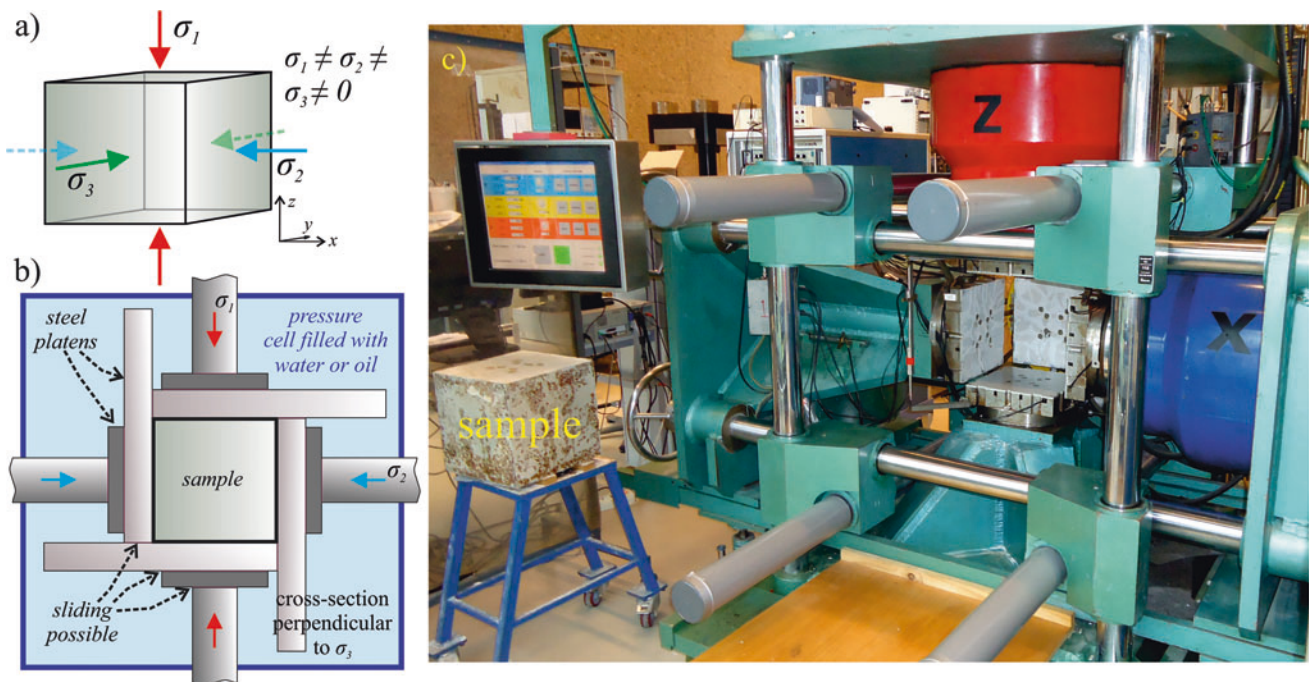
Impact and Rebound Tests; Schmidt Hammer, Ball Rebound, and Equotip

Rebound measurements are based on a piston or ball that drops from a certain height onto the surface of the material

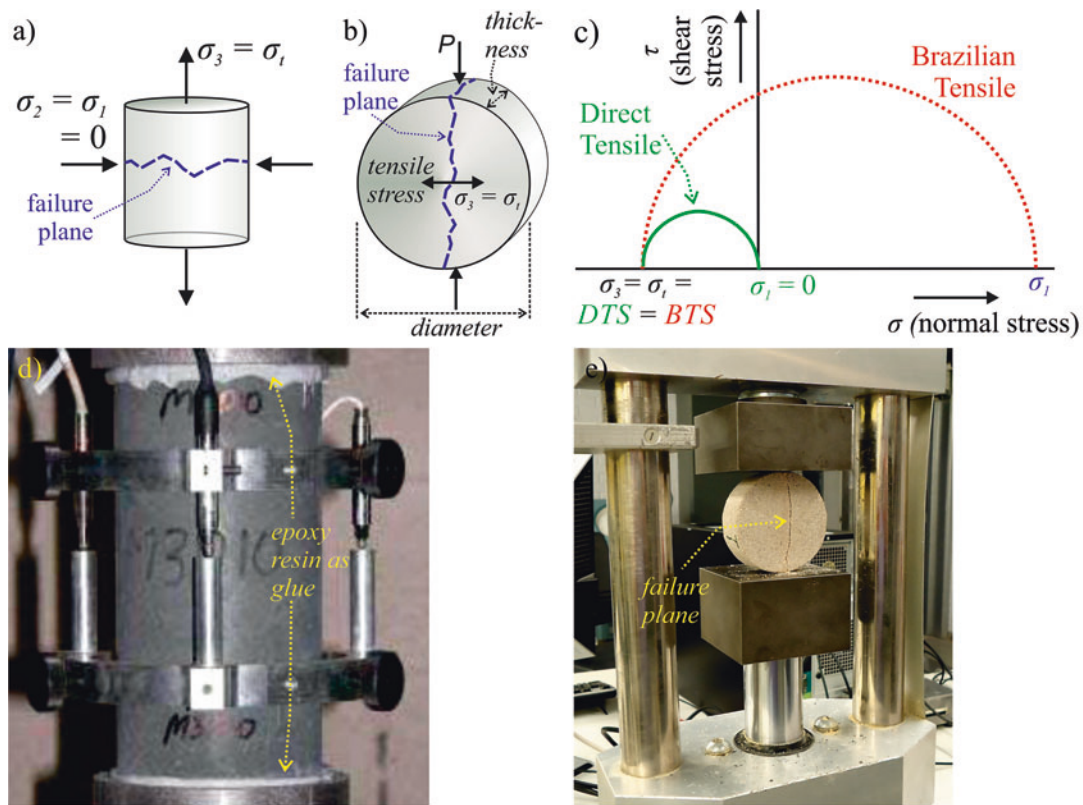


Mechanical Properties, Fig. 5 (Normal) Triaxial test (a, b) sample and stress configuration; (c) transparent pressure cell for soil; (d) stress configuration in a Mohr-circle diagram in terms of total and

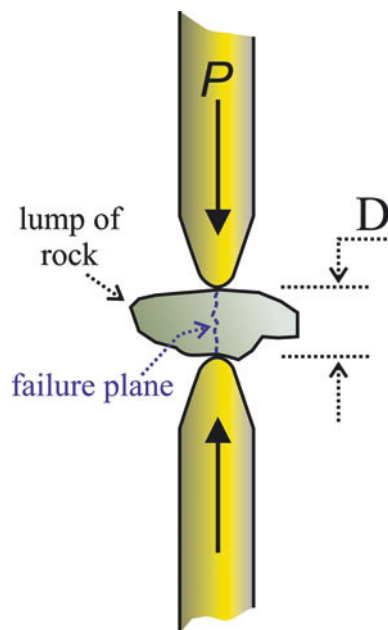
effective stresses; (e) disassembled pressure cell for rock, left bottom plate with sample, right steel upper part (Photos courtesy W. Verwaal, TU Delft, 2017)



Mechanical Properties, Fig. 6 True-triaxial test; (a, b) sample and stress configuration; (c) test equipment (x and z are two of the three pressure jacks) (Photo courtesy W. Verwaal, TU Delft, 2017)

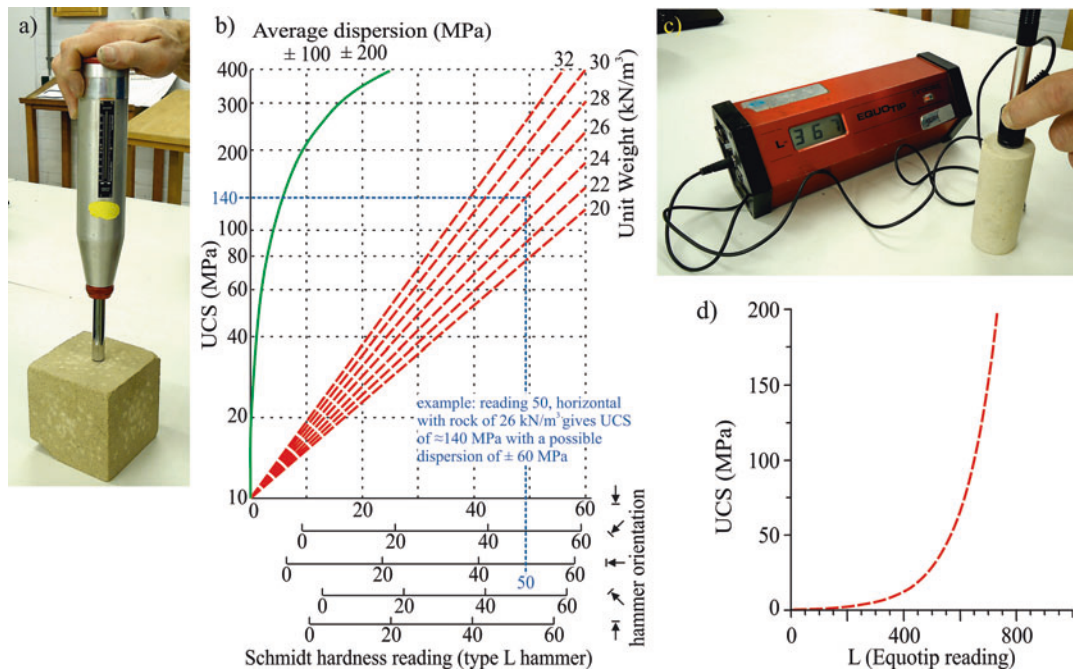


Mechanical Properties, Fig. 7 Tensile strength; (a, d) Direct Tensile Strength (DTS); (b, e) Brazilian Tensile Strength (BTS); (c) stress configuration in a Mohr-circle diagram (Photos courtesy W. Verwaal, TU Delft, 2017)



Mechanical Properties, Fig. 8 Point Load Strength (PLS)

to be measured. The rebound of the piston or ball after hitting the surface depends on the elastic parameters of the tested material and on the strength of the material at the surface. The crushing of surface asperities and surface material, which dissipates energy, causes this latter effect. Rebound measurement apparatus are the Schmidt Hammer which was originally developed for testing concrete quality (Fig. 9a, b) (Schmidt 1951; ASTM C805/C805M-13 2013; ASTM D5873-14 2014), the Equotip developed for steel testing (Fig. 9c, d) (Equotip 2018), and ball rebound (Pool 1981). The rebound values on rock surfaces have been correlated with intact rock strength (Fig. 9b, d) (Deere and Miller 1966; Stimpson 1965; Pool 1981; Verwaal and Mulder 1993; Hack et al. 1993; Hoek 2017; Ulusay and Hudson 2007). The execution of the test damages the rock at the impact point; asperities are crushed, and generally, the rock material will be compressed. Therefore, repeated impacts on the same location show increasing values. The tests are also influenced by local differences in structure and texture, presence of fluids (water), asperities (roughness of the surface), loose material on the surface, and in particular, a discontinuity beneath the



Mechanical Properties, Fig. 9 (a) Schmidt Hammer; (b) conversion Schmidt Hammer to UCS (Modified from Hoek 2017); (c) Equotip equipment; (d) Equotip conversion to UCS (Modified from Verwaal and Mulder 1993) (Photos courtesy A. Mulder, TU Delft, 2017)

surface. Schmidt hammer values are influenced by the material to a fairly large depth (order of centimeters) beneath the surface whereas ball rebound and Equotip release considerably less energy and are influenced by a thinner layer of material (order of millimeters).

“Simple Means” Intact Rock Strength Field Estimates

“Simple means” field tests make use of hand pressure, geological hammer, etc., to estimate the strength of cohesive soil and intact rock in classes following the British and ISO standards (BS 5930: 1981 (1981); BS 5930: 1999 (1999); ISO 14689-1: 2003 (2003)) (Table 3). Extensive numbers of tests allowed a thorough analysis of the accuracy and reliability of the simple means field tests for estimating intact rock strength (Fig. 10).

Shear Strength Tests

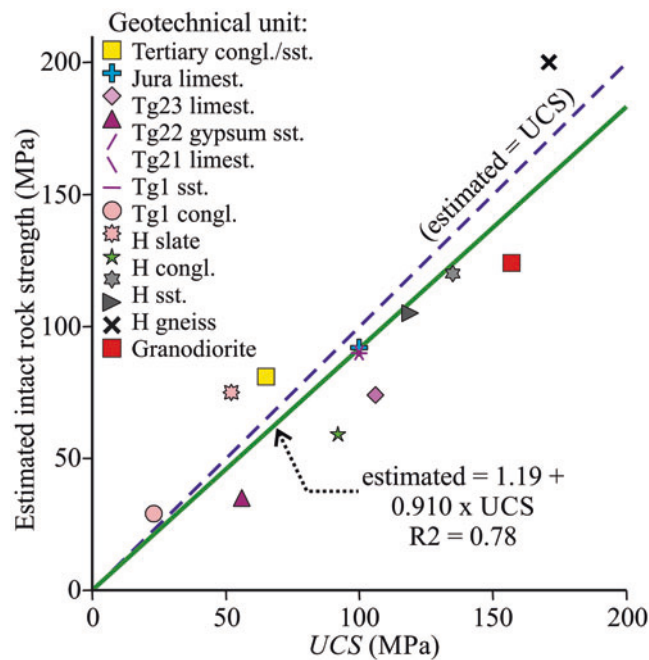
Direct shear strength of discontinuities and ground material can be established in a “Golden shear box,” respectively, a “direct shear testing apparatus” (Fig. 11). The sample is mounted in two half steel boxes and opposite forces are

applied to the two steel boxes. The horizontal displacement ($\delta_{\text{horizontal}}$) and vertical opening (δ_{vertical}) between the two steel box halves are measured during the test. This allows the “angle of roughness” of the failure plane of a discontinuity, or the “dilatancy” under shear displacement of intact ground material to be established. Standards for testing are ASTM D3080M-11 (2011); ASTM D5607-08 (2008) and to be found in Ulusay and Hudson (2007). The shear test can also be executed as a “ring shear test” (Bromhead 1979; ASTM D6467-13 2013). In the latter test, the two halves of the test box are not translated but rotated. The advantage of the ring over a direct shear test is the potentially unlimited displacement. This makes the test particularly suitable for measuring residual shear strength properties or for determining how the shear strength changes during transition from undisturbed to remolded material. The Golden, direct, or ring test equipment can be placed in a fluid (e.g., water, oil) tank with fluid under pressure and a pore pressure transducer incorporated in the sample near to the discontinuity or the expected failure surface to measure the pore pressure. Shear strength of intact rock that is more than “very weak” in strength is seldom established by direct or ring-shear, but mostly by triaxial or true-triaxial testing, because the necessary stresses are so high that testing equipment is very heavy and expensive. For further details, refer to chapters on Rock Field Tests and Soil Field Tests.

Mechanical Properties, Table 3 “Simple means” field test for estimating strength (ISO 14689-1: 2003 2003; BS 5930: 1999 1999)

Cohesive soil (BS 5930: 1999)			Intact rock (ISO 14689-1 2003)		
Term	Field identification	Undrained compressive strength (kPa)	Term	Field identification	Unconfined compressive strength (MPa)
Very soft	Finger easily pushed in up to 25 mm	0–20	Extremely weak	Indented by thumbnail	<1
Soft	Finger pushed in up to 10 mm	20–40	Very weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1–5
Firm	Thumb makes impression easily	40–75	Weak	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5–25
Stiff	Can be indented slightly by thumb	75–150	Medium strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25–50
Very stiff	Can be indented by thumbnail	150–300	Strong	Specimen requires more than one blow of geological hammer to fracture it	50–100
Hard or very weak mudstone	Can be scratched by thumbnail	>300	Very strong	Specimen requires many blows of geological hammer to fracture it	100–250
			Extremely strong	Specimen can only be chipped with geological hammer	>250

Notes: Some extremely weak rocks will behave as soils and should be described as soils. Geological hammer should be about 1 kg. Blows by the point of a geological hammer should be done with care as the hammer may break on impact with an unexpected harder item (e.g., a piece of quartz)

**Mechanical Properties, Fig. 10** Correlation between “simple means” estimate and UCS (Modified from Hack and Huisman 2002)

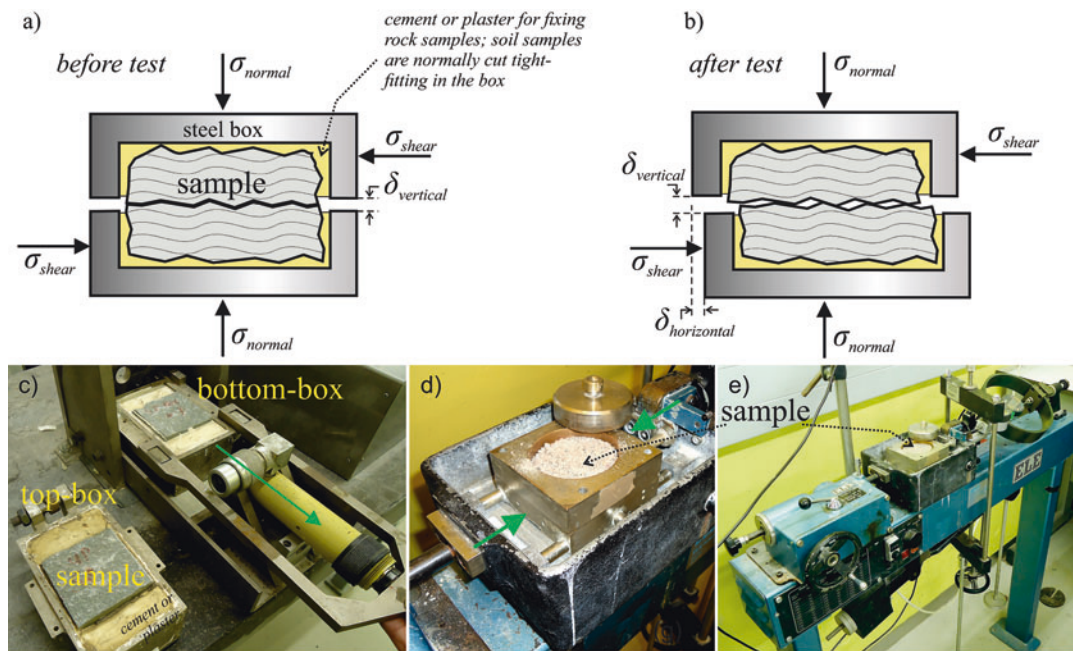
Constitutive Models

Constitutive models are the aggregate term for relations that describe mathematically the chemical-physical-mechanical

behavior of the ground or groundmass, normally the relation between the parameters stress, strain, strength, time, and temperature (Wang and Huang 2009; Yang et al. 2013). They may also encompass parameters such as electricity, magnetism, and nuclear radiation. Equations 1 and 2 are examples of simple constitutive models as is the Mohr-Coulomb failure envelope. Many hundreds of constitutive models for ground and groundmasses are defined; some highly complicated, for example, time and temperature-dependent viscous behavior of an anisotropic discontinuous groundmass with gases and fluids in which nuclear material is stored (e.g., Karato 2012; Cai and Horii 1992; Wang and Huang 2009). Many models for groundmasses are (in part) based on rock mass classification (Marinos and Hoek 2000; Hack et al. 2003; Barton 2002; Bieniawski 1989; Price et al. 2009). Description of these is outside the scope of this chapter.

Test Procedures, Standards and Codes of Practice

The results of the various tests mentioned in this chapter depend often on factors such as sample size and form (i.e., cylindrical, rectangular, or cubical), methodology of testing (e.g., rate of stress increase), environment temperature, and gas or fluid content. The dependencies mean that results can be compared and be applied in design of engineering structures only if the tests are done strictly following a particular



Mechanical Properties, Fig. 11 (a–c) Golden shear box test for shear strength of a discontinuity; during the test the top-box is installed on top of the bottom-box and fixed in horizontal direction, the bottom-box is

pulled from underneath the top-box (green arrow); (d, e) Direct shear box test with detail of the direct sample box (Photos courtesy W. Verwaal, TU Delft, 2017)

test procedure as described in “standards” or “codes of practice,” for instance, standards of the International Standard Organization (ISO). Often test results depend to a certain extent on the method of testing; therefore, the test conditions should be as much as possible similar to those that will exist where applied, for example, the confining stress in a triaxial test should be similar to the stress environment *in situ* where the test result is applied.

Summary

Mechanical properties of ground are diverse and numerous. Although tests have been designed for all properties, inhomogeneity of the ground, restricted size of laboratory samples, and high costs of full-scale testing often prohibits a complete characterization of the ground by tests alone. Expert judgement and estimation of ground characteristics and properties is often as, if not more, important than testing.

Cross-References

- [Angle of Internal Friction](#)
- [Biological Weathering](#)
- [Chemical Weathering](#)
- [Consolidation](#)
- [Deviatoric Stress](#)
- [Dilatancy](#)

- [Mohr Circle](#)
- [Mohr-Coulomb Failure Envelope](#)
- [Physical Weathering](#)
- [Rock Field Tests](#)
- [Rock Mass Classification](#)
- [Shear Modulus](#)

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Metamorphic Rocks

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Definition

Rocks derived from other pre-existing rocks that, in the course of geological processes, have undergone mineralogical, chemical, and structural changes in the solid state, in response to the changes in physical and chemical conditions existing at depth.

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