

This is Chapter 3 “Some properties of rocks and rock masses” including the related Appendix B “Some mechanical properties of rocks” from my PhD thesis

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Enjoy reading.

Best regards,
Alexander Kluckner



Dipl.-Ing. Alexander Kluckner, BSc

**Tunnelling at greater depths:
Study on the ground and system behaviour
when passing a stiff rock block in a weak zone**

DOCTORAL THESIS

to achieve the university degree of
Doktor der technischen Wissenschaften

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Chapter 3

Some properties of rocks and rock masses

The studies in Chapter 6 (p. 81) and Chapter 7 (p. 159) simulate a tunnel drive through a weak zone of matrix material comprising one stiff block next to the tunnel. This chapter here reports about information from literature on geometric properties of blocks and mechanical properties of block and matrix material. Since the studies aim to consider for a wide range of material parameter combinations existing in nature, many sections below cite properties and parameter relationships not relating to bimrock but to rocks in general. Because some fault zone material is soil-like, also some soil parameters are listed. The parameter relationships help to identify reasonable parameter combinations for the parametric study in Chapter 6.

The information on material properties cited in this chapter is limited to results from tests under drained conditions. Because the thesis focusses on scenarios where the block is stronger and stiffer than the matrix material, data from contrary cases (i.e., matrix material is stronger and stiffer than the block; cf., e.g., [9]) is not considered here. And this chapter does also not describe the overall strength and stiffness of bimrock and related determinants (e.g., volumetric block proportion), because here, it is all about a single-block scenario. For the former, refer to, e.g., [306] and literature cited therein.

Section 3.1 lists information on the geometric properties of bimrock blocks. And Section 3.2 (p. 22) identifies ranges of some mechanical properties of rocks and rock masses.

3.1 Geometric properties of bimrock blocks

The following subsections give some information on the shape, location and orientation, and size of blocks of block-in-matrix rocks (bimrocks).

3.1.1 Block shape

The more competent, less strained rock bodies embedded within a matrix often show an ellipsoidal, lenticular, or prolate shape ([114, p. 184], [254, p. 264]). Such a rock body (i.e., block) forms when shears of various types link with each other ([114, p. 184]; cf. also Section 2.2 on p. 14). In subsequent shear movements, the block may wear and round off. If it has been detached from the host rock, during further shearing it may also translate and rotate within the matrix.

Which block shape eventually results probably depends on many factors, some of which might be: initial shape defined by the shears, stress conditions, thickness and structure of the zone

within which the block is embedded (determines, e.g., the degree of freedom regarding a block rotation), and location of concentrated shear within the zone. Some of these factors vary in time. Hence, depending on the history a block has experienced, many shapes seem possible. [250, p. 22], for example, report that large blocks in Franciscan *mélange* can feature a smoothly ellipsoidal but also an irregular shape. Random shape of blocks (with typically rounded edges) in serpentinite bimrock was reported by [127, p. 55]. The two examples of bimrocks different in scale given in Fig. 3.1 highlight the variability in the block shapes within each imaged bimrock zone ranging from compact to elongated.

In case of Franciscan *mélange*, the aspect ratio (i.e., major axis length to minor axis length) of some mapped large blocks ranges from 1 to 3, and is 2 on average ([255, p. 74]). [255, Fig. 3.12, p. 74] determined a similar average ratio for some blocks in sheared serpentinite. [317, p. 23] reports an average aspect ratio of 3.6 of blocks in phyllitic matrix mapped at the north tube of the tunnel *Spital* between chainage 933.4 m and 1094.7 m (cf. Fig. 3.2). Blocks with an initially larger aspect ratio most probably fail during the fault zone's evolution due to bending. In line, [227] (cited in [255, p. 74]) fabricated blocks with an aspect ratio of 2 to 3 for his tests on artificial bimrock specimens; [306, p. 54] used ellipsoids with a ratio of 2.3 to 2.5 for his laboratory tests; and [23] modelled ellipsoids with a ratio of 2 in their two-dimensional numerical studies.¹



Figure 3.1: Images of exemplary bimrocks: (a) Transition from blocky rock mass into bimrock (central part of picture); mesoscale (from [324, Fig. 7, p. 18]). (b) Franciscan *mélange* showing anisotropic fabric of elongated blocks entrained within sheared shale matrix; scale bar is 1.5 m (from [255] graphed in [253, Fig. 1, p. 600]).

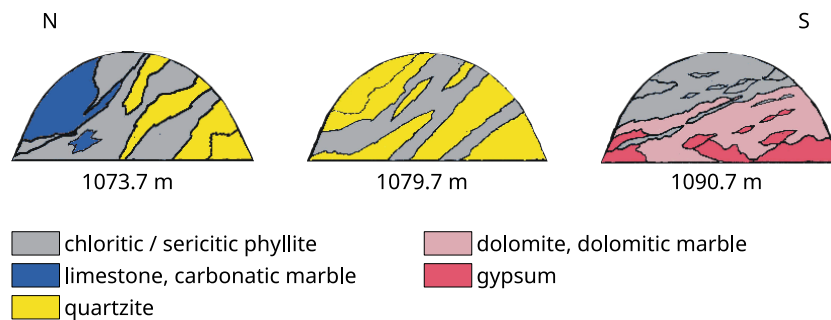


Figure 3.2: Sketches of the observed geological conditions at tunnel faces at chainage 1073.9 m, 1079.7 m, and 1090.7 m of the north tube of the tunnel *Spital*, Austria (modified from [317, Fig. 10, p. 13]). In the graphs, the phyllites (coloured in grey) are the weak matrix material.

¹In [255, p. B-6], the major axis is the longest line that can be drawn within the block, and the minor axis is the longest line that can be drawn normal to the major axis. [317, p. 23] fitted ellipsoids to the mapped rock blocks. So, the authors used different approaches to determine the block size and shape. Thus, given aspect ratios cannot be compared directly.

3.1.2 Block location and orientation

The incorporation of competent blocks (i.e., rock fragments or lenses) into the fault zone can happen when progressing faults (through the host rock) link at dilational jogs² and form relays³ (cf. also previous subsection). These relays then surround blocks of relatively undeformed protolith. [420, p. 9]

Relays are shears that accommodated much displacement and comprise fault gouges or other highly strained fault rocks ([420, p. 9]). Commonly, blocks entrained within the shears are oriented subparallel to these ([254, p. 268], [88] in [227, p. 4]). Shears can cross through a fault zone in a tortuous way ([254, p. 268]). For example, because of linkage with other shears, or because the path along which they progress changes at a tectonic event different in its characteristics to a previous one. Hence, considering the complexity a fault zone can have, the orientation of blocks can change abruptly from one place to another within the fault zone ([254, p. 268]). If the matrix features a well-developed foliation, traces of fault systems and related shear zones, and, therefore, blocks embedded in those systems or zones, may align sub-parallel to the foliation planes (c.f., e.g., [317, p. 12ff, 23]).

Depending on the fault zone's evolutionary history, the block may be still next to the host rock walls or far away from them closer to the damage zone or fault core centre.

3.1.3 Block size

Block sizes in fault zones can range between millimetres and hundreds of metres. The block size distribution in brittle faults is often fractal (i.e., conforming to negative power laws; [254, p. 266]). That means that at a specific scale, a fault or a fault zone comprises "relatively few large blocks and increasing numbers of ever diminishing smaller blocks". [324, p. 17f]

At a different scale, a similar picture results. Hence, block size distributions are scale-independent (probably between some natural upper and lower fractal limits), showing a self-repeating pattern at different scales: small blocks at one scale of interest are part of the matrix at the smaller scale ([254, p. 265f]). [256, p. 911] report about the scale-independency of Franciscan mélange over seven orders of magnitude regardless of the scale of measurement or the nature of the mélange's fabric.

According to [255], because of the scale-independency of block size distributions, for each engineering problem dealing with bimrocks, a scale of interest must be specified. For this purpose, he introduced the characteristic engineering dimension, ced , being, for example, the length of a straight line through an object or space, or the square root of an area of interest, $\sqrt{A}$. Depending on the problem at hand, this can be the slope height, the size of the yield zone around openings, or the dimensions of laboratory test specimens ([255, p. 3]). Now, to discriminate between matrix material, blocks, and blocky rock mass for a given zone, [255] further introduced two threshold values for the block size:

- $d_{max} < 0.75 \cdot ced$: Blocks with a size below this threshold are treated as single blocks of the bimrock. Is the block larger, then the block takes up almost the entire area (or volume) of interest, and the zone should be treated as blocky rock mass with matrix-filled joints.
- $d_{min} > 0.05 \cdot ced$: Blocks with a size below this threshold can be seen as part of the matrix

²Jogs (or bends, or ramps) are curved parts of a continuous fault trace that connect two noncoplanar but approximately parallel segments of faults ([397, p. 86, 193]). Such jogs can be of compressional or dilational nature (cf. Fig. 5b in [373, p. 1035]).

³Along relays (a fault, or a zone) displacement (or slip) is *transferred* from the end of one fault to an adjacent parallel fault ([397, p. 86]).

material. They may represent up to 95% of the total number of blocks but less than 1% of the total volume of blocks and, therefore, contribute little to the strength of the bimrock ([255, p. 31]). Larger blocks are treated as single blocks of the bimrock.

Here, d_{max} is the size⁴ of the largest geotechnically significant block in the given population, and d_{min} is the size of the smallest block. One can imagine that at a laboratory test sample with, for example, a maximum dimension of 15 cm, a block with contrasting mechanical properties but a size of less than $0.05 \cdot 15 = 0.75$ cm may not alter the overall behaviour of the sample much during testing. However, a block with a size of 5 cm will. And in case the block features a size of greater than $0.75 \cdot 15 = 11.25$ cm (maximum observed dimension at the test sample; d_{mod} in [255]), it makes up almost the whole sample and one cannot talk about a block-in-matrix structure here anymore. Similar considerations apply to a tunnel surrounded by a yielded zone. Note that depending on the scale of interest, e.g., a rock boulder⁵ or a sand grain⁶ can be declared as a *block*. According to [254, p. 267, 274], "the most conservative block-matrix threshold that can be justified should be selected"; and for the characteristic engineering dimension, ced , it is the area or dimension within which most likely the critical mode of failure will occur. Note that the selected thresholds for the block size then decide upon the number of blocks the investigation rock or fault zone mass comprises.

[255] identified the threshold values above analysing outcrops and geological maps of several Franciscan mélanges with the measurement areas ranging from 0.01 m² to 1000 km² (cf. Fig. 2.8 in [255, p. 27]). Considering that other bimrocks also exhibit scale-independent (or self-similar) block size distributions—e.g., fault gouges ([338] in [254, p. 266]), Italian olistostromes ([79] in [254, p. 265]), or carbonate and quartzite blocks within phyllitic matrix ([317, p. 61])—the threshold values probably apply to them as well.

3.2 Mechanical properties of rocks and rock masses

Since block-in-matrix rocks (bimrocks) comprise blocks (strong rock) and matrix material (weak rock or rock mass), the following subsections cite some general information on the mechanical properties of rocks and rock masses, and some specifics on bimrock. The information is required to identify suitable material parameters for the numerical studies.

Note that more information is available on fault rocks and fault-zone masses than on bimrock. Anyway, depending on the scale of interest, the former can make up the matrix material of a bimrock. Because the parametric study relates the block strength and stiffness to the matrix strength and stiffness, attention is paid on the matrix material and on general parameter relationships.

If not state otherwise, parameters cited below refer to rock and not to rock mass. Bimrock is a rock mass. Its constituents, the block and the matrix, are rocks. Fault zone material is also considered as rock, unless it is homogenised with, e.g., more competent fault zone features.

3.2.1 Shear strength of the matrix material

This section lists strength parameters of the weaker matrix material of bimrocks, but also of weak fault zone material that is not characterised as bimrock. A fault zone that mainly comprises such weak fault zone material still can feature single stiff blocks even if not encountered, e.g., with investigation drillings or during tunnelling.

⁴Diameter or any other characteristic dimension of the block (cf. [255, p. xvii]). Cf. also Footnote 1 (p. 20).

⁵A particle is classified as a *boulder* if it features a length of 256 ... 4096 mm (cf., e.g., [40, Fig. 2, p. 8]).

⁶A particle is classified as a *grain* if it features a length of 0.063 ... 2 mm (cf., e.g., [40, Fig. 2, p. 8]).

[106] compiled and re-analysed published laboratory data about shear strength properties of shear zone materials in crystalline rocks. Fig. 3.3 compares the cohesion and the friction angle (peak shear strength) of different shear zone material. Most of the values for the internal angle of friction range from 22° to 36° with $\varphi_{min} \approx 16^\circ$ and $\varphi_{max} \approx 42^\circ$. The cohesion ranges from 0 MPa to approx. 1.8 MPa. From the graph it seems that at least for the schist kakirite there is a negative relationship: a material sample with a high friction angle shows a lower cohesion than a material sample with a lower friction angle. The material samples then may be at a different stage of evolution regarding shearing. Consider that the cementation of (intact)⁷ rock material is gradually destroyed with shearing, reducing the cohesive strength (softening of cohesion) and transforming the material to a granular one eventually featuring frictional hardening only ([410, p. 37]). But, during shearing also composites of bonded particles break apart. Those fragments now involved in the shearing process feature an angular shape contributing to an increase in the internal angle of friction (for more details, refer to Section 3.2.10 on p. 40; there, it is also described in which case the friction angle decreases with increasing shearing). Anyhow, when trying to identify relationships one has to keep in mind that test results strongly depend on the sample preparation and test settings (e.g., loading sequence).

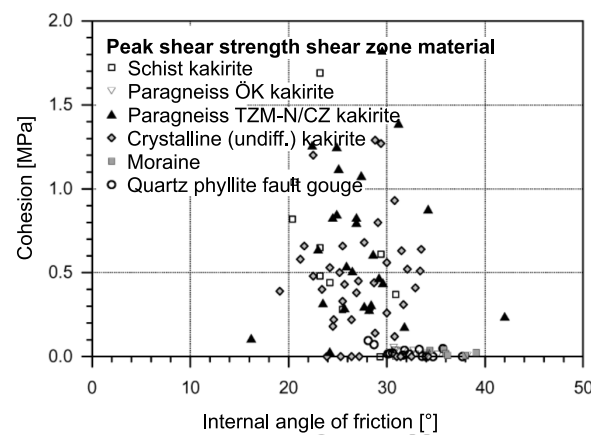


Figure 3.3: Peak shear strength of shear zone materials in crystalline rocks (modified from [106, Fig. 7, p. 70]; modification: translation). For results of linear regression of this data, cf. Fig. 3.4. ÖK ... *Ötztal Kristallin* (crystalline), TZM-N ... northern *Tavetscher Zwischenmassiv* (intermediate massif), CZ ... *Clavanièr* zone.

The tests on shear zone material from which [106] have reviewed the results (triaxial compression, direct shear, and ring shear tests; cf. text above), have been performed under different stress conditions. The Mohr-Coulomb parameters, i.e., the internal angle of friction, φ , and the cohesion, c , are fitting parameters and depend on the stress range within which the fit is performed. Thus, the authors decided to do the linear regression analysis on the normal stress, σ_n , and shear stress, τ , data pairs rather than on the Mohr-Coulomb parameters. From the

⁷In this context, *intact* relates to a pre-peak state of a particular rock (or rock mass) volume for a given stress state. If the material reaches its final residual (post-peak) state after intense shearing, and, for whatever reason, the stress state changes (e.g., confinement increases), it might regain a pre-peak state capable of resisting additional load. This is probably not an infinite process. At high temperature and pressure, depending on the characteristics of elements present (e.g., fluids, gases), chemical reactions may take place (e.g., metamorphose, recrystallisation, or cementation). In geotechnics, *intact* also relates to the state a material exhibits prior to additional straining because of natural (e.g., earthquake) or man-made events (e.g., tunnel excavation). As material inevitably gets disturbed at recovery (e.g., damage during cutting or drilling, unloading), transportation or preparation, the amount of which is often unknown or indefinable, the term *intact* generally relates to the material state prior testing (laboratory or in situ). Moreover, in rock mechanics, the term *intact* is often used to differentiate between (intact) rock and (imperfect) rock mass. The former does not feature a persistent plane of weakness. Few or many persistent or non-persistent discontinuities cross the latter, splitting it into a volume comprising several connected or unconnected (intact) rock pieces. It is apparent that this all is a question of scale.

fits, they then determined average values for the Mohr-Coulomb parameters. By introducing the requirement that the maximum value for φ can only relate to the minimum value for c , and vice versa (cf. *negative relationship* in text above), they could identify a region of confidence. The requirement somehow attempts to satisfy the fact that for most rocks the shape of failure envelopes (here, the trend of normal stress and shear stress data pairs) is between straight and parabolic (i.e., the envelope curves downward with decreasing normal stress) ([165, 180] in [130, p. 87]). Fig. 3.4 graphs some results of the fitting processes. Even though it compares the strengths of different shear zone material, it indicates the problem regarding the stress-dependency of the Mohr-Coulomb parameters: at low normal-stress levels, high friction angles and low values for the cohesion result; at high normal-stress levels, low friction angles and high values for the cohesion result. Depending on the geotechnical problem at hand and on the stress state and its temporal evolution, using the standard Mohr-Coulomb model (like in this thesis) with a fixed set of Mohr-Coulomb parameters, the rock strength may either get over- or underestimated. However, the characteristics of this relationship probably depend on the maturity of the shear zone material (e.g., level of decomposition). Fig. 3.15 (p. 42) compares peak and residual strengths from shear tests on joints in weak phyllites. It is not a strong sign, but the scatter of the residual strengths (represented by the error bars) is less than of the peak strengths. Thus, the influence of the difference in the stress level may be smaller.

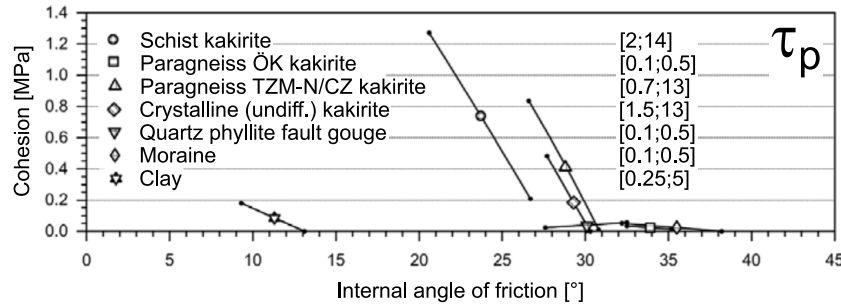


Figure 3.4: Peak shear strength, τ_p , of different crystalline shear zone material (from [106, Fig. 10, p. 75]; translated, and information on the range of normal stress, σ_n , from Tab. 1 in [106, p. 74] constituting the limits for the individual regression analysis has been added). Symbols constitute average values from linear regression of normal stress, σ_n , and shear stress, τ , data pairs from triaxial compression and shear tests. Inclination of error bars (90% confidence level) indicate the influence of uncertainties of the internal angle of friction or cohesion on the shear strength. ÖK ... *Ötztal Kristallin* (crystalline), TZM-N ... northern *Tavetscher Zwischenmassiv* (intermediate massif), CZ ... *Clavaniev zone*.

Fig. 3.5 shows the change in shear strength—from peak to residual strength—of some initially intact but weak rock samples from the *Semmering Base Tunnel* project subjected to direct shear tests (for details on the tests, refer to Section B.2.2 on p. 325 in the appendix). In most cases, φ and c are lower in the residual state (arrows point to the left and downwards). Section 3.2.10 (p. 40) details why strength can also increase with shearing. By normalising the change in shear strength to the peak shear strength, i.e.,

$$(\varphi - \varphi_r) / \varphi \quad \text{and} \quad (3.1)$$

$$(c - c_r) / c, \quad (3.2)$$

the graph in Fig. 3.6 results. In 26 of 35 cases, φ reduces by up to 20% and c by 50% to 100%.

Tab. 3.1 lists some statistical parameters of the normalised changes in strength graphed in Fig. 3.6.

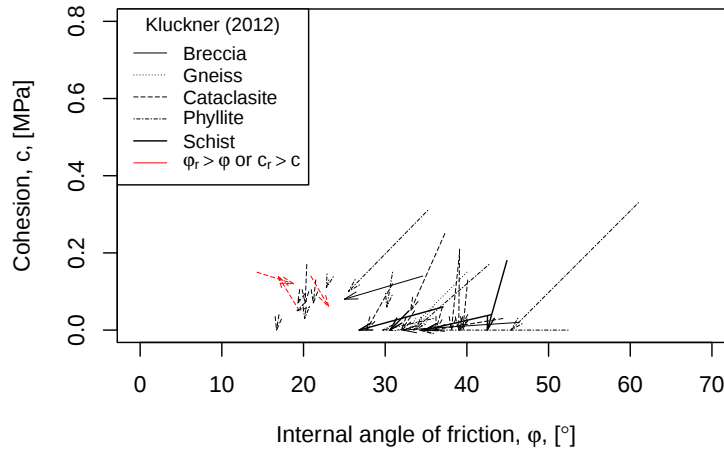


Figure 3.5: Shear strengths (in terms of internal angle of friction, φ , and cohesion, c , data pairs) from direct shear tests on intact rock specimens under constant normal stiffness (CNS) or constant normal load (CNL) conditions: Change from peak to residual strength. Orientation of weakness planes (e.g., foliation planes; if present) relative to shear direction not considered here. Rear end of arrows: peak shear strength (φ , c); arrowhead: residual shear strength (φ_r , c_r). Data from Kluckner (2012) [199]. CNS: $n = 23$, $\tau_{max} = [0.24; 2.83]$ (in mega-pascal), $s(\tau = \tau_{max}) = [0.21; 18.35]$ (in millimetres); for more details, cf. Section B.2.2 (p. 325) in the appendix. CNL: $n = 12$; multistage test; phyllites, schists, or phyllite or schist cataclasites. Data also plotted in Fig. 3.6.

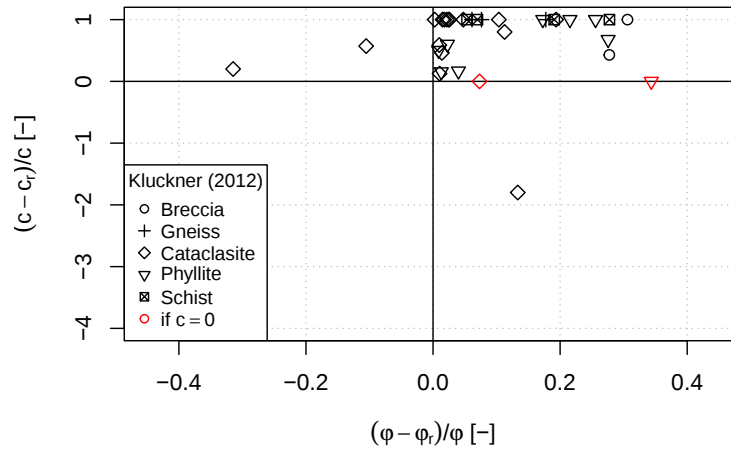


Figure 3.6: Shear strengths from direct shear tests on intact rock specimens: Normalised change in strength (from peak strength $\{\varphi; c\}$ to residual strength $\{\varphi_r; c_r\}$). Same data as for Fig. 3.5. Corrections (marked in the graph with a different colour): $c = 0 \wedge c_r = 0 \Rightarrow (c - c_r)/c = 0$; $c = 0 \wedge c_r \neq 0 \Rightarrow (c - c_r)/c = -4$.

Table 3.1: Shear strengths from direct shear tests on intact rock specimens: Statistics on normalised change in strength (cf. Fig. 3.6).

Ratio	Statistical parameter	Shear test on intact rocks (cf. Fig. 3.6) [199]
$(\varphi - \varphi_r) / \varphi$	Median	0.06
	Average	0.10
	SD	0.10
$(c - c_r) / c$	Median	1
	Average	0.82
	SD	0.29
SD...standard deviation		

In 2016, the Swiss Geological Survey published the final report on the geological, geotechnical, and hydrogeological conditions of the *Gotthard Base Tunnel* project ([145]). Tab. 3.2 lists some rock parameters of fault gouge from fault zones given in the report. Here, the friction angle ranges from 21° to 37°, and the cohesion from 0.3 MPa to 2.7 MPa.

Table 3.2: Rock parameters of fault gouge from the *Gotthard Base Tunnel* project ([145, Tab. 5.3–5.5, p. 56f]).

Rock type	φ^a [°]	c^a [MPa]	$E^{a,b}$ [GPa]
Prognosis for the construction project from literature, technical reports, and in situ and laboratory tests			
Kakirite (gneiss, schist, phyllite)	25–34	0.4–0.5	-
Triaxial compression tests on samples recovered from exploratory drillings			
Kakirite (CZ): strongly sheared, >30% ^c	30.1	0.5	1.4/3.9
Kakirite (TZM N): strongly sheared, >30% ^c	24.9	0.5	0.8/2.9
Kakirite (TZM N): sheared, crumbled, 10–30% ^c	24.6	0.4	0.9/2.9
Kakirite (TZM N): sheared, crushed, <10% ^c	27.4	0.5	1.2/3.6
Triaxial compression tests on samples recovered from core drillings performed at the tunnel level during construction			
Kakirite (CZ)	25.8–29.0	0.3–0.4	-/2.7–3.5
Kakirite (TZM N)	20.8–36.5	0.3–2.7	3.2–5.5
CZ... <i>Clavaniev</i> zone; TZM N... northern <i>Tavetscher Zwischenmassiv</i> (intermediate massif); both zones mainly comprise chloritic two-mica gneiss to schist and chlorite-sericite schist with biotite			
^a Single value (e.g., 30.1) ... average value; range of values (e.g., 0.4–0.5)			
^b Values depending on the lateral pressure, σ_3 , at the triaxial compression tests (e.g., 1.4/3.9): left value: $\sigma_3 = 1 \dots 2$ MPa; right value: $\sigma_3 = 9 \dots 12$ MPa			
^c Content of fines			

Tab. 3.3 summarises the representative design rock parameters (engineering judgement based on literature, technical reports, and in situ and laboratory test results) of five fault rock types from the *Semmering Base Tunnel* project. For the fault rock parameters, $\varphi = [23; 40]$ (in degrees) and $c = [0.06; 0.2]$ (in mega-pascal). And for the fault-zone mass parameters it is $\varphi = [23; 36]$ (in degrees) and $c = [0.06; 0.8]$ (in mega-pascal).

Table 3.3: Fault rock parameters and fault-zone mass parameters (in parentheses; homogenisation of fault rock and less disturbed zones) from the *Semmering Base Tunnel* project ([380, p. 30–34]). For a brief description of the fault rock types, refer to, e.g., Tab. 2 in [114, p. 192].

Rock type	φ [°]	c [MPa]	E [GPa]	ν [-]
Carbonate cataclasites, silty/sandy/gravelly, <30% ^a or >30% ^a	25–36 (27–33)	<0.2 (0.2–0.8)	0.3–1 (0.75–3)	NA (0.2–0.3)
Quartzite cataclasites, sandy/gravelly	25–40 (27–36)	<0.1 (0.3–0.7)	<3 (1–8)	NA (0.2–0.3)
Phyllosilicate-rich cataclasites (schist, phyllite), silty/sandy/gravelly, <30% ^a or >30% ^a	23–27 (23–27) ⊥ (25–30)	0.06–0.15 (0.06–0.15) ⊥ (0.4–0.8)	0.16–1.63 ^b (2–10) ^b ⊥ (0.4–2) ^b	≈ 0.3 (0.25–0.3)
Phyllosilicate-rich cataclasites (schist, phyllite), silty/sandy, >30% ^a	23–27 rm=r	0.06–0.15 rm=r	0.18–0.43 ^b rm=r	≈ 0.3 (0.25–0.35)
Phyllosilicate-rich cataclasites (sericite phyllite), silty, >50% ^a	23–25 rm=r	0.1–0.15 rm=r	0.16–0.37 ^b rm=r	≈ 0.3 (0.25–0.35)

without parentheses: fault rock parameters; with parentheses: fault-zone mass parameters; || ... parallel to cataclastic layers; ⊥ ... perpendicular to cataclastic layers; rm ... rock mass; r ... rock; NA ... not available

^a Content of fine grains (fraction <63 μm)

^b Oedometer modulus; values refer to virgin loading; values valid for $5 \leq \sigma_1 \leq 20$ (in mega-pascal)

[127] report about serpentinite bimrock material making up parts of the foundation of a gravity dam in Indonesia. They performed uniaxial and multistage triaxial compression tests on intact matrix specimens. The average cohesion results to $c = 3.7$ MPa, and the average friction angle to $\varphi = 36^\circ$ ([127, Tab. 2, p. 56]). [418] performed in situ direct shear tests on soil-rock mixtures at the Xiazanri slope in China. The mixture comprises limestone blocks (selected thresholds for the block size: 2 cm and 30 cm; maximum observed dimension) and a clayey matrix. From a test on a specimen with no blocks (i.e., matrix material only), they determined a cohesion of $c = 4.3$ kPa and a friction angle of $\varphi = 25.6^\circ$ ([418, Fig. 15, p. 1245]). [426] also analysed a soil-rock mixture of sedimentary origin. They performed triaxial compression tests on specimens different in block content (selected lower threshold for the block size: 5 mm; grain size). The material was recovered from the Tangjiashan Barrier Dam in China. From a test on a specimen comprising no blocks, they determined a cohesion of $c = 25.9$ kPa and a friction angle of $\varphi = 33.1^\circ$ ([426, Tab. 2, p. 50]).

The data listed above suggests a lower bound for the friction angle of approx. 20° . In contrast, [254, p. 269] report that a sheared matrix material in Franciscan mélangé (matrix often composed of sheared shale, argillite, siltstone, serpentinite, or sandstone) can feature an effective friction angle of $<10^\circ$ (and low to zero cohesion). However, such a low friction angle seems to be extraordinary and not representative for the material usually encountered at tunnelling projects. The friction angle of the fault gouge at the section of the *Galgenberg* tunnel where the collapse occurred ranged from 12° to 14° (cf. Section 1.1 on p. 1).

3.2.2 Uniaxial compressive strength of the matrix material

With the Mohr-Coulomb strength criterion (cf. Eq. 6.3 on p. 96), the equivalent uniaxial compressive strengths, σ_c , of two exemplarily selected specimens (both Paragneiss TZM-N/CZ kikirite) plotted in Fig. 3.3 (p. 23) are approx. 0.27 MPa (with $c \approx 0.1$ MPa and $\varphi \approx 16^\circ$) and approx. 6.2 MPa (with $c \approx 1.8$ MPa and $\varphi \approx 29.7^\circ$). Applying the same approach, it is approx. 14.5 MPa for the matrix material investigated by [127], approx. 0.01 MPa for the one from [418], and approx. 0.1 MPa for the one from [426] (cf. Section 3.2.1 on p. 22 for [127, 418, 426]). Anyway, σ_c of matrix material can be much higher:

- [378, Tab. 1, p. 555]: volcanoclastic Ankara agglomerate (matrix: tuff; blocks: andesite) with $\sigma_c = [6.4; 14.4]$ (in mega-pascal) for the matrix material;
- [185, Tab. 1, p. 1472]: strongly cemented Misis fault breccia (matrix: claystone; blocks: dolomitic limestone) with $\sigma_c = [40.2; 76.0]$ (in mega-pascal) for the matrix material.

Both the lower and upper bound values cited above are close to or within the range of strength values for fault zone material reported by [339] (cf. list on p. 39).

3.2.3 Shear strength of the block material

According to [255, p. 29], the friction angle contrast ($\tan \varphi_b / \tan \varphi_m$) between (the weakest) block and matrix material must be at least 1.5–2.0 to discriminate the components. Then, failure surfaces propagate around the blocks as observed at the laboratory tests of [227] rather than through the blocks (i.e., if contrast is too low and strength matrix $\sim$ strength block).

The matrix material of the mélange below the foundation of the Scott Dam (California), for example, features an average friction angle of 29° . It is 54° for the blocks within the mélange. The frictional strength contrast results to 2.48. [412] in [255, p. 29]

A similar ratio exists at chainage 282.3 m of the access tunnel *Göstritz* of the *Semmering Base Tunnel* project.⁸ Fig. 3.7 shows a sketch of the geological situation observed at the tunnel face. Here, weak cataclasites (zones A–C) surround a strong shear body of gypsum dolomite breccia (zone D). The frictional strength contrast ranges from 1.7 to 2.8 (based on the intact rock properties) and from 1.3 to 2.3 (based on the homogenised rock mass properties) (cf. Tab. 3.4).

[255, p. 28f] suggests that not only the friction angle can be used to identify a minimum contrast, but also the cohesion, or the uniaxial compressive strength (cf. Section 3.2.4). No threshold is given for the cohesion. For the bimrock given in Tab. 3.4, the contrast in cohesion ranges from 6 to 417.

3.2.4 Uniaxial compressive strength of the block material

Highlighted by [254, p. 268], in terms of the Mohr-Coulomb failure criterion, the rock strength is not only determined by the internal angle of friction but also by the cohesion. Thus, to discriminate between rock mixtures where larger blocks have negligible influence on the overall behaviour of the mixture, and bimrocks comprising significant blocks, a strength-contrast concept can be utilised rather than a friction angle-contrast concept (cf. Section 3.2.3). After a personal communication with Prof. Harun Sönmez in 2009, [254, p. 268] propose that the uniaxial compressive strength of the block should be greater than 1.5 times the uniaxial compressive strength of the matrix material (i.e., $\sigma_{c,b} > 1.5 \cdot \sigma_{c,m}$).

⁸The thesis uses the tunnel section this chainage is in as validation case (cf. Chapter 10 on p. 243).

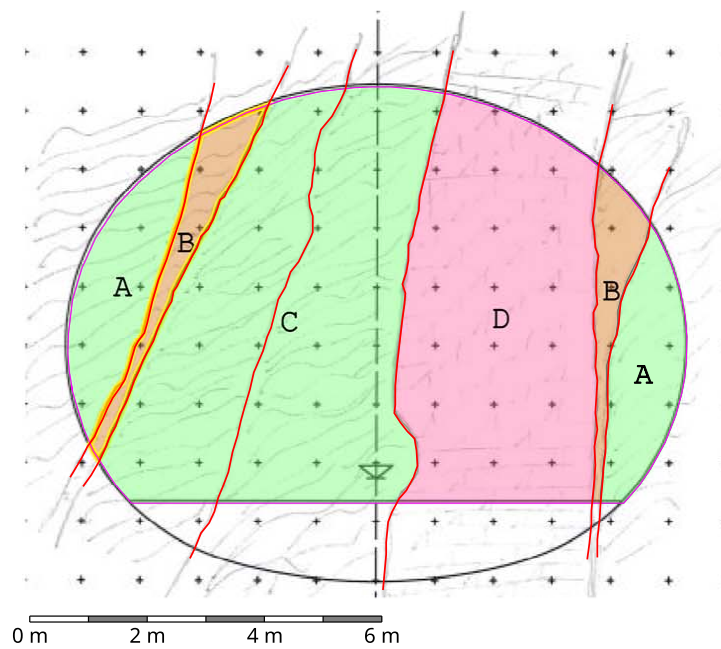


Figure 3.7: Sketch of the top/bench-heading tunnel face at chainage 282.3 m of the access tunnel *Göstritz* (construction lot SBT 1.1) of the *Semmering Base Tunnel* project (from [152]). Area A: cataclasite (sericite phyllite); area B: cataclasite (calcareous clay schist); area C: cataclasite (sericite phyllite); area D: gypsum dolomite breccia. Tab. 3.4 lists some strength parameters.

Table 3.4: Design values for the friction angle and cohesion of intact rocks and rock masses encountered at chainage 282.3 m of the access tunnel *Göstritz* (construction lot SBT 1.1) of the *Semmering Base Tunnel* project ([380, p. 25, 32, 34]). Note that the values given are design values and do not account for the actual composition of each zone observed at site (e.g., actual share of fault rocks and less disturbed zones within fault-zone masses). Fig. 3.7 graphs the sketch of the tunnel face.

Zone	bimrock component	φ [°]	c [MPa]	$\tan \varphi_b / \tan \varphi_m$ [-]		c_b / c_m [-]	
				min.	max.	min.	max.
A, B	matrix	23–27	0.06–0.15	1.7	2.8	113	417
		(23–27)	(0.06–0.15)	(1.4)	(2.3)	33	250
		⊥ (25–30)	(0.4–0.8)	(1.3)	(2.1)	6	38
C	matrix	23–25	0.10–0.15	1.8	2.8	113	250
		rm=r	rm=r	(1.6)	(2.3)	33	150
D	block	40–50	17–25	-	-	-	-
		(36–44)	(5–15)	-	-	-	-

without parentheses: intact rock parameters; with parentheses: rock mass parameters;
|| ... parallel to cataclastic layers; ⊥ ... perpendicular to cataclastic layers; rm ... rock mass;
r ... rock

[378, p. 555] reports a strength contrast between the block and the matrix material of Ankara Agglomerate of up to $\sigma_{c,b}/\sigma_{c,m} = 18.7$. Utilising Eq. 6.3 (p. 96) and the values for friction angle and cohesion of each component of the soil-rock mixture at the Longpan landslide (China; matrix: clay; blocks: sandstones, slate; [417, p. 753, 759]), the strength contrast results to approx. 11.1. The strength contrast between the cement-based matrix and block materials prepared by [306] for laboratory direct shear tests on artificial bimrocks is approx. 13.4 after a curing period of 28 days.

3.2.5 Tensile strength

Because tensile tests are far less often executed than, e.g., uniaxial compression tests, few tensile strength values exist in literature. If no data is available, engineers often assume the uniaxial peak tensile strength, σ_t , of intact rock to be one-tenth of the uniaxial peak compressive strength, σ_c . Literature quotes this approximation being valid for common rocks (cf., e.g., [181, p. 94], or [113] in [329, p. 147]).

Many authors have analysed the σ_c to σ_t ratio and tried to identify a general correlation. However, usually the correlation relates to the splitting tensile strength, $\sigma_{t,sp}$, since more often Brazilian tests are selected over direct tensile tests; most probably because of a lack in a direct tensile test apparatus.

[329, Tab. 1, p. 148] have listed some correlations published between 1955 and 2013, most of which are linear or have an exponential form (tensile strength from Brazilian tests). The strength ratios vary between 1.9 and 176.6 ([329, p. 147]). The ratios of some rock specimens [130, Tab. 3.1, p. 61] reports about vary between 10.0 and 167.6 (tensile strength from point load or Brazilian tests). And also ratios reported by [213] (cited in [183, p. 742]) range from 1 to well over 100.⁹

Fig. 3.8 plots pairs of uniaxial compressive and tensile strengths from [199] and correlations between the two strengths identified by [183] and [329]. Note that it is the splitting tensile strength, $\sigma_{t,sp}$, in case of [199, 329], and the uniaxial peak tensile strength, σ_t , in case of [183]. Section B.1 (p. 321) in the appendix lists some details on the rock types tested, on the fittings, and on criteria checking the suitability of tests to be considered.

Neither [183] nor [329] mention in any way the sizes and shapes of the specimens from the tests of which they used the results for their fittings. For example, for both the uniaxial compression test and the Brazilian test, the maximum load applicable to the test specimen (reflecting the specimen's strength) commonly decreases with increasing specimen height to diameter ratio (cf. Fig. 6.5a on p. 92 in the appendix; [392, p. 171]). In contrast, in literature, conclusions on the influence of the specimen size (i.e., volume) on the test results are inconsistent. Fig. 6.8 (p. 99) in the appendix suggests that the uniaxial compressive strength decreases with increasing specimen size. At other studies, the specimen size had little or no effect on the test results (e.g., Brazilian test: [392, p. 172]; uniaxial compression test: [392, p. 172], [162, 286] in [166, p. 155]). Anyway, many other factors affect the specimen's behaviour during testing and cause scatter in correlation plots: moisture content, loading rate, rock foliation resulting in anisotropy ([329, p. 149]), included defects or zones of weakness ([183, p. 742, 744]), etc.

In Fig. 3.8, all fitted correlations from [183] show an increase in the σ_t to σ_c ratio if σ_c decreases (inclination of lines in the graph is less than 45°). This applies also to the fittings

⁹[183, p. 732] already warned the reader that quality and reliability of test results published in literature varies. He, for example, examined the results published by [213] and doubts some of them because he either missed relevant information on applied techniques or involved failure modes, or he identified involved failure modes non-typical for particular test types ([183, p. 744]).

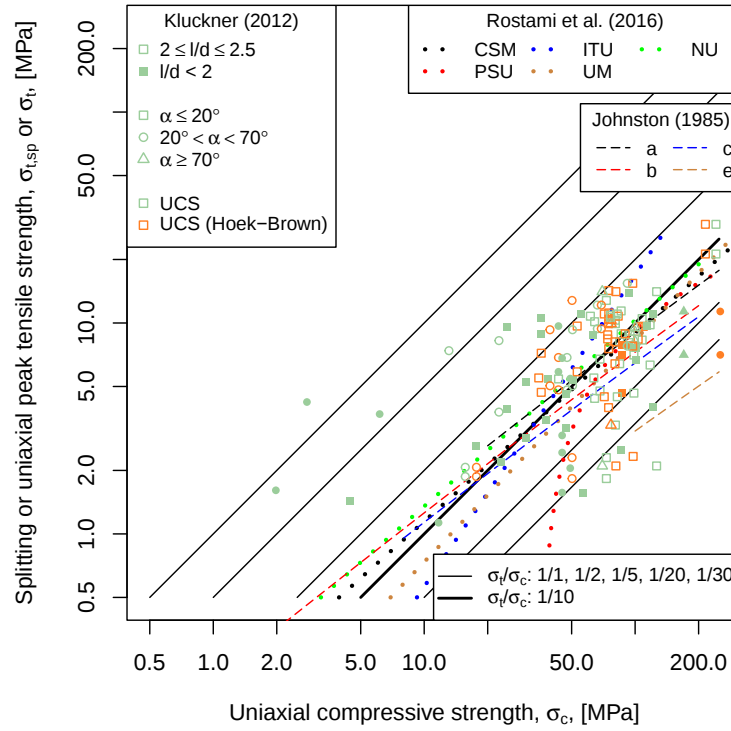


Figure 3.8: Relationship between splitting tensile strength, $\sigma_{t,sp}$, or uniaxial peak tensile strength, σ_t , and uniaxial peak compressive strength, σ_c , of rocks. Log-log plot. Johnston (1985) [183]: σ_t ; carbonate (a), argillaceous (b), arenaceous (c), and igneous and metamorphic (e) materials; for details, cf. Section B.1.1 (p. 321). Kluckner (2012) [199]: $\sigma_{t,sp}$; data from the *Semmering Base Tunnel* project; dolomite, gneiss, limestone, phyllite, and schist; l/d ... specimen length to specimen diameter ratio, α ... dip of weakness planes (if existing; if not: $\alpha = 0^\circ$), UCS ... σ_c from uniaxial compression test, UCS (Hoek-Brown) ... σ_c from triaxial compression test utilising the Hoek-Brown criterion; for details, cf. Section B.1.2 (p. 322). Rostami et al. (2016) [329] (rock types not differentiated): $\sigma_{t,sp}$; data from Colorado School of Mines (CSM), Pennsylvania State University (PSU), Istanbul Technical University (ITU), University of Melbourne (UM), and Nidge University (NU); for details, cf. Section B.1.3 (p. 324).

of the CSM and NU data by [329], and to the data by [199]. Skipping test results from [199] with $l/d < 2$, this increase cannot be observed anymore. A contrary picture results for the fittings of the ITU, PSU and UM data. Here, the ratio decreases with decreasing σ_c . Note that there might be some discrepancies in the fittings of [329] as the authors did not differentiate between rock types. Also, the dominance of anisotropic or isotropic rocks determines the best fits which the authors recognised themselves (cf. [329, p. 150]). They did not differentiate between isotropic and anisotropic specimens, nor between anisotropic specimens with different orientations of the weakness planes relative to the loading direction (for a brief introduction on the orientation's influence, refer to Section B.1.2 on p. 322 in the appendix). The adverse effect of varying anisotropy on the best fits by [183] is probably less because the fitting procedure considers several tests of different types (e.g., triaxial compression, Brazilian) on the same rock type (cf. Fig. B.1 on p. 322 in the appendix.).

[130, p. 65] reports that tensile strengths obtained with Brazilian tests are generally higher than those from direct tensile strengths. According to [394] (cited in [130, p. 65]), the splitting tensile strength exceeds the uniaxial peak tensile strength by a factor of >1 up to over 10 .¹⁰

¹⁰For comparison: A factor of 1 is to be assumed for concrete if no information on the correlation between the two strengths is available (cf. [121, p. 78, 199]).

Now, considering this, in Fig. 3.8, data pairs from [199] and correlation lines from [329] need to be shifted downwards a little (e.g., from being close to the 1/5 line to then being closer to the 1/10 line) to get the σ_t to σ_c ratio instead of the $\sigma_{t,sp}$ to σ_c ratio. Then, $\sigma_t/\sigma_c = 1/5$ seems to be a good approximation of the upper bound. And $\sigma_t/\sigma_c = 1/10$ is still a good approximation of the mean, especially for rocks with a low to moderate compressive strength. Considering the shift described above, for rocks with a high compressive strength, a ratio of 1/10 is then more of an upper bound. Even if the data from [183, 329] comprise rocks with a strength below 2 MPa, the major part of the evaluated rocks feature a higher strength. Thus, correlations in Fig. 3.8 are to be treated with caution for low-strength rocks.

3.2.6 Dilation angle (zone failure, disintegration)

If uncemented or cemented granular material (e.g., sand, rock, concrete) experiences deviatoric deformation (i.e., shearing), it usually changes its volume. This process is termed dilatancy.¹¹ Whether the volume remains more or less constant, decreases (contraction), or increases (dilation), depends on the material characteristics and state. Determining factors are, e.g., the properties of the material's constituents (e.g., shape, size), the constituents share and arrangement within the composition (e.g., state of packing, void space), the bonding between each of the constituents, and the experienced shear distortion so far. In general, if a specimen is loaded (compression, or shear), it first contracts by rearranging its constituents (if possible), closing cracks and other voids (if present), and deforms elastically. At a particular load level, the formation of new cracks and crack propagation initiates, and with that void spaces start to enlarge because some constituents slip over others (frictional sliding). From that moment on, both contraction and dilation take place simultaneously. For a dilatant material and if the confining stress is not too high (cf. text below), at any one time when deformation increases, the share of dilation exceeds the share of contraction and the test specimen features a larger volume than prior to testing ([130, p. 70]). Volume increase associated with plastic deformation is a pervasive phenomenon for granular material subjected to compression ([82] in [427, p. 369]) and can be verified theoretically based on plastic work considerations ([161] in [427, p. 368]). After a large deformation, the specimen or a particular zone within the rock mass reaches a state at which subsequent loading does not change the volume anymore. Further deformations are then localised in narrow shear bands. [410]

Note that the dilation described here refers to the increase in volume of a soil, rock, or rock mass zone with all its weaknesses because of zone failure upon loading. In rock mechanics, it differs from the increase in volume because of shear movement along a single rock fracture. At the latter, the mean aperture typically increases, since "the asperities of one fracture surface must by necessity ride up in order to move past those of the other surface" ([181, p. 375]). This is also termed dilation. In both cases, sliding along rock surfaces—either of rock fragments, or of rock asperities—causes the volume to increase. Anyway, the former refers to shear failure of a zone and zone disintegration, but the latter refers to shear failure of a rock fracture (cf. Section 3.2.10 on p. 40).

Classical continuum mechanics utilises the dilation angle, ψ , to record dilation ([427, p. 368]). For simple shear, it is the arcsine of the ratio of plastic volume change over plastic shear strain ([410, p. 7, 19f]). For triaxial compression and plane strain conditions, [410, Eq. 3.2, p. 13] propose a formula considering the plastic volume change and the uniaxial plastic strain.

The onset and amount of dilation strongly depends on the confining stress. At higher confining

¹¹Literature review disclosed the variety of how differently terms like dilatancy or dilation are used. Section B.2.1 (p. 325) in the appendix briefly list some uses and how it is understood for working out this thesis.

stress, dilation gets delayed, and dilation rate and peak dilation are lower ([427, p. 370]). Studying evaluations of triaxial test results in [427, Fig. 8, p. 375] shows that already low confining stress can restrain dilation significantly. In case of very high confining stress, dilation vanishes ([410, p. 14] with data from [395]).

According to [410, p. 16, 60], the dilation angle is always lower than the angle of internal friction, at least by 20° (i.e., $\varphi - \psi \geq 20^\circ$). Typical values for soils and rocks are ([410, p. 13f]):

- very dense sand: $\psi = 15^\circ$ (data from [160]);
- loose sand: $\psi =$ just a few degrees;
- normally consolidated clay: $\psi = 0^\circ$;
- granular (high porosity: 4.6%) to intact (low porosity: 0.45%) marble: $\psi = [12; 20]$ (in degrees), which reduces to $\psi = [6; 9]$ (in degrees) at very high confining pressure (200 MPa) (data from [262]).

In line, [95, p. 15] generally suggests $\psi < \varphi$ (non-associated plasticity), and $\psi = \varphi - 30^\circ$ for $\varphi \geq 30^\circ$ or $\psi = 0^\circ$ for $\varphi < 30^\circ$ in case no information is available from experiments. [95, p. 15] further suggests that after a certain plastic shear strain reaching a critical volume increase or a critical void ratio, $\psi = 0^\circ$. According to [244], these recommendations by [95] relate more to soils than to rocks.

Somewhat contrary, [293] (cited in [427, p. 371]) concluded based on numerical modelling that for most geological materials, particularly brittle ones, the dilation angle can be greater than the internal angle of friction. And [163] (cited in [427, p. 371]) suggest using a constant dilation angle with the value depending on the rock mass quality (very good: $\psi \approx \varphi/4$; average: $\psi \approx \varphi/8$; poor: $\psi \approx 0$; in degrees). Data from [199] (detailed in Section B.2.2 on p. 325 in the appendix) on weak rock material show a dilation angle of below 11° ; and $(\varphi - \psi)_{\min} = 23.8^\circ$, which complies with the statement by [410, p. 60] (cf. text above).

3.2.7 Poisson's ratio

At compression tests, the Poisson's ratio, ν , is the negative of the ratio of the absolute transverse (or lateral) strain to the absolute axial strain, $-|\varepsilon_l|/|\varepsilon_a|$, ([181, p. 84, 108], [130, p. 67]). Upon the theory of elasticity, $-1 < \nu < 0.5$, where usually only man-made materials feature negative values and a value of 0.5 refers to an incompressible material ([181, p. 110f], [241, p. 165]). $0 < \nu < 0.5$ applies to most real materials ([334, p. 92]).¹²

For a mere linear elastic material, ν is independent of the stress state ([181, p. 84]) and a constant of proportionality ([130, p. 67]). This is also true for rock material subjected to a load at which it deforms approximately elastically. At compression tests, that is

- at the virgin loading after pre-existing cracks have closed upon loading but prior to when pre-existing cracks grow and new ones initiate (i.e., from point A to point B in Fig. 3.9a), and
- at unloading-reloading loops (cf. Fig. 3.10a).

Performing unloading-reloading loops prior and beyond peak load at different material states, the development of the Poisson's ratio can be evaluated (cf. dashed line in Fig. 3.10c). It is not

¹²In this context, it must be noted that the Poisson effect is not to be confused with dilatancy both being a measure for volume change. The former only concerns the elastic (i.e., recoverable) part of deformations of a material upon loading or unloading. The latter refers to plastic deformations upon loading only (cf. Section 3.2.6).

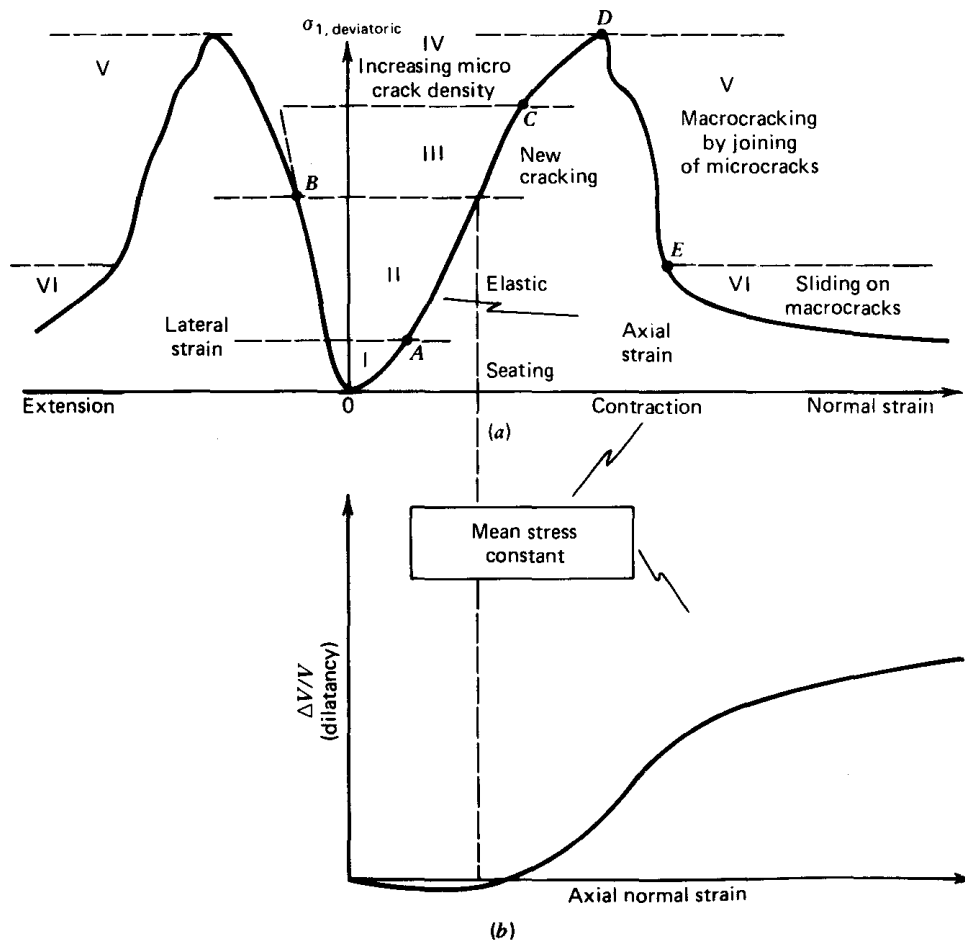


Figure 3.9: Triaxial compression test: Deformation under increasing deviatoric stress, with constant mean stress (hypothetical curves) (from [130, Fig. 3.7, p. 70]). (a) Axial and lateral normal strain with increasing deviatoric axial stress; (b) volumetric strain with increasing axial normal strain (dilatancy).

constant but increases with increasing deformation. According to [241, p. 178], the Poisson's ratio not only depends on the differential (i.e., deviatoric) stress, but also on the effective mean stress (i.e., hydrostatic or non-deviatoric). It generally increases with increasing confining or differential stress ([241, p. 165]). This stress-dependency is because of internal changes of pore space in a dry rock upon external load ([241, p. 171]). Thus, considering all that but also specimen characteristics not addressed here (e.g., anisotropy), a single value for the Poisson's ratio (e.g., given in a technical report or publication) should be understood as a mean value (cf. [241, p. 178]).

Typical values for rock range between 0.1 and 0.3 ([181, p. 149, 291], [241, p. 165]). Except for shale, all average values given in the labels of the y-axis in Fig. 3.11 fall into this range. Fig. B.3 (p. 327) in the appendix graphs data from [130, 199] aiming to find a trend between the Poisson's ratio, ν , and the Young's modulus, E , or the uniaxial peak compressive strength, σ_c . The data plotted covers a wide range of rock stiffness and strength. Most values for the Poisson's ratio are below 0.3. For a rock with a higher stiffness, usually a lower Poisson's ratio is determined ([241, p. 349, 179]). However, such a trend cannot be identified in Fig. B.3b/d. And considering the criteria detailed in Section B.3 (p. 326) limiting the comparison of data, no other trend is to be constructed either.

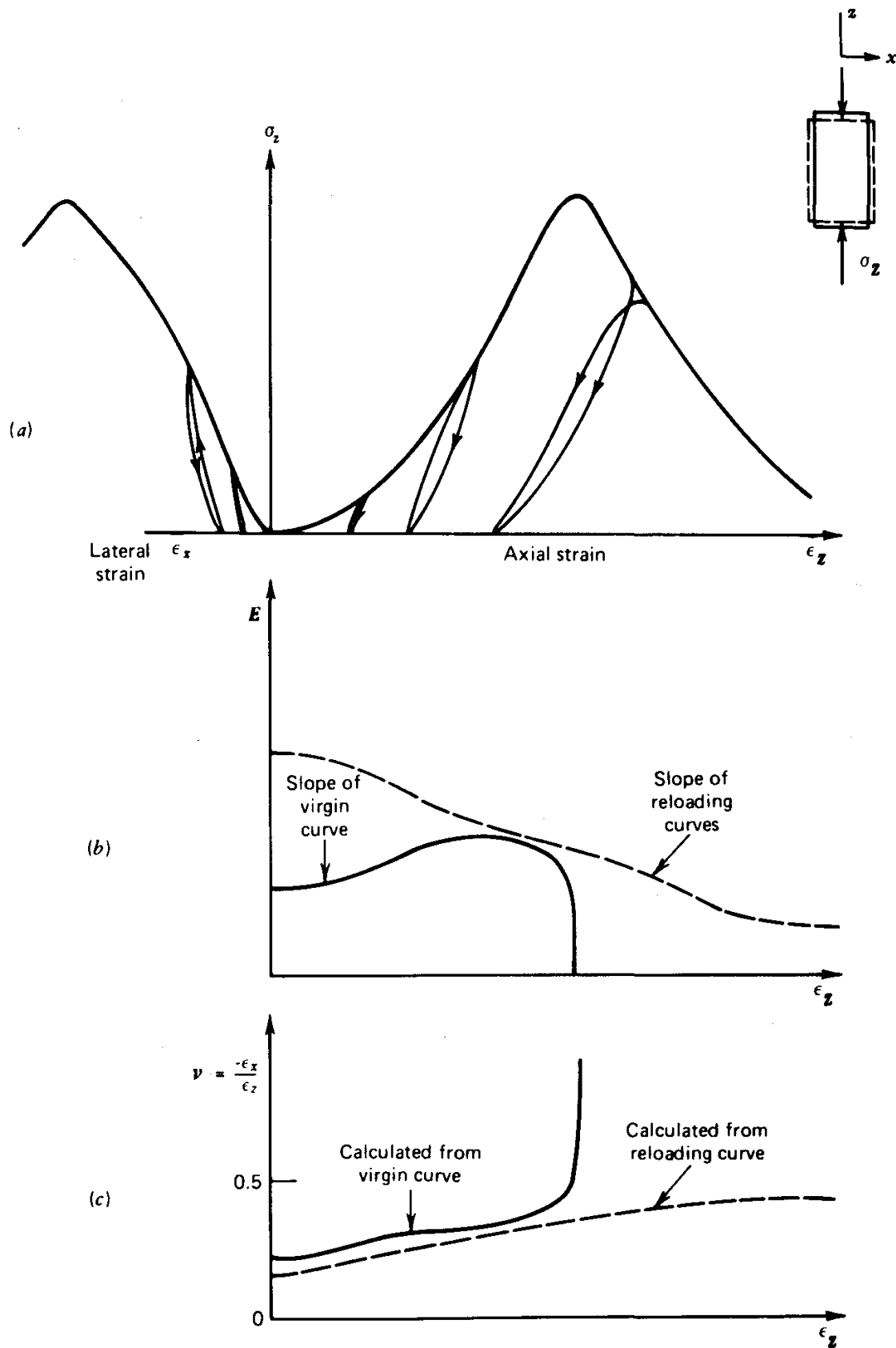


Figure 3.10: Uniaxial compression test: Variation of the modulus of elasticity, E , (tangent modulus) and the Poisson's ratio, ν , with axial strain (from [130, Fig. 6.3, p. 185]). (a) Development of axial strain, ϵ_z , and lateral strain, ϵ_x , with increasing axial stress, σ_z ; (b) variation of E with increasing ϵ_z ; (c) variation of ν with increasing ϵ_z .

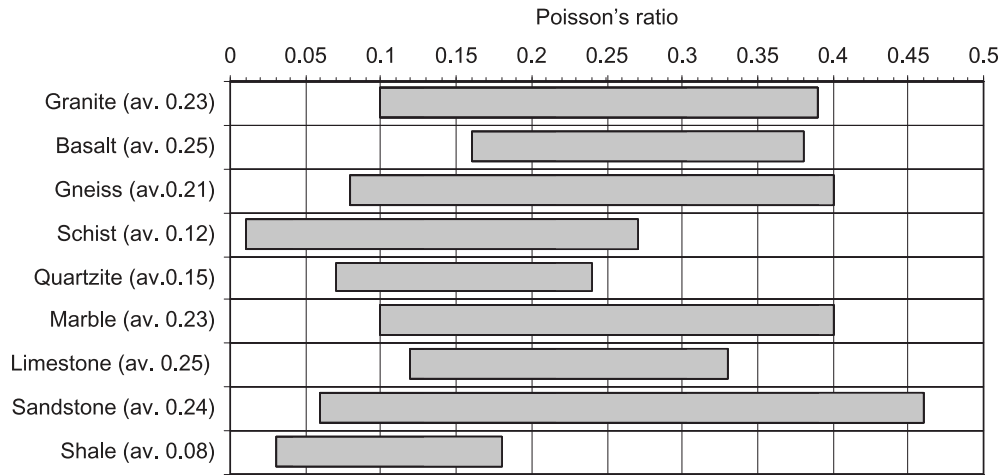


Figure 3.11: Poisson's ratio: Average (av.) and value range for common rock types (from [357, Fig. 7.8, p. 277] after [182]). For related values and value ranges for the Young's modulus, refer to Fig. 6.9 (p. 101); and for the uniaxial peak compressive strength, refer to Fig. 6.7 (p. 98).

3.2.8 Density

A material comprising more open cracks and other empty spaces and, thus, less solid constituents, necessarily features a lower density and stiffness than a similar material with less empty spaces; for at least as long as those spaces have not closed upon loading.¹³ Accordingly, from two specimens of the same rock type but different in the density, the denser one must feature a higher stiffness. [377] identified such a positive correlation for Ankara agglomerate (not shown).

Now, a stiffer rock material usually also features a higher strength (cf. Section 3.2.9). Thus, rock density must increase with increasing rock strength. [298], for example, identified this positive correlation for limestone to be of an exponential form (not shown). They have analysed results from tests on 1150 samples from 220 different limestones.

Considering the above statements, it is safe to assume that for a particular bimrock zone, the weak matrix material features a lower density than the stronger block material. The following examples of published data verify this assumption ($\rho \dots$ density):

- [185, Tab. 1, p. 1472]: strongly cemented Misis fault breccia (matrix: claystone; blocks: dolomitic limestone) with $\rho_m = 2.44 \text{ g/cm}^3$ and $\rho_b = 2.68 \text{ g/cm}^3$;
- [378, Tab. 1, p. 555]: volcanoclastic Ankara agglomerate (matrix: tuff; blocks: andesite) with $\rho_m = 1.72 \text{ g/cm}^3$ and $\rho_b = [2.31; 2.48]$ (in g/cm^3);
- [417, Tab. 2, p. 759]: soil-rock mixture (matrix: clay; blocks: sandstones, slate) with $\rho_m = 1.80 \text{ g/cm}^3$ and $\rho_b = 2.41 \text{ g/cm}^3$.

3.2.9 Young's modulus

Similar as for the strength, [254] demand for a minimum contrast between the stiffness of the block and matrix material so that blocks have a significant influence on the ground behaviour. They suggest a value of 2. The same value was utilised by [227] (cited in [254, p. 268]) for his experimental studies.

Because the parametric study of this thesis determines the material stiffnesses from relationships with the material strengths, such relationships are researched. In rock mechanics, the

¹³The differences depend also on the properties of the substances the spaces are filled with. That can be gases, fluids, or other solids being weaker or even stronger.

non-dimensional ratio of the Young's modulus, E , to the uniaxial peak compressive strength, σ_c , is known as modulus ratio, MR , (or modulus reduction; cf. [93, p. 138] and [164, p. 209]). For the determination of MR , the original formulation utilises the tangent modulus, $E_{t,50}$, at the virgin loading curve at 50% of the peak strength (cf. [93, p. 138]). [93, p. 136f] considered MR for his rock classification (cf. Fig. 3.12), which depends on σ_c (strength classes A to E) and on MR (high for $MR > 500$, average for $200 \leq MR \leq 500$, and low for $MR < 200$).

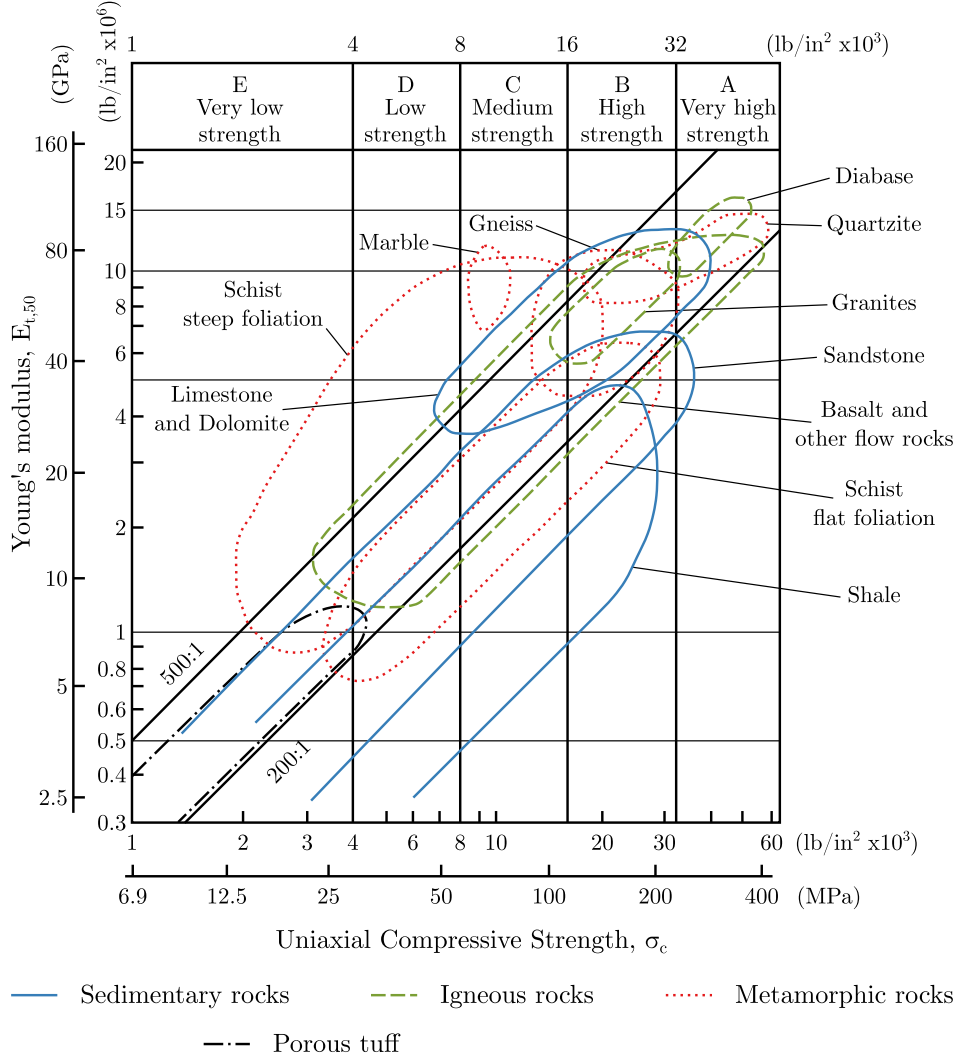


Figure 3.12: Relationships between the Young's modulus, $E_{t,50}$, (tangent modulus at 50% of peak strength) and the uniaxial peak compressive strength, σ_c , of intact rocks. Log-log plot. This graph combines Fig. 6.1, Fig. 6.2, and Fig. 6.3 from [331, p. 151–153]. All those figures originate from [94], but have been modified. The envelope for porous tuff in Fig. 6.12 in [94, p. 154] is added to the graph here. Each envelope covers 75% of all data from tests on specimens of the particular rock type (e.g., data from tests on shale specimens). Total data: 193 sedimentary rocks, 176 igneous rocks, 167 metamorphic rocks, and 44 porous tuff. Straight inclined solid lines represent a modulus ratio of $MR = \{200; 500\}$ (non-dimensional). lb ... pound ≈ 0.45 kg, in ... inch = 25.4 mm.

The values for MR covered by all the envelopes in Fig. 3.12 range from approx. 60 (shale) to approx. 1700 (schist with a steep foliation).¹⁴ Values given in [164, Tab. 3, p. 210] (not shown) base on [93, 297] and range from 150 for shales and marls to 1100 for schists. [130, p. 184] refers

¹⁴The extrema cited in [94] but being outside the envelopes are: approx. 35 for hard shale ($E_{t,50} \approx 6.2$ GPa, $\sigma_c \approx 179.3$ MPa; cf. Fig. 6.11 on p. 152 in the reference), and approx. 2500 for schist with a steep foliation ($E_{t,50} \approx 51.7$ GPa, $\sigma_c \approx 20.7$ MPa; cf. Fig. 6.17 on p. 159 in the reference).

also to [93] and states that for most rocks MR ranges "from 200 to 500 but extreme values range as widely as 100 to about 1200." Some other published ranges are: from 84 for schists to 1157 for sandstones ([398, Tab. 1, p. 4])¹⁵; and from 80 for flysch sandstone to 760 for peridotite ([245, Tab. 3, p. 1263]).

The scatter of published MR values is large. There is no clear trend showing that lower-strength rocks generally feature either a lower or a higher value than rocks with a relatively higher strength. Reasons for the scatter are manifold. Every process a rock volume undergoes, starting at the recovery, transportation, storage, and preparation and, eventually, ending at testing, changes its state. The stress relaxation of a rock volume when recovered from depth, for example, already damages it because of initiation and propagation of microcracks ([164, p. 209]). From the same altered rock volume, however, later a specimen for testing will be prepared (accompanied by further altering) and the test results will then be used to characterise the non-altered (i.e., undisturbed; as long as the man-made construction is outside the range of influence; cf. Footnote 7 on p. 23) in situ rock and rock mass. Here, different characterisations result depending on the way laboratory test results are evaluated (often not cited in publications). Fig. 3.13 shows possible ways to determine the rock mass deformability from in situ plate jacking tests. Similarly, different values for the rock deformability can be obtained from laboratory compression tests (for more details refer to, e.g., [238]).

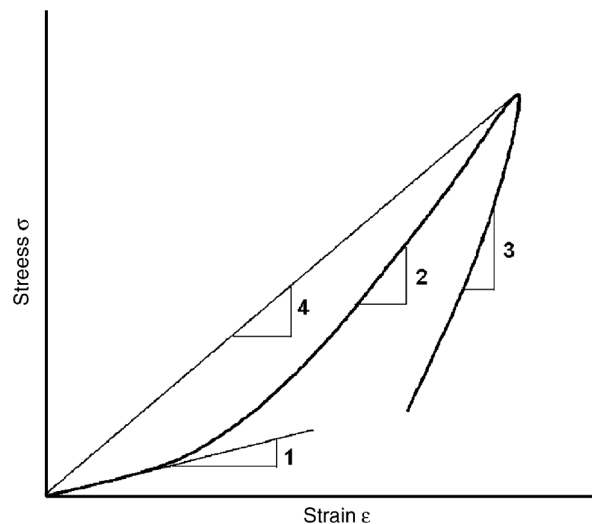


Figure 3.13: Alternative definitions for the deformability of a rock mass (from [164, Fig. 4, p. 205]). (1) Initial tangent modulus, (2) elastic tangent modulus (or modulus of elasticity), (3) recovery modulus, and (4) (secant) modulus of deformation. Note that the peak in the graph does not represent the moment of failure. It is just the point where the direction of loading gets reversed (from loading to unloading).

Thus, depending on how the rock deformability is determined, a different value for MR results for the particular rock specimen (or rock mass zone) tested. The identification of a *true* value is even more difficult since this in-the-course-of- and post-recovery damage of the specimen has a greater impact on its deformability than on its strength (cf. [164, p. 209]). In addition, the orientation of weakness planes (if present) relative to the loading direction also determines MR . It will be higher if the rock specimen gets loaded parallel to its weakness planes, and lower if loaded perpendicular to them ([164, Note a, Tab. 3, p. 210]).

The graph in Fig. 3.14 (p. 42) shows some published relationships between and data pairs

¹⁵Somehow inconsistent in [398, p. 3f]: the maximum value of 1157 given in Tab. 1 in the reference is much lower than the maximum value of approx. 3985 plotted in Fig. 1 in the reference.

of the Young's modulus, E , and the uniaxial peak compressive strength, σ_c . Note that in this section (especially regarding labels in graphs), *Young's modulus* is synonymous with all alternative pre-peak moduli of which some are illustrated in Fig. 3.13. Otherwise, the Young's modulus relates to the secant modulus from an unloading-reloading loop as it represents the true modulus of elasticity quite well ([272, p. 826], [130, p. 184]).

Fig. 3.14 plots the data from [199] two times: first, σ_c relates to E with E determined as a secant modulus at the unloading-reloading loop (unfilled circles); and second, it relates to the deformation modulus, V , determined as a secant modulus at the virgin loading curve (filled circles). Section B.4 (p. 326) in the appendix describes the same data from [199] in more detail. Using V , lower values for MR result. Differences are often quite large, e.g., $MR_E = 1336$ vs. $MR_V = 245$ for a rock with $\sigma_c = 6.25$ MPa, $E = 8.35$ GPa, and $V = 1.53$ GPa. This highlights the difficulty of comparing data from various sources and the identification of valid MR values, especially when no information is available on the method of test result interpretation. Considering all data from [199] plotted in Fig. 3.14, $MR_E = [191; 2580]$ and $MR_V = [137; 1404]$.

Most data pairs in Fig. 3.14, especially those of moderate- to high-strength rock, fall within the upper bound for sedimentary rocks ($MR = 2500$) and the lower bound for metamorphic rocks ($MR = 85$) (both relationships are from [398]). An exception is the data of low-strength rock from [274], where many of the pairs are below the lower bound. But this data may be faulty (cf. caption of Fig. 3.14). Anyway, the graph somehow agrees with the statement by [93] (cited in [130, p. 184]) that clastic rocks (here data from [274, 403] generally feature a lower MR than crystalline rocks (here most of the other data pairs; cf. also Fig. B.4 on p. 328 in the appendix). The linear relationships for sandstone, siltstone, conglomerate, limestone, and peridotite cited in [245] (not plotted; data of rocks with $2 \text{ MPa} < \sigma_c < 110 \text{ MPa}$) all fall within the range $MR = [130; 430]$ (non-dimensional). Considering only $\sigma_c < 50$ MPa in Fig. 3.14, the non-linear relationships by [298, 377, 398] indicate a higher MR for rocks with a lower strength (inclination of the lines in the graph $< 45^\circ$).

The following list comprises MR values of fault rocks¹⁶ and have been estimated from Fig. 4 in [339, p. 161]:

- Cohesionless kakirites, fault breccias (primary phyllitic structure):
 - Dolomite kakirite (sand- to gravel-size): $E = 0.1$ GPa, $\sigma_c \approx 0.12$ MPa, $MR = 833$;
- Cohesive kakirites, fault gouge (no internal structure visible anymore, soil-like):
 - Phyllite kakirite: $E = 0.24$ GPa, $\sigma_c \approx 1.3$ MPa, $MR = 185$;
 - Shale kakirite: $E = 0.1$ GPa, $\sigma_c \approx 0.7$ MPa, $MR = 143$;
- Intact rocks and cataclasites (cohesive):
 - Bedded limestone: $E = 47$ GPa, $\sigma_c \approx 71.5$ MPa, $MR = 657$;
 - Carbonate cataclasite (well-cemented fault breccia): $E = 48$ GPa, $\sigma_c \approx 87$ MPa, $MR = 552$; observed ranges are $E = [32; 66]$ (in giga-pascal) and $\sigma_c = [63; 139]$ (in mega-pascal) ([339, p. 165]).

For some stiffness values for fault rocks and fault-zone masses from the *Gothard Base Tunnel* project and the *Semmering Base Tunnel* project, refer to Tab. 3.2 (p. 26) and Tab. 3.3 (p. 27), respectively.

¹⁶It is assumed that the bedded limestones (intact rock) in the list make up the host rock or larger blocks embedded in the fault zone.

3.2.10 Block-matrix contacts

This study focuses on brittlely formed bimrocks with competent blocks, a weak matrix, and even weaker block-matrix contacts (cf. Section 2.2 on p. 14). Since the contact surface between a block and the surrounding matrix material constitutes a plane of weakness, this subsection lists some mechanical properties from joints in weak rock material. However, results and conclusions to some extent may also apply to welded or cemented bimrocks with competent blocks, a weak matrix, and block-matrix contacts which are at least as competent as the matrix material. For the latter, if numerical models do not consider interfaces between blocks and the matrix, because of deformation incompatibility and resulting stress concentrations along the block's surfaces (cf. Section 2.2), zones of the weak matrix will fail first around the blocks. Eventually, distinct shears will border the blocks. Some results and conclusions may apply to a bimsoil only if the matrix of this composite soil is cohesive. In case of a purely granular matrix possessing no cohesion (cf. [33, p. 14]), rearrangement of clasts will dominate the deformation behaviour rather than brittle failure mechanisms.

Fig. 3.15 plots the results of fittings done by [106] on data from shear tests on weakness planes in three different groups of weak phyllites (cf. Section 3.2.1 on p. 22 for some information on the fitting approach). In the figure, the filled symbols refer to the peak strength of the joints (labelled with *joint // cleav. peak*), and the unfilled symbols to the residual strength of the same joints but now comprising shear products (labelled with *kak. res.*). One can observe that the shear products of the quartz and limestone phyllites formed during the shearing along joints parallel to the cleavage feature a lower friction angle and a lower cohesion than was obtained for the unfilled joints. However, for the group of undifferentiated phyllites (labelled with *Phyl. joint*), the cohesion decreases but the friction angle increases.

Consider that if rock gets sheared (intact rock, joint walls, or joint fillings), bonds between individual rock fragments or minerals break. This reduces the cohesive strength. Whether frictional strength increases or decreases with additional shear depends on a few factors. If the constituents within the shear zone (i.e., those which are involved in the shearing process, e.g., broken off from the joint walls) mainly feature a round shape, turbulent rolling mechanisms will dominate and prevent platy or elongate constituents to align in a preferred direction (i.e., parallel to the shear direction) ([106, p. 66]). Because rolling friction (macroscopic friction of granulate) generally is higher than sliding friction (contact between particles) ([146, 237, 391] in [106, p. 78]), frictional strength increases when the mechanisms during shear change from pure sliding along the asperities on a joint surface or along a smooth joint surface to partly rolling of shear products ([106, p. 78], [62, p. 92]). A low joint normal stress promotes rolling of shear products (once formed) ([62, p. 81]) as it allows for some dilation and particles to roll over others. At high normal stresses, dilation and roll-over get restricted and rather a discrete sliding shear plane forms through the shear zone. In contrast, if the rock (or shear zone) comprises much platy or elongate constituents (e.g., phyllosilicates), those constituents align parallel to the shear direction and promote laminar sliding mechanisms ([106, p. 66]). Here, the frictional strength decreases with additional shear and alignment of constituents. But also the mineral composition affects the evolution of the shear strength. For example, shear zones in rocks comprising a significant amount of quartz or other competent minerals still may feature some cohesive strength and (an increased) frictional strength in the residual state ([106, p. 78]). To some extent, the competent minerals prevent sliding along phyllosilicates ([234, 375] in [106, p. 78]). They will not align themselves parallel to the shear direction as phyllosilicates do, and they serve as essentials for rolling friction as soon as they break off from the joint walls. If the rock lacks competent

minerals, in the residual state, the shear zone may feature no cohesion at all (i.e., purely frictional material) and a lower frictional strength than a shear zone material rich in competent minerals ([106, p. 78]).

[127, Tab. 1, p. 56] reports about the shear strength of surfaces of weakness in the matrix material of serpentinite bimrock. Here, the cohesion reduces from the initial to the residual state: $c = [0.22; 0.64] \rightarrow c_r = [0; 0.24]$ (in mega-pascal). In contrast, the range of the friction angle in the residual state is larger than in the initial state, indicating that for some joints the friction angle decreases but for others it increases: $\varphi = [21; 23.1] \rightarrow \varphi_r = [18.5; 24.2]$ (in degrees).

Fig. 3.16 graphs the alteration of the shear strength from its initial state (peak strength) to its residual state (residual strength) of joints in different rocks. The data is from direct shear tests performed under constant normal stiffness (CNS) conditions with the external stiffness $K \approx \infty$ (i.e., suppression of any vertical displacement). In the residual state, the rock joints necessarily comprise shear products (cf. text above). In almost all cases, the friction angle, φ , and the cohesion, c , are lower in the residual state than in the initial state (arrows point to the left and downwards). Fig. 3.17 plots the normalised strength (cf. Eq. 3.1 and Eq. 3.2 on p. 24). In 43 of 64 cases, the normalised change in φ ranges between 0 and 0.2. And it is 0.5 to 1 for the normalised change in c in 55 of 64 cases. The dilation angles of the joints of which the Mohr-Coulomb parameters are plotted in Fig. 3.16 range from 1.2° to 25° .

Tab. 3.5 lists some statistical parameters of the normalised changes in strength graphed in Fig. 3.17.

Table 3.5: Shear strengths from direct shear tests on rock discontinuities: Statistics on normalised change in strength (cf. Fig. 3.17).

Ratio	Statistical parameter	Shear test on discontinuities (cf. Fig. 3.17)	
		[199]	[62]
$(\varphi - \varphi_r) / \varphi$	Median	0.13	0.15
	Average	0.15	0.14
	SD	0.10	0.08
$(c - c_r) / c$	Median	1	0.97
	Average	0.95	0.90
	SD	0.12	0.19
SD...standard deviation			

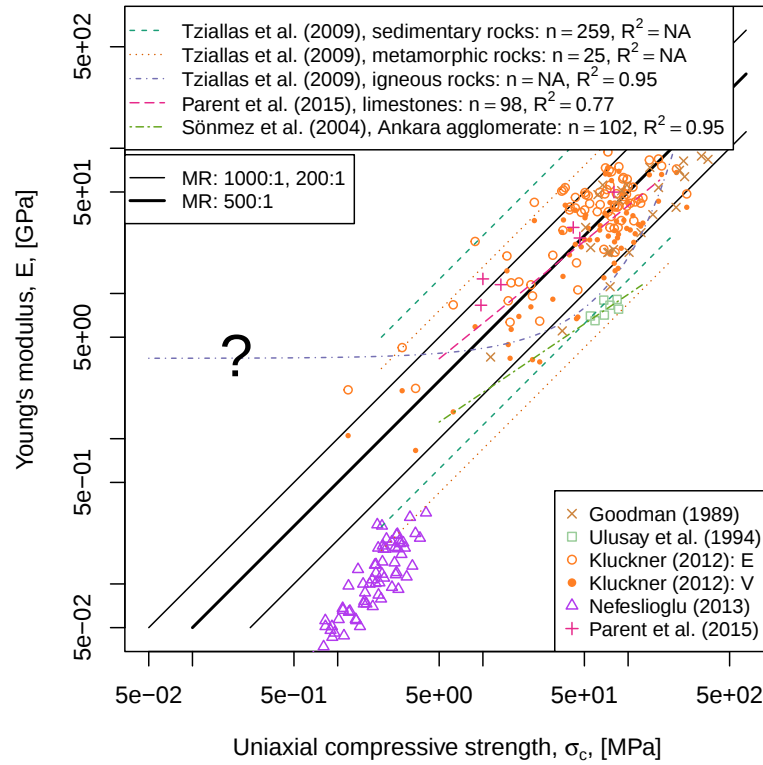


Figure 3.14: Comparison of the Young's modulus, E , with the uniaxial peak compressive strength, σ_c , of some rock specimens, and some relationships between the two parameters. Log-log plot. Data from: Tziallas et al. (2009) [398], tangent modulus; Parent et al. (2015) [298], secant modulus from unloading-reloading loop; Sönmez et al. (2004) [377], tangent modulus from virgin loading curve; Goodman (1989) [130], tangent modulus from virgin loading curve; Ulusay et al. (1994) [403], secant modulus; Kluckner (2012) [199], secant modulus from virgin loading curve (V) and from unloading-reloading loop (E); Nefeslioglu (2013) [274], type of modulus unknown. The stress level for the determination of the modulus is mostly unknown; for [199], cf. Fig. B.4; [298]: between 33% of the estimated failure load and 1 MPa. [130]: sandstone, siltstone, limestone, dolomite, shale, gneiss, schist, quartzite, marble, granite, tonalite, diabase, basalt, and tuff. [403]: medium-grained litharenite sandstone with low porosity, loading perpendicular to bedding planes (i.e., $\alpha = 90^\circ$). [199]: cf. Fig. B.4. [274]: claystone and mudstone; caution is recommended when interpreting data as Fig. 2 in [274, p. 12] suggests poor deformation measurement. [298]: limestones with a porosity ranging from 4.5% to 36.2%. Relationships of [398] for sedimentary and metamorphic rocks represent fitted boundaries of analysed data pairs. Range of σ_c for fitting by [398] for igneous rocks is unknown (marked with ? in the graph). NA ... not available.

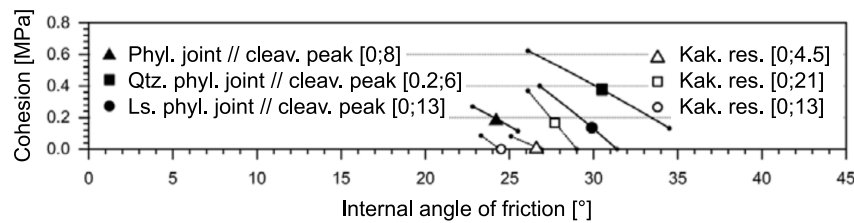


Figure 3.15: Peak shear strength (shear tests on unfilled joints parallel to cleavage) and residual shear strength (same joints but now filled with kakirites formed during shearing) of phyllites (from [106, Fig. 14, p. 75]; translated; abbreviations used: cleavage (cleav.), kakirite (kak.), limestone (ls.), phyllite (phyl.), quartz (qtz.), residual (res.)). Regarding symbols and error bars refer to Fig. 3.4. [106] used data from [62] some of which are also used in Fig. 3.16.

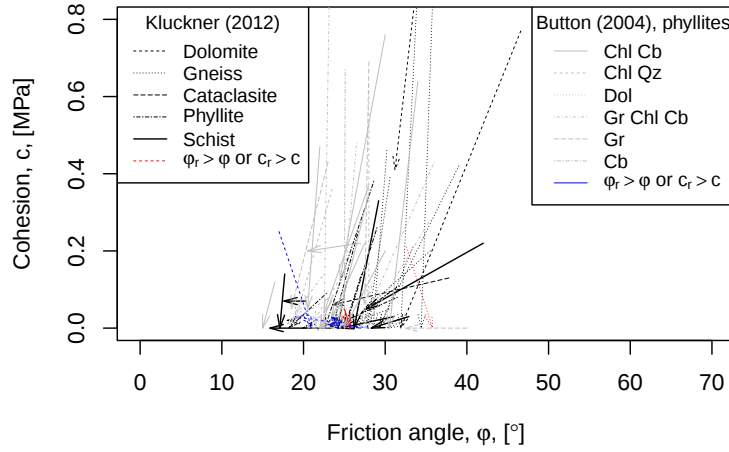


Figure 3.16: Shear strengths (in terms of friction angle, φ , and cohesion, c , data pairs) from direct shear tests on rock discontinuities (pre-existing joints or open cleavage planes) under constant normal stiffness conditions: Change from peak to residual strength. Rear end of arrows: peak shear strength with $\{\varphi; c\}$; arrowhead: residual shear strength with $\{\varphi_r; c_r\}$. Data from Kluckner (2012) [199]: $n = 33$, $\psi = [1.2; 25]$ (in degrees), $\tau_{max} = [0.38; 10.79]$ (in mega-pascal), $s(\tau = \tau_{max}) = [0.32; 16.2]$ (in millimetres). Data from Button (2004) [62, Tab. 7, p. 88]: phyllites comprising mainly following minerals (or mineral groups): chlorite (Chl), carbonate (Cb), quartzite (Qz), dolomite (Dol), graphite (Gr); $\tau_{max} = [0.4; 6.9]$ (in mega-pascal), $s(\tau = \tau_{max}) = [0.27; 19.16]$ (in millimetres), $\sigma_{n,max} = [0.94; 14.98]$ (in mega-pascal), $s(\sigma_n = \sigma_{n,max}) = [0.75; 22.01]$ (in millimetres). Data also plotted in Fig. 3.17 (p. 43).

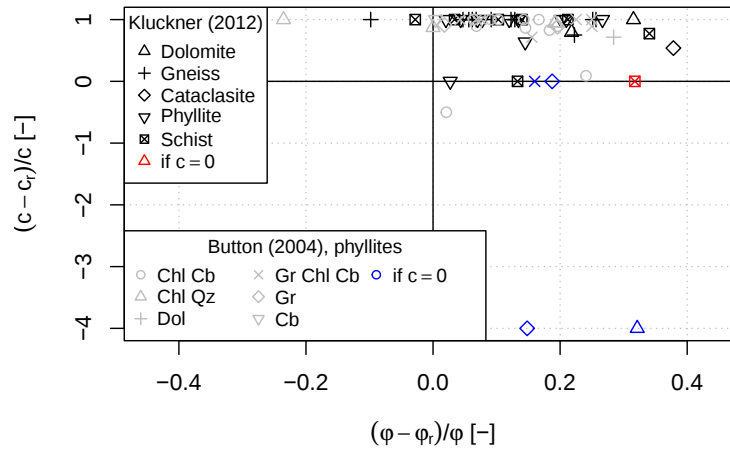


Figure 3.17: Shear strengths from direct shear tests on rock discontinuities: Normalised change in strength (from peak strength $\{\varphi; c\}$ to residual strength $\{\varphi_r; c_r\}$). Same data as for Fig. 3.16. Corrections (marked in the graph with different colours): $c = 0 \wedge c_r = 0 \Rightarrow (c - c_r)/c = 0$; $c = 0 \wedge c_r \neq 0 \Rightarrow (c - c_r)/c = -4$.

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Appendix B: Some mechanical properties of rocks

This appendix comprises information completing Chapter 3 (p. 19). For clarity, the section titles are identical.

B.1 Tensile strength

The following subsections list published test data referring to the tensile strength of rocks and to correlations of the tensile strength with the uniaxial peak compressive strength. Fig. 3.8 (p. 31) graphs the data.

B.1.1 Johnston (1985)

Johnston (1985) [183] compiled about 1700 results from various laboratory tests (triaxial compression, uniaxial compression, Brazilian, uniaxial tensile) on different groups of rock types (note the range of the uniaxial peak compressive strength, σ_c , different for each group):

- Group a: Carbonate materials with well-developed crystal cleavage (e.g., dolomite, limestone, and marble); $20 \text{ MPa} < \sigma_c < 600 \text{ MPa}$;
- Group b: Lithified argillaceous materials (e.g., mudstone, shale, slate, and clay); $6 \text{ kPa} < \sigma_c < 200 \text{ MPa}$;
- Group c: Arenaceous materials with strong crystals and poorly developed crystal cleavage (e.g., sandstone and quartzite); $8 \text{ MPa} < \sigma_c < 200 \text{ MPa}$;
- Group e: Coarse-grained polyminerallic igneous and metamorphic materials (e.g., amphibolite, gabbro, gneiss, granite, norite, and granodiorite); $100 \text{ MPa} < \sigma_c < 400 \text{ MPa}$.

And for each group Johnston performed a best curve fit (Fig. B.1; observed difference between measurement and prediction max. 16%). With the fittings of the failure envelope, it is also possible to predict the uniaxial peak tensile strength, σ_t , for a given uniaxial peak compressive strength, σ_c . These predictions are added to the graph in Fig. 3.8 (p. 31). Johnston disregarded data sets if they did not satisfy several (quality) criteria. He, for example, studied the failure modes involved at the individual uniaxial compression tests. If tensile splitting dominated failure rather than shear failure along a plane inclined to the specimen axis, then the test result remained unconsidered. The peak load (i.e., uniaxial compressive strength) is lower in the former case than in the latter and, thus, underestimating the shear strength of the particular rock ([183, p. 732]). Predictions of σ_t have been verified by the author comparing them with results from Brazilian

and direct uniaxial tensile tests. [183, p. 739] comments that the correlation for group e may be inadequate as one dominant data set used for the fitting featured unusual values.

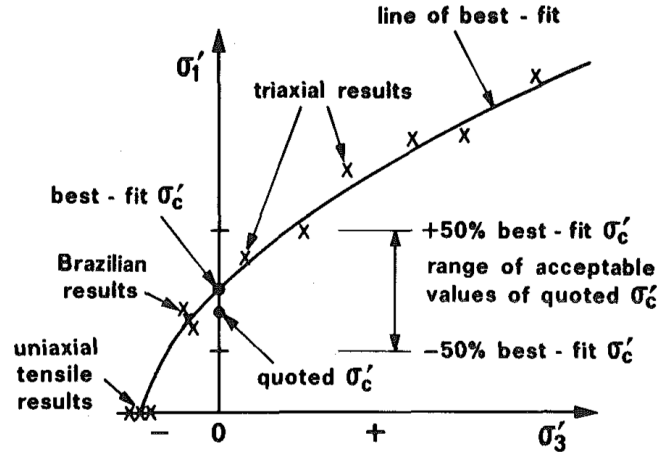


Figure B.1: Fitting of results from various laboratory tests in the σ'_3 - σ'_1 plane (from [183, Fig. 2, p. 733]). Fitting method: method of least squares. Fitting equation: $\sigma'_{1n} = (M \cdot \sigma'_{3n}/B + S)^B$ ([183, Eq. 1, p. 731] with $S = 1$ for intact materials; similar to the Hoek-Brown criterion, cf. [181, Eq. 4.26, p. 96]). $\sigma'_{1n} = \sigma'_1/\sigma'_c$, $\sigma'_{3n} = \sigma'_3/\sigma'_c$: normalised effective principal stresses at failure; intact material parameter B : describes the non-linearity of the failure envelope; intact material parameter M : describes the slope of the failure envelope at $\sigma'_{3n} = 0$. Symbols with apostrophe refer to effective stresses or to effective or drained strength parameters.

B.1.2 Kluckner (2012)

For the determination of characteristic ground types according to [290], Kluckner (2012) [199] analysed results from various laboratory tests (e.g., triaxial compression, uniaxial compression, Brazilian) on different rock types from the *Semmering Base Tunnel* project. Next to checking the photographs of the test specimens for failure along pre-existing planes of weakness, he applied following thresholds referring to the test specimen size and shape and published in standards to increase the comparability of the results ([199, p. 16ff]; $l \dots$ specimen length, $d \dots$ specimen diameter):

- Brazilian test: $d \geq 50$ mm ([223, p. 624]), $l/d = 0.5 \pm 0.2$ ([223, p. 624]);
- Triaxial compression test: $d \geq 47$ mm ([15, p. 4]), $2 \leq l/d \leq 2.5$ ([15, p. 4]); for the compilation in Fig. 3.8 on p. 31, the lower limit is relaxed to have results from tests on specimens with a l/d ratio below but close to 2 included; those results are marked accordingly);
- Uniaxial compression test: $d \geq 50$ mm ([111, p. 285]), $1 \leq l/d \leq 2.5$ ([272, Tab. 1, p. 828]); if $l/d < 2$, reduction of strength according to Eq. 16 in [272, p. 827] (from [285]).

The characteristics of the remaining test data are listed in Tab. B.1. For Fig. 3.8, the splitting tensile strengths, $\sigma_{t,sp}$, from Brazilian tests on specimens of a particular rock type recovered from a particular depth below surface (with core drillings) have been linked to uniaxial peak compressive strengths, σ_c , from uniaxial compression tests or triaxial compression tests (utilising the Hoek-Brown criterion, cf. [181, p. 96]) on specimens of the same rock type recovered from a similar depth. These pairs of σ_c and $\sigma_{t,sp}$ values are added to Fig. 3.8 (p. 31).

If rocks feature a pronounced foliation, their behaviour is anisotropic. Thus, when analysing results from tests on rock specimens with such a feature, the orientation of the weakness planes

Table B.1: Data from laboratory tests on rock specimens from the *Semmering Base Tunnel* project (from [199]).

Test type			Dolomite		Gneiss		Rock type Limestone		Phyllite	Schist
Brazilian	n	-	6	51	3	15	24			
	d	mm	{51.4; 102.0}	[51.2;102.2]	51.4	[51.2;102.0]	[51.2;102.5]			
	l/d	-	[0.48;0.53]	[0.42;0.58]	[0.55;0.60]	[0.41;0.58]	[0.47;0.54]			
	$\sigma_{t,sp}$	MPa	[2.19;10.49]	[1.57;14.25]	[4.65;7.83]	[1.87;29.36]	[1.13;15.42]			
Uniaxial compression	n	-	6	24	1	8	14			
	d	mm	{51.4; 102.0}	[51.2;102.1]	51.4	[51.2;102.0]	[51.2;102.5]			
	l/d	-	[1.36;2.00]	[1.06;2.00]	2.01	[1.22;2.00]	[1.07;2.01]			
	σ_c	MPa	[23.07;107.96]	[24.62;126.44]	98.68	[15.66;241.71]	[1.98;170.10]			
Triaxial compression	n	-	1	12	1	5	5			
	d	mm	51.3	[51.2;83.6]	51.4	[51.2;51.4]	[51.2;51.5]			
	l/d	-	2.01	[1.99;2.01]	1.99	[2.00;2.01]	[1.89;2.01]			
	σ_c	MPa	34.92	[43.31;109.40]	86.39	[17.69;215.39]	[35.89;253.34]			
n ... number of specimens, l ... specimen length, d ... specimen diameter, $\sigma_{t,sp}$... splitting tensile strength, σ_c ... uniaxial peak compressive strength										

relative to the load direction must be considered.² [75], who simulated uniaxial compression tests on anisotropic specimens numerically, observed different failure mechanisms involved depending on the dip, α , of a single weakness plane passing through the specimen centre ([75, p. 6407]):

- $\alpha = 0^\circ$: tensile and shear failure in the intact rock;
- $\alpha = 30^\circ$, $\alpha = 75^\circ$: both slip along the weakness plane and tensile failure in the intact rock;
- $\alpha = 45^\circ$, $\alpha = 60^\circ$: mainly slip along the weakness plane;
- $\alpha = 90^\circ$: splitting of the weakness plane and tensile failure in the intact rock.

The lowest uniaxial compressive strength resulted for the case where the weakness plane dips at 60° (cf. Fig. B.2). The highest values resulted for $\alpha = 0^\circ$ (i.e., axial load direction normal to the weakness plane) and $\alpha = 90^\circ$ (i.e., axial load direction parallel to the weakness plane). These findings match with other literature. [181, p. 104f] reports that depending on the orientation of planes of weakness, failure will either occur along a plane of weakness or along a plane within the intact rock. [130, p. 95] confirms that the strength is usually higher when the loading is parallel to the weakness planes than when the loading is perpendicular them (cf. Fig. B.2). He also reports a similar dependency of the behaviour on the orientation of weakness planes for triaxial compression tests (cf. [130, p. 94]).

Thus, [199] divided compressive strength data into three groups relating to the dip of the weakness planes: $\alpha \leq 20^\circ$ (weakness planes sub-perpendicular to the axial load direction), $20^\circ < \alpha < 70^\circ$ (weakness planes inclined to the axial load direction), $\alpha \geq 70^\circ$ (weakness planes sub-parallel to the axial load direction). These groups are marked accordingly in Fig. 3.8. Where weakness planes are inclined, slip along those planes might dominate the failure resulting to a low peak compressive strength of the rock specimen. This might explain why some data points of specimens with inclined weakness planes align far left of the majority in Fig. 3.8. Results from Brazilian tests also depend on the orientation of weakness planes relative to the major principal stress. Tests on specimens with weakness planes perpendicular to the load direction normally yield the highest tensile strengths; they are lowest for tests on specimens with weakness

²For all tests described here (Brazilian, uniaxial compression, triaxial compression), it is the direction of the load applied to the top and bottom of the test specimen.

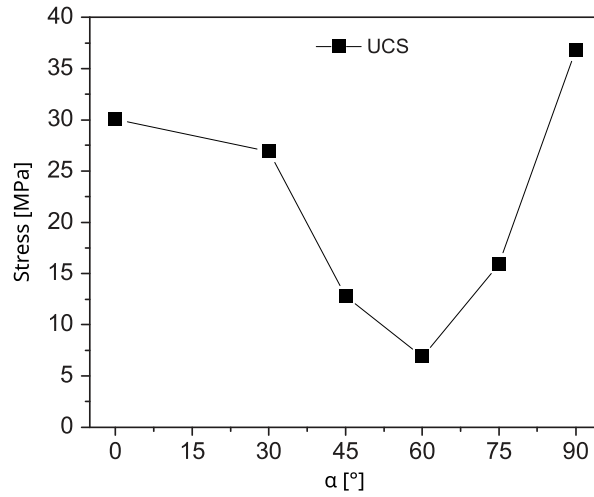


Figure B.2: Influence of the dip of a single weakness plane, α , passing through the specimen's centre on the peak strength (i.e., UCS ... uniaxial compressive strength) at numerical uniaxial compression tests (modified from [75, Fig. 5, p. 6408]).

planes parallel to the loading ([223, p. 625]). Here, no distinction is made for the plot in Fig. 3.8. Considering only strength pairs from tests featuring a similar loading direction relative to the dip of the weakness planes, very few data would remain.

B.1.3 Rostami et al. (2016)

Rostami et al. (2016) [329] have compiled more than 1800 pairs of uniaxial peak compressive strength, σ_c , and splitting tensile strength, $\sigma_{t,sp}$, values from laboratory tests on different rock types with a wide range of σ_c :

- Colorado School of Mines (CSM): $n = 182$; $\sigma_c = [1.3; 468.5]$ (in mega-pascal); mainly welded tuff, granite, sandstone, limestone, and argillite;
- Pennsylvania State University (PSU): $n = 198$; $\sigma_c = [8.1; 247.0]$ (in mega-pascal); mainly limestone, sandstone, claystone, siltstone, shale, and gneiss;
- Istanbul Technical University (ITU): $n = 158$; $\sigma_c = [3.6; 174.0]$ (in mega-pascal); mainly sandstone, limestone, shale, marl, and silt-claystone;
- University of Melbourne (UM): $n = 1202$; $\sigma_c = [0.2; 458.5]$ (in mega-pascal); no information on rock type available;
- Nidge University (NU): $n = 122$; $\sigma_c = [2.2; 210.6]$ (in mega-pascal); mainly limestone, travertine, granite, pyroclastics, marble, and metallic ores.

The authors performed a fitting with the method of least squares for each data set utilising a linear or exponential relation between σ_c and $\sigma_{t,sp}$. The correlation coefficient of the best fits range between 0.54 and 0.89 for linear relations and between 0.61 and 0.90 for exponential relations. Before fitting, the authors disregarded data from tests on specimens which featured a plane of weakness prior to the test having adverse effects on their behaviour during the test (e.g., failure along weakness plane) ([329, p. 148]). Fig. 3.8 illustrates the relations with the higher correlation coefficient (either linear or exponential) for each data set.

B.2 Dilation angle (zone failure, disintegration)

Section B.2.1 addresses the puzzling wording for volume change upon failure, and Section B.2.2 details results of shear tests on weak rock material.

B.2.1 Terminology

[410, p. 6] associate the term *dilatancy* (short for shear dilatancy) with volume changes which accompany plastic shear distortion. These volume changes can be negative or positive, i.e., resulting to a decrease or an increase in volume, respectively. A material which changes its volume upon shear distortion is called a dilatant material, independent of whether the volume decreases or increases (as the dilatancy angle used to characterise a dilatant material can be negative or positive; cf. [410, p. 7, 27]). If it preserves its volume, it is a non-dilatant material ([410, p. 8]). The process associated with a decrease in volume is termed contraction (the material contracts). In case of an increase in volume, it is termed dilation (or dilatation; the material dilates; cf. [410, p. 42]).

[427, Fig. 4, p. 371] use the terms contraction and dilation in the same way. [130, p. 70] explicitly relates the term *dilatancy* to volume increase associated with cracking (i.e., plastic deformation) only. However, somehow inconsistent, he uses this term also as a general term for volume change (cf. Fig. 3.9b on p. 34 in this document, or other Figures in [130], e.g., Fig. 3.9 on p. 73). Adding more confusion, in [11, p. 169], dilatancy is associated with volume increase only, and dilatation (or dilation) is used as a general term for volume change. And for [181, p. 85], dilatancy is the "phenomenon by which the volume of the rock decreases under the action of an additional compressive stress", but it also describes the phenomenon when "the total volume will in fact increase" (cf. [181, p. 266] referring to [82]). Further, it is a dilatant material if the dilatancy angle is positive, and it is contractant if it is negative ([181, p. 267]).

For this study, dilatancy is used as a general term describing volume change upon deviatoric loading. The dilatancy angle quantifies the volume change. Contraction refers to volume decrease (negative dilatancy), dilation (or dilatation) to volume increase (positive dilatancy). Thus, a material either contracts or dilates. Volume increase (i.e., dilation), however, occurs only at frictional sliding. And this is possible only in case of plastic (i.e., unrecoverable) deformation. This definition agrees, for example, with [166, p. 249] who quote that at "failure, the rock will dilate (increase in volume)". The dilation angle (not the dilatancy angle), therefore, quantifies volume increase only and equals the dilatancy angle if the latter is positive. A material is dilatant, if it changes its volume upon loading, and it is non-dilatant, if it preserves its volume.

B.2.2 Kluckner (2012)

In the course of his investigations, [199] examined also some laboratory direct shear tests on weak intact rock material (from which specimen preparation for triaxial compression tests was not possible). The data set ($n = 23$) comprises results from tests on phyllites, schists, carbonates, and gneiss, most of them in the form of breccias, cataclasites, or tectonites. The material has been recovered from depths of 20 m to 609 m below surface. The tests have been performed under constant normal stiffness (CNS) conditions with the external stiffness $K \approx \infty$ ([41]) (i.e., vertical displacement of shear box fully constrained, thus, no dilation possible; for details, cf., e.g., [310, 312]). The initial normal load, $\sigma_{n,0}$, ranges from approx. 0.3 MPa to approx. 0.85 MPa depending on the material tested. The shear area of all specimens was greater than 80 cm² exceeding the lower limit of 19 cm² according to [14, p. 4] by far. The values for the dilation

angle, ψ , range from 0.1° to 10.7° , those for the internal angle of friction, φ , from 30.9° to 61° , and those for the cohesion, c , from 0 MPa to 0.33 MPa. The data is included in the graph in Fig. 3.5 (p. 25). The difference between φ and ψ is 23.8° at minimum. The residual values are $\varphi_r = [25; 45.4]$ (in degrees) and $c_r = [0; 0.08]$ (in mega-pascal). Note that the analysis of the test results lacks a differentiation regarding the shear direction relative to the orientation of weakness planes (if present; e.g., foliation planes).

B.3 Poisson's ratio

Fig. B.3 compares the Poisson's ratio, ν , with the Young's modulus, E , and the uniaxial peak compressive strength, σ_c , of some rock specimens. The data is from [130, 199].

In the top graphs, the data from [199] is coloured according the upper value $\sigma_{1,u}$ of the axial stress range $\{\sigma_{1,u}; \sigma_{1,l}\}$ within which the unloading-reloading loop has been performed prior peak load to determine E and ν at the compression tests. If the developments of E and ν with axial strain shown in Fig. 3.10b/c (p. 35) generally apply to rocks, then each data in Fig. B.3 is valid for one particular material state (e.g., accumulated axial strain) only. Then, for example, data points in the top right graph from unloading-reloading loops where $\sigma_{1,u} > 15$ MPa probably would be closer to the bottom right corner of the graph in case the loops would have been performed at a lower stress level (e.g., $\sigma_{1,u} = 10$ MPa). Considering only data with the same $\sigma_{1,u}$, no clear trend can be identified, or too few data points exist. Being unaware of the stress levels the parameters cited in [130] have been determined at, no correlations in the bottom graphs can be constructed either. Note that other factors like differences in the testing procedure also contribute to the data scatter. Anyway, all graphs in Fig. B.3 show that independent of E and σ_c , ν varies within the natural range for real materials of $0 < \nu < 0.5$ (cf. Section 3.2.7 on p. 33). Most values are below 0.3.

B.4 Young's modulus

Both Fig. B.4 and Fig. B.5 plot data of sedimentary and metamorphic rocks from [199]. The former compares the Young's modulus, E , with the uniaxial peak compressive strength, σ_c . In the latter, E is compared with the deformation modulus, V . E is the secant modulus determined at an unloading-reloading loop, and V is the secant modulus determined at the virgin loading curve (cf. Fig. 3.13 on p. 38). For the determination of both E and V , the same axial stress levels are used between which the secants are constructed. The legend in the figures cites the upper bound $\sigma_{1,u}$ of the stress range $\{\sigma_{1,l}; \sigma_{1,u}\}$.

Because the data is limited, no trend can be constructed in Fig. B.4 regarding l/d , α , or $\sigma_{1,u}$. But most data pairs are above the 200 : 1 line and below the 1000 : 1 line. Fig. B.5 suggests that for less-stiff rocks, the increase from the modulus at the virgin loading curve, E , to the modulus at the unloading-reloading curve, V , is larger. This implies that at high-stiff rocks, the difference between E and V is less.

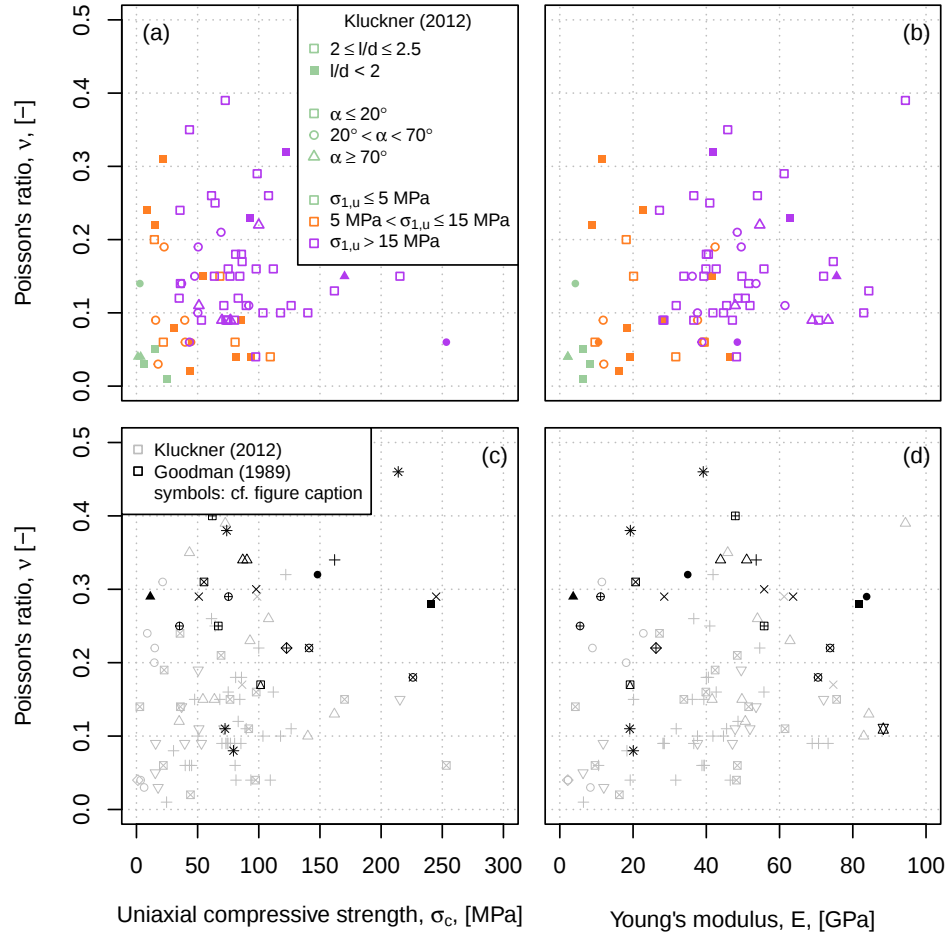


Figure B.3: Comparison of the Poisson's ratio, ν , with the uniaxial peak compressive strength, σ_c , (left column) and Young's modulus, E , (right column) of some rock specimens. Kluckner (2012) [199]: data from the *Semmering Base Tunnel* project (cf. also Section B.1.2); secant modulus from the unloading-reloading loop; l/d ... specimen length to specimen diameter ratio, α ... dip of weakness planes (if existing; if not: $\alpha = 0^\circ$), $\sigma_{1,u}$... upper bound of axial stress range within which the unloading-reloading loop has been performed to determine E and ν . Goodman (1989) [130, Tab. 3.1, p. 61; Tab. 6.1, p. 186]: for the origin of the data, cf. reference list in [130, Footnote c, p. 62]; tangent modulus from the virgin loading curve (stress level for determination is not reported). Symbols in bottom graphs: $\circ$ breccia ($n = 6$), Δ dolomite ($n = 11$), $+$ gneiss ($n = 34$), $\times$ limestone ($n = 5$), $\diamond$ cataclasite ($n = 1$), ∇ phyllite ($n = 10$), $\boxtimes$ schist ($n = 14$), $*$ sandstone ($n = 4$), $\diamond$ siltstone ($n = 1$), $\oplus$ shale ($n = 2$), $\boxtimes$ quartzite ($n = 1$), $\boxplus$ marble ($n = 2$), $\boxtimes$ granite ($n = 2$), $\boxtimes$ tonalite ($n = 1$), $\blacksquare$ diabase ($n = 1$), $\bullet$ basalt ($n = 2$), $\blacktriangle$ tuff ($n = 1$).

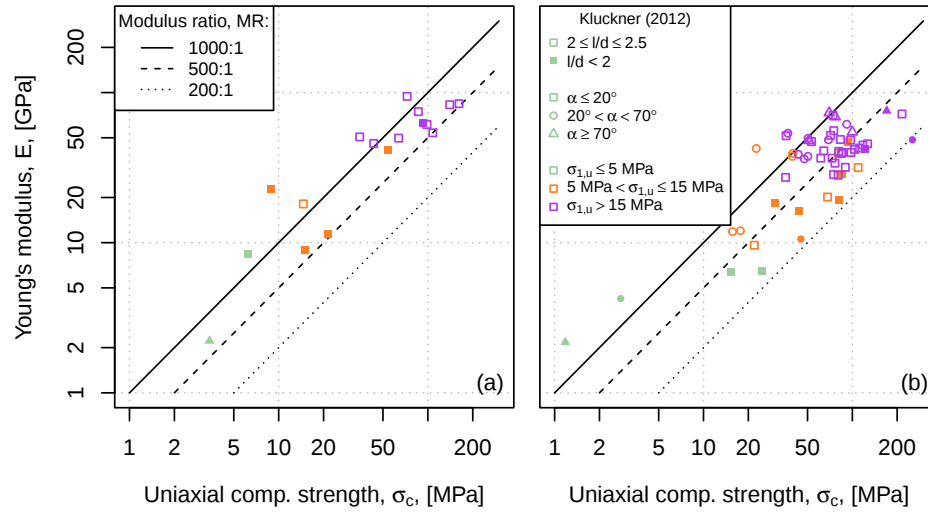


Figure B.4: Comparison of the Young's modulus, E , with the uniaxial peak compressive strength, σ_c , of some rocks. Log-log plot. (a) Sedimentary rocks: breccia, dolomite, limestone; (b) metamorphic rocks: gneiss, cataclasite, phyllite, schist. Kluckner (2012) [199]: data from the *Semmering Base Tunnel* project (cf. also Section B.1.2); secant modulus from the unloading-reloading loop; l/d ... specimen length to specimen diameter ratio, α ... dip of weakness planes (if existing; if not: $\alpha = 0^\circ$), $\sigma_{1,u}$... upper bound of axial stress range within which the unloading-reloading loop has been performed to determine E .

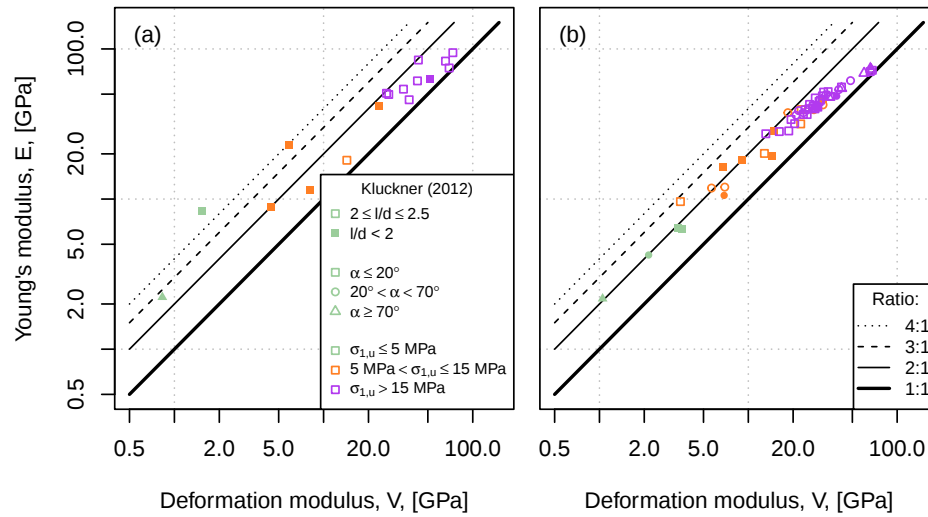


Figure B.5: Comparison of the Young's modulus, E , with the deformation modulus, V , of some rocks. Log-log plot. (a) Sedimentary rocks: breccia, dolomite, limestone; (b) metamorphic rocks: gneiss, cataclasite, phyllite, schist. Kluckner (2012) [199]: data from the *Semmering Base Tunnel* project (cf. also Section B.1.2); E ... secant modulus from the unloading-reloading loop; V ... secant modulus from the virgin loading curve; l/d ... specimen length to specimen diameter ratio, α ... dip of weakness planes (if existing; if not: $\alpha = 0^\circ$), $\sigma_{1,u}$... upper bound of axial stress range within which the secant moduli are determined.