

Chapter 3

Physical and Mechanical Properties of Rocks

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Abstract Since early antiquity dimension stones have been used as building materials due to their natural beauty and availability, and the diversity of their applications has increased ever since. As any other building material, dimension stones today have to fulfill the physical and technical requirements demanded by architects. This chapter focuses on the physical and mechanical properties of dimension stones, while emphasizing that stones are an old, but still modern building material. Among the parameters discussed here are water absorption, thermal conductivity and expansion, hygric and hydric properties, strength, abrasion, the more modern aspect of breaking load at the dowel hole, and ultrasonic wave velocities. Extensive data sets and a variety of case studies reveal relationships between the physical properties and the internal fabric elements of the dimension stones, such as sedimentary layering, metamorphic foliation, pores, and microcracks. In addition, these fabric elements are often responsible for the weathering behavior of the dimension stones, which not only affects the heritage but also the safety of modern buildings. This is illustrated through laboratory experiments and case studies.

3.1 Density, Mean Atomic Weight and Cation Packing Index

The density (ρ) of a material can be separated into the matrix density (ρ_{Matrix}) and the bulk density (ρ_{Bulk}). Whereas the value of the bulk density is stated with respect to the porosity of a material, the matrix density only depends on the components and does not take the porosity into account. Therefore, the matrix density provides evidence about the composition of a material. For example, calcitic and dolomitic marbles can be differentiated by the matrix density as the main constituents, calcite and dolomite, have significant different mineral densities, 2.710 g/cm³ and 2.866 g/cm³.

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There are different methods to determine the bulk and matrix density for rock samples. A simple and an often promising method is the measurement of the buoyancy acting on a sample immersed in water, the Archimedes method (e.g. Monicard 1980). Cubic samples with an edge length of 60–100 mm are preferable. The parameter to determine the density here are (i) the weight of the sample under dry conditions (m_{dry}), (ii) the weight under fully saturated conditions (m_{sat}), and (iii) the sample weight with the sample fully submerged in water (m_{sub}). For the last two values demineralised water is used for the saturation of the pore space. Fully saturated conditions are achieved through the excavation of the air in the pores before saturation. The density of the water needs to be known or should be determined. The matrix density can then be determined with the following equation:

$$\rho_{\text{Matrix}} = \rho_{\text{Water}} \cdot (m_{\text{dry}}/m_{\text{dry}} - m_{\text{sub}}) \quad (3.1)$$

The bulk density can then be determined as follows:

$$\rho_{\text{Bulk}} = \rho_{\text{Water}} \cdot (m_{\text{dry}}/m_{\text{sat}} - m_{\text{sub}}) \quad (3.2)$$

This method does not consider the closed pore space that cannot be accessed during the saturation process. As a result the weight values for the fully saturated and fully submerged states tend to be lower, and therefore the values of the matrix density.

The bulk density values for representative plutonic rocks range between 2.55 g/cm³ and 3.26 g/cm³ with only a few exceptions (Table 3.1, Fig. 3.1). A further analysis of the box plot reveals that a separation into two groups is possible. Generally, if the central segments of a box plot are not overlapping, the related distributions are significantly different at a 5% level (Popp 1994). Therefore, all subgroups, except the gabbro/diorite group, can be statistically combined. However, it is clear that the bulk density of the highly compacted rocks mainly depends on the mineralogy.

This can be seen in the comparison between the rock types Kösseine Granite (see Table 3.1) and the Ban Tak Granite (Strohmeyer 2003, Hoffmann and Siegesmund 2007, see Table 3.1). The relatively high bulk density of the Thai variety is the result of the mineral composition; here a higher amphibole content of ca. 24 vol.-%. The Kösseine Granite, on the other hand, is composed mainly of quartz and feldspar, and additionally 10 vol.-% of biotite. Data from Steindlberger (2004) have shown that volcanic tuff can exhibit a significant range for the matrix density values, from 1.90 g/cm³ to 2.80 g/cm³.

In contrast to the plutonic rocks, where the bulk density and the matrix density values are often similar due to the very low porosity that can be neglected (see Sect. 3.2), a differentiation between both density values is necessary for the sandstones. The bulk density is significantly affected by the pore space. For a mean porosity of about 15% and a matrix density of 2.67 g/cm³ the bulk density gives a mean value of about 2.27 g/cm³. The value for the matrix density (2.67 g/cm³) reflects the mineral density of quartz as the main component of sandstones. A clear separation between matrix and bulk density values is also necessary for most of the carbonate rocks as they also can exhibit significant porosities (see Fig. 3.1; Table 3.1), which is due to the variability in their depositional characteristics and in their mineralogical composition.

Table 3.1 Modal analysis (vol-%), densities, mean atomic weights (M), and cation packing index (K-values) for various dimension stones with commercial name, country of origin, and rock type given (Strohmeyer 2003, Mosch 2009, Hoffmann 2008, Ruedrich and Siegesmund 2007, Morales 2011 and unpublished data)

Commercial name	Country of origin	Rock type	Modal analysis (vol.-%)	Matrix density (g/cm ³)	Bulk density (g/cm ³)	Mean atomic weight	K-value
Ben Tak Black	Thailand	Granite	6.6qz, 6.6mikr, 40plg, 8.8bio, 24hbl, 8ser, 6chl, acc	2.66	2.64	21.97	5.08
Muang Tak Orange 1	Thailand	Granite	30qz, 34mikr, 27plg, 6ser, 2chl, acc	2.62	2.60	20.90	4.59
Kösseine	Germany	Granite	33qz, 26kspar, 29plg, 10bio, 1chl	2.68	2.67	20.77	4.71
Antrona	Italy	Granite	33qz, 24kspar, 24plg, 15mus, 3bio, acc	2.67	2.65	20.86	4.70
Rojo Dragon	Argentina	Granite	32plg, 27.5mikr, 26.6qz, 13.9phyl	2.63	2.62	20.97	4.72
Padang	China	Granodiorite	19qz, 10kspar, 51plg, 11bio, 7cpx, 2mag, acc	2.80	2.80	20.95	4.98
Ben Tak Blue	Thailand	Quartz-monzonite	16qz, 38mikr, 32plg, 3bio, 2hbl, 2chl, 6ser, acc	2.66	2.64	21.20	4.63
Salmon Red	Uruguay	Syenite	13qz, 53plg, 32mikr, 1hbl, 1chl	2.63	2.62	21.01	5.20
Artigas	Uruguay	Syenite	1qz, 14kspar, 20mikr, 46plg, 1hbl, acc	2.67	2.66	21.34	4.93
Nero Absoluto	Uruguay	Dolerite	15qz, 8kspar, 42plg, 20px, 6hbl, 2ilm, 5bio, acc	2.99	2.99	22.20	5.25
Anzola	Italy	Gabbro	35plg, 44hbl, 16opx and 5cpx	3.06	3.05	21.77	5.59
Nero Impala	South Africa	Gabbro-norite	74plg, 13cpx, 12opx, acc	2.92	2.92	20.69	5.28
Negro Riojano	Argentina	Gabbro	51.4plg, 18.6hbl, 18.6hbl, 5.2phlo, 4.5bio, 0.8chl	2.91	2.91	21.66	5.07
Balmuccia	Italy	Peridotite	59ol, 26opx, 13cpx, 2spi	3.34	3.34	20.28	6.61
Löbejün	Germany	Rhyolite	27qz, 40kspar, 27plg, 5bio, acc	2.64	2.57	21.68	4.66
Stardust Grey	Argentina	Rhyolite		2.54	2.53	21.51	
Monte Merlo	Italy	Trachyte	53kspar, 15plg, 8qz, 8hbl, 8 bio, 7pyr, 4 cal	2.53	2.39	21.38	4.92
Drachenfels	Germany	Trachyte	49kspar, 20plg, 9qz, 7 hbl, 7 bio, 4pyr, 4 cal	2.66	2.30	21.67	4.80
Weibern	Germany	Phonolite tuff	30neph, 25leu, 23kspar, 12nos, 9px, acc	2.56	1.43	22.31	
Rochlitz	Germany	Rhyolite tuff	63.9qz, 33.6phyl, 2.5haem	2.66	1.91	19.72	4.66

Table 3.1 (continued)

Commercial name	Country of origin	Rock type	Modal analysis (vol.-%)	Matrix density (g/cm ³)	Bulk density (g/cm ³)	Mean atomic weight	K-value
Phanom Sarakham Grey	Thailand	Gneiss	32qz, 22mikr, 34plg, 7bio, 6ser, acc	2.67	2.65	20.82	4.78
Azul Tango	Argentina	Gneiss	28.3qz, 27.1cord, 24plg, 16.5grt, 1.8ilm, 1.7chl, 0.7phyl	2.96	2.96	21.63	5.34
Calanca	Italy	Gneiss	34qz, 9kspar, 38plg, 18bio, 1chl	2.73	2.71	20.76	4.80
Verde Andeer	Italy	Gneiss	34qz, 19kspar, 21plg, 22mus, 4chl	2.71	2.68	20.94	4.64
Serizzo Monte Rosa	Italy	Gneiss	32qz, 11kspar, 38plg, 4mus, 13bio, 1chl	2.67	2.64	20.81	4.70
Wang Nam Kiew Black	Thailand	Hornblende	90hbl, 4plg, 4chl, 2 acc	3.17	3.16	22.56	5.57
Franco Veteado	Argentina	Migmatite	46.5qz, 17.5cord, 14.8pla, 14.1bio, 7.1kspar	2.75	2.75	21.12	4.87
Rosa Estremoz	Portugal	Marble	98 cal, 2haem	2.72	2.71	20.03	5.44
Carrara	Italy	Marble	96.8 cal, 3.2dol			20.04	5.40
Phran Kratai Grey	Thailand	Marble	100 cal	2.69	2.68	19.96	5.42
Azul Cielo	Argentina	Marble	100 cal	2.73	2.71	19.84	5.42
Azul Imperial	Brazil	Quartzite	87qz, 9mus, 4dum	2.69	2.69	20.31	4.50
Camino Rufo	Uruguay	Slate	32qz, 38dol, 18mus, 2chl, 9hl	2.81	2.81	20.31	4.67
San Luis Slate	Argentina	Slate	30.2qz, 39.3mus, 15.6plg, 14.9chl	2.79	2.79	23.25	4.56
Theuma	Germany	Slate				21.93	
Globegerina	Malta	Limestone	86-99 cal, 1-12phyl, 1-8qz, 1kspar	2.63	1.63-1.85	19.97	5.33
Caliza Amarilla	Argentina	Limestone	95.5 cal, 4.0qz	2.70	2.39	20.12	5.37
Dolomita Dorada	Argentina	Dolomitic limestone	96dol, 4qz	2.84	2.60	19.16	4.66
Mae Phrik Yellow	Thailand	Limestone	100 cal	2.71	2.70	19.95	5.42
Soskut	Hungary	Limestone	82 cal, 15qz, 2ser, acc	2.68	2.14	20.14	5.21
Jerusalem	Israel	Limestone	95 cal, 3qz, acc	2.68	2.59		5.28
Kuacker	Germany	Limestone		2.75	2.65	19.25	
Thüster	Germany	Limestone	95 cal, 5qz	2.68	2.09	20.15	5.37

Table 3.1 (continued)

Commercial name	Country of origin	Rock type	Modal analysis (vol.-%)	Matrix density (g/cm ³)	Bulk density (g/cm ³)	Mean atomic weight	K-value
Bad Langensalza Baumberger	Germany	Travertine		2.65	2.40	20.64	
	Germany	Sandstone-like limestone	55 cal, 30qz, lkspars, 10ill, 2glau, acc	2.68	2.09	20.27	4.91
Anröchte	Germany	Sandstone-like limestone	64 cal, 18glau, 17qz, acc	2.74	2.45	20.15	4.87
Bad Bentheim	Germany	Sandstone	98qz, 1rf, acc, kaol	2.65	2.04	20.54	4.42
Schleerither	Germany	Sandstone	80qz, 14rf, 3plg, 2phyl, acc	2.72	2.29	21.50	4.42
Weser	Germany	Sandstone	85qz, 1rf, 19fsp, 2phyl	2.65	2.47	20.93	4.36
Sikhiu Brown	Thailand	Sandstone	93qz, 4kaol, 3ill	2.66	2.38	20.48	4.40
Pakchong Green	Thailand	Sandstone	43qz, 8kspar, 25plg, 16ill, 4chl, 3 cal	2.73	2.53	20.93	4.60
Tacuarembó	Uruguay	Sandstone	98qz, 2kspar	2.26	2.03	20.25	4.41

Mineral abbreviations: acc = accessories, bio = biotite, cal = calcite, chl = chlorite, cord = cordierite, ctd = chloritoid, czs = clinzoisite, dol = dolomite, dum = dumortierite, epi = epidote, glau = glauconite, grt/gt = garnet, haem = haematite, hbl = hornblende, hyperst = hypersthene, ill = illite, ilm = ilmenite, kaol = kaolinite, kspar = K-feldspar, leu = leucite, mikr = microcline, mikr = mikroccline, mus/ms = muscovite, neph = nepheline, nos = nosean, ol = olivine, opx = ortho pyroxene, phlo = phlogopite, phyl = phyllite, plg = plagioclase, px = pyroxene, pyr = pyrite, opx = orthopyroxene, cpx = clinopyroxene, qz = quartz, rf = rock fragments, ser = sericite, serp = serpentine, sil = sillimanite, staur = staurolite
Rock abbreviations: metapel = metapelite, myl = mylonite, pend = pendantite. These abbreviations are used throughout this chapter if not otherwise stated.

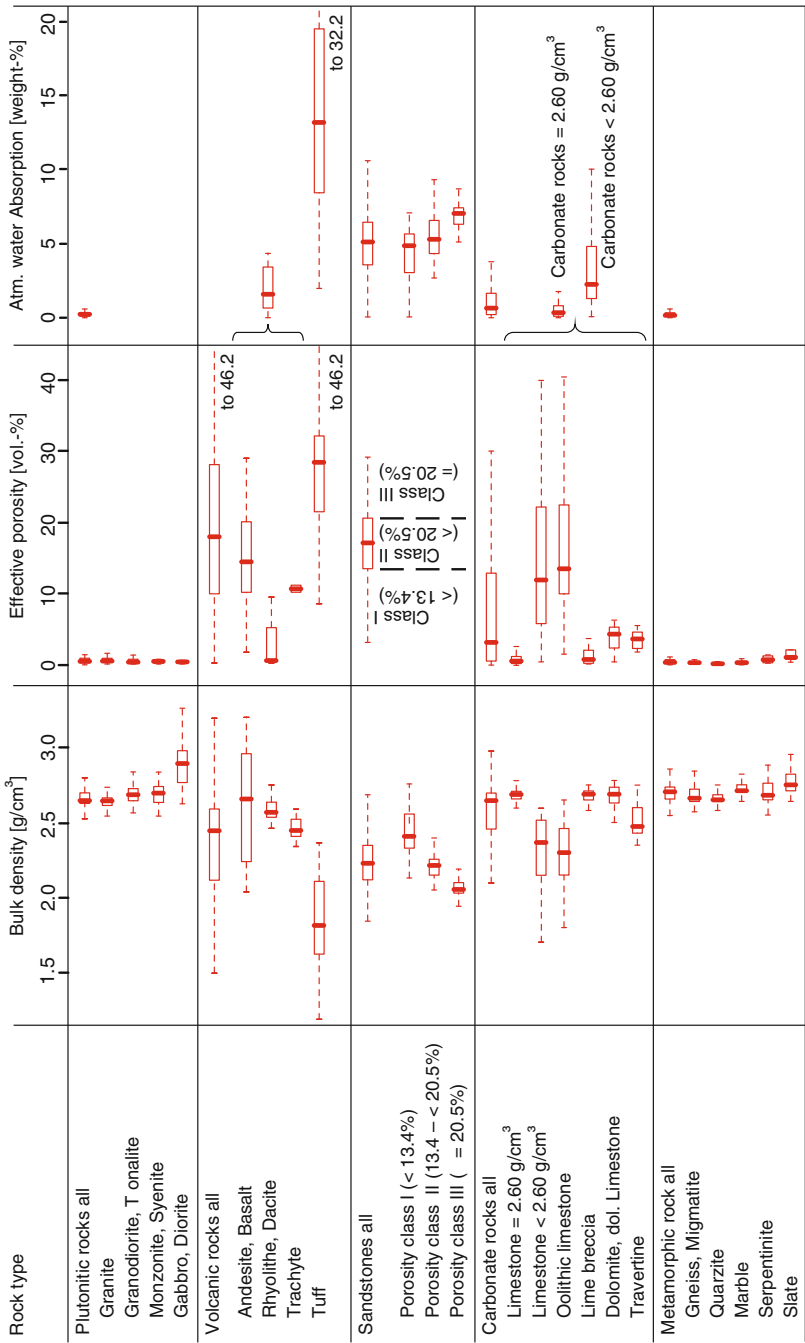


Fig. 3.1 Lithology specific value ranges for bulk density, effective porosity, and water absorption under atmospheric conditions based on a statistical analysis of data from 2,335 dimension stones (Mosch and Siegesmund 2007, Mosch 2009). The box plots describe in each case the median value, the 25%- and 75%-quartile, as well as the upper and lower extreme values

The highest median values of the bulk density in the group of sedimentary rocks can be found in the subgroups of dolomite and lime-breccias. The limestone group also exhibits a quite higher median value, however with a much larger range and a higher number of outliers. The lowest median values are reported for the subgroup of oolitic limestone, also with a wide range of data, but of uniform distribution. Generally, the values and the distribution of the porosity show a diametrical trend in comparison to the bulk density (see Sect. 3.2) as expected due to the definition of the bulk density.

The group of metamorphic rocks with its subgroups show no large ranges in the data distribution (see Fig. 3.1, Table 3.1). The highest median value here can be found in the schist group with 2.75 g/cm^3 . For the group of marbles (2.71 g/cm^3), gneisses/migmatites (2.67 g/cm^3), and quartzites (2.65 g/cm^3) the median value of the bulk density reflects the mineral specific or matrix density of the main mineral phases and by this indicate an almost zero percent porosity (see Table 3.1). Since marble and quartzite can be assumed to be more or less monomineralic rocks, the matrix density is related to calcite and quartz, respectively. The gneisses can be better compared to the plutonic rocks as in both groups the main mineral phases are feldspar and quartz. The subgroup serpentinite has a median value of 2.68 g/cm^3 combined with a larger data range; both can be related to the occurrence of minerals with higher matrix density values remaining from the earlier peridotite stage.

After studying the density and chemical composition (SiO_2 -content), Popp (1994) could separate three major rock groups (Fig. 3.2): felsic with $\rho < 2.8 \text{ g/cm}^3$ und SiO_2 -content $> 70\%$, intermediate with $2.8 \text{ g/cm}^3 < \rho < 2.9 \text{ g/cm}^3$ and $55\% < \text{SiO}_2$ -content $< 70\%$, and the mafic group with $\rho > 2.9 \text{ g/cm}^3$ and SiO_2 -content $< 55\%$.

Classifications like this or others are of importance as several authors have proposed similar empirical relationships in order to estimate certain geotechnical rock properties from the density values alone (Fig. 3.3). Examples are the empirically derived ultrasonic wave velocity-density relationships (Wollard curve, Wollard 1959; Nafe-Drake-curve, Nafe und Drake 1963), which attempts to estimate the velocities from the densities and vice versa (see Sects. 3.2 and 3.7).

Birch (1961) developed another velocity-density relationship as follows:

$$V_p = a(M) \cdot b \cdot \rho, \quad (3.3)$$

where a is a function of the mean atomic weight (M), b a constant, and ρ the density. The mean atomic weight can be determined from the chemical analysis of the main element oxides as follows:

$$M = (\sum x_i/M_i) - 1, \quad (3.4)$$

where M_i is the mean atomic weight of the i -th oxide of the rock sample, and x_i the weight percentage of the i -th oxide. In general, there is a good correlation between the ultrasonic velocities and the density (Fig. 3.4). The observation that velocity-density relationships are more or less parallel to lines of constant mean atomic weight values is well known in the literature, for example Christensen (1968). The

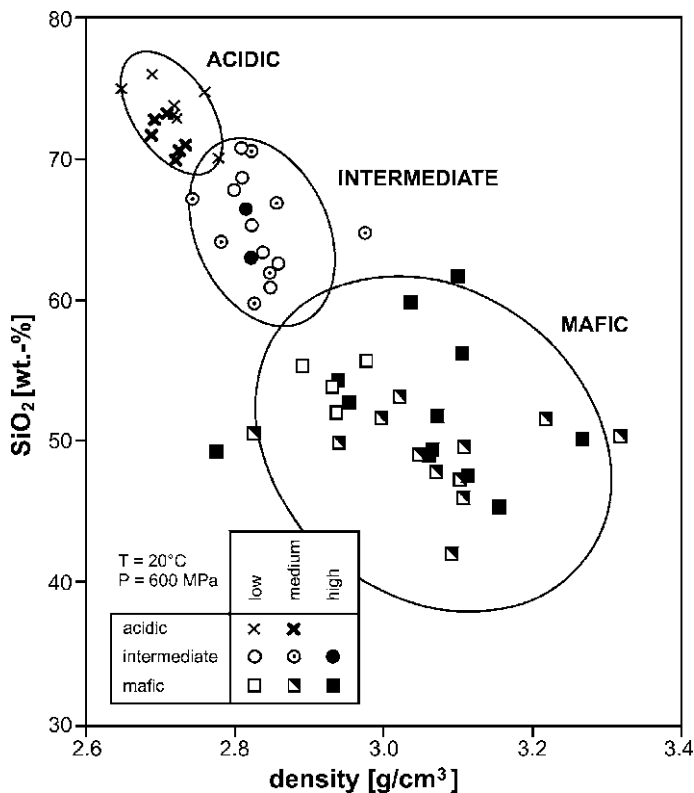


Fig. 3.2 Correlation of density with the SiO₂ content to differentiate between acid, intermediate and mafic rocks (after Popp 1994)

mean atomic weights in Table 3.1 range from 19.70 to 22.31, with the highest values being calculated for the basic/ultrabasic rocks. For these rocks an increase of the fayalite content in the olivine minerals of 10–25% comes with a decrease in the seismic velocities of 0.25 km/s and a density increase of 0.2 g/cm³. The mean atomic weight hereby varies between 20.10 for forsterite (Mg₂SiO₄) and 29.70 for fayalite (Fe₂SiO₄). The large fluctuation in the mean atomic weight of the olivine varieties comes from the lower atomic weight of the Mg in comparison to the Fe. Higher mean atomic weight values, greater than 21.50, can be related to higher contents of

Fig. 3.3 Correlation of the volume percent of minerals (left) with the variation in density (g/cm³) and ultrasonic compressional wave velocity (V_p in km/s; at 600 MPa, 20°C) (right). The samples in each group are arranged based on increasing density. In addition the Voigt-Reuss-Hill (VRH) mean V_p-values of minerals are given (bottom). The influence of the mineralogical composition is clearly visible on the density and also on the P-wave velocity (after Popp 1994); for mineral abbreviations see Table 3.1

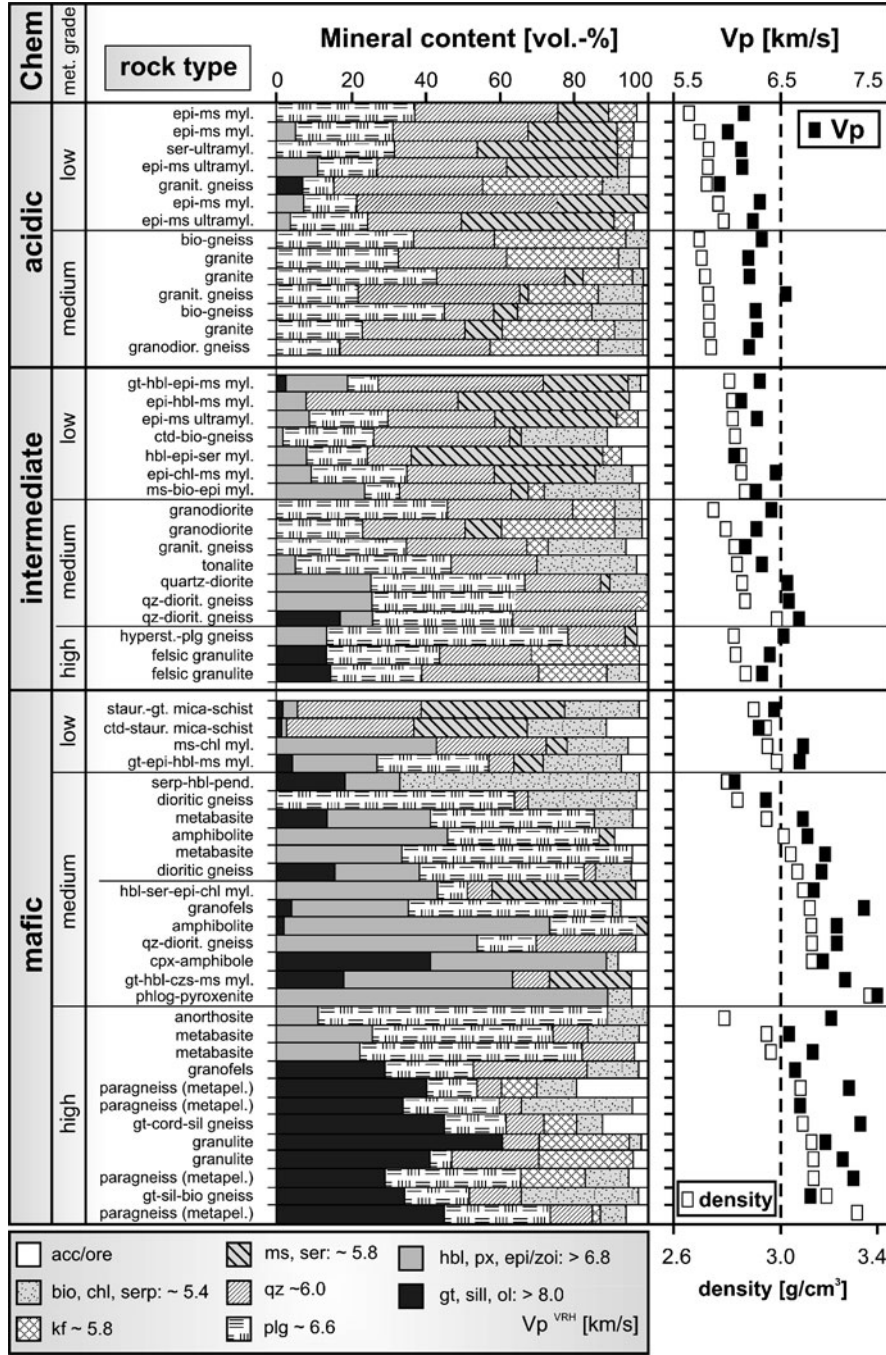


Fig. 3.3 (continued)

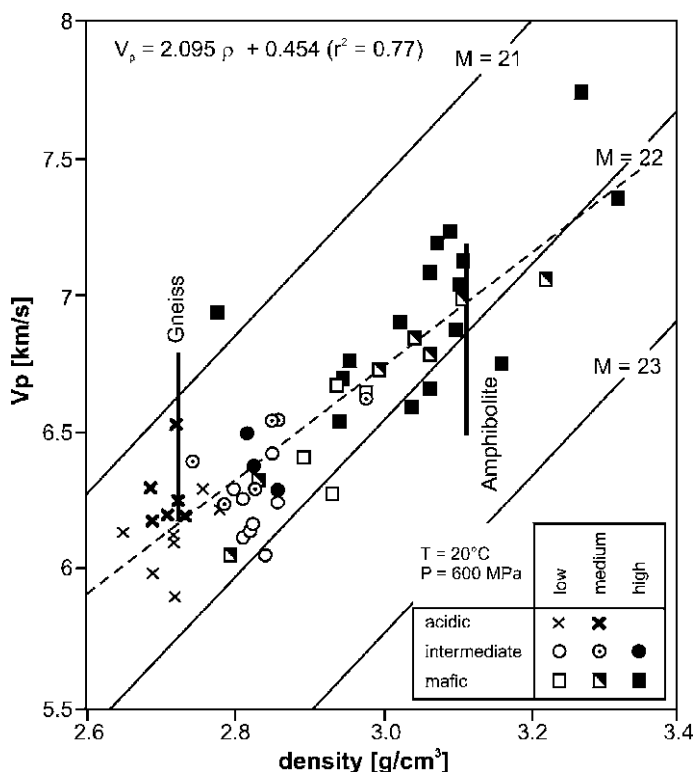


Fig. 3.4 Velocity-density relationships for V_p (600 MPa, 20°C), after Popp (1994). The lines of constant mean atomic weight (M) originate from Birch (1961) and Christensen (1968). Additionally shown are the extreme values of the experimental determined P-wave velocities for a gneiss and an amphibolite sample

Ca, Ti, and Fe, mainly found in hornblende, garnet, pyroxene, and epidote minerals (see Table 3.1). In general, seismic velocities, densities, and mean atomic weight values can be correlated with each other.

A more promising relationship than using the mean atomic weights was introduced by Buntebarth (1982) and Rybach and Buntebarth (1982) with the use of the cation packing index or k -value, with $k = (\text{the number of cations per mole}) / (\text{Avogadro's number} \cdot \text{molar volume})$. For the whole rock the k -value can be determined with the following equation:

$$k = \sum x_i \cdot k_i \quad (3.5)$$

where k_i is the k -value of the i -th mineral phase, and x_i is the amount of the i -th mineral in the rock determined by modal analysis. However, the empirically derived

correlation of Rybach and Buntebarth (1982) with a more or less linear relationship between the P-wave velocities and the densities of various rocks could not be verified by Kern and Siegesmund (1989). On the other hand, the k-value correlates much better with the density values than the mean atomic weight. For further verification sufficient data sets are needed for statistical analyses.

Ceryan et al. (2008) could show that the k-value of granitic rocks decrease with increasing weathering. This correlation reflects the observation that during the weathering process of granitic rocks, for example, feldspar minerals (plagioclase k-values $4.97\text{--}4.99 \times 10^{-2} \text{ mol/cm}^3$) disintegrate and decompose to clay minerals (k-values $3.997\text{--}4.52 \times 10^{-2} \text{ mol/cm}^3$), and biotite ($4.656 \times 10^{-2} \text{ mol/cm}^3$) and hornblende ($5.309 \times 10^{-2} \text{ mol/cm}^3$) have changed into chlorite ($4.10 \times 10^{-2} \text{ mol/cm}^3$), Fe-oxides and clay minerals. Since the main cause of the weathering is leaching, the densities and the k-values of the resulting minerals, the weathering products, are lower than those of the original ones. Based on their results Ceryan et al. (2008) proposed that the k-value has the potential for being applied in investigating the engineering lifetime of building stones.

3.2 Porosity

The porosity of a rock is defined by the ratio of the pore volume (e.g. pores, open cracks) to the volume of the whole rock. It can be calculated by applying the same variables used for the determination of the bulk and matrix density. Two different types of porosity can be distinguished, the effective porosity or often called “accessible porosity” and the total porosity. The first one comprises all the pore spaces in a rock, where fluids and gases can access, whereas the latter one includes all pores, even the isolated ones, which cannot be accessed (Fig. 3.5). For weathering processes, the effective porosity is of primary interest (e.g. Weiss 1992). For rocks with relatively large pore space values, the effective porosity is assumed to be the dominating type (e.g. Fitzner and Basten 1994). Based on the definition of both porosity types, the effective porosity is either equal or smaller than the total porosity; however, the difference might be small.

The porosity has a direct and indirect effect on most of the physical properties of rocks, and is therefore considered the most important rock parameter. An increasing porosity has an unfavourable influence on the weathering characteristics (e.g. Hirschwald 1912, Fitzner 1970, Poschlod 1990, Fitzner and Snethlage 1983, Viles et al. 1997, Ruedrich et al. 2005b, Ruedrich et al. 2010a). All “hollow spaces or cavities” in a rock can be classified under the term “pores”. For the description of the pore space, different classification schemes were developed depending mainly on the application (see e.g. Fitzner and Basten 1994).

- (i) The pore space classification under petrogenetic aspects distinguishes between the primary porosity as the result of sedimentary processes and the secondary porosity as the result of post-sedimentary or diagenetic processes including all

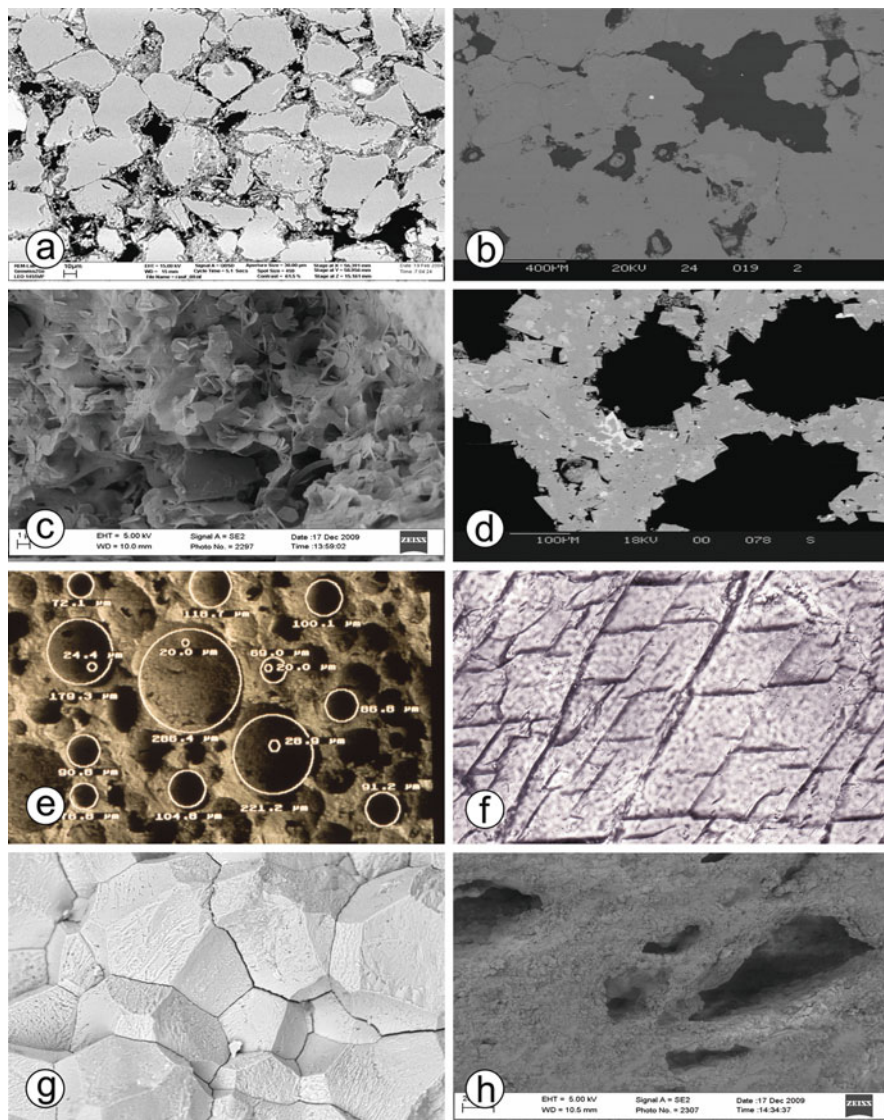


Fig. 3.5 SEM images showing rocks with their different pore space and porosity: (a) and (b) Sandstone with open pores (black) interconnected by open cracks, (c) claymineral-rich sandstone with a very heterogeneous pore sizes ranging from micro-to macropores, (d) Dolomitic limestone with nearly spherical pore shape and dolomite crystals, (e) volcanic tuff with intraparticle pores of different sizes (Figure by B. Fitzner), (f) open cleavage cracks and transgranular cracks in hornblende mineral also known as platy pores, (g) open grain boundaries in weathered marbles and (h) pores in travertine with a heterogeneous pore size distribution ranging up to several centimetres in size

processes after the primary sedimentation. The so-called “weathering porosity” of dimension stones is therefore a special case of the secondary porosity.

- (ii) Another classification scheme is based on the separation of ideal single pore types by applying the pore geometry and the pore genesis, for example, spherical pores, platy pores, cylindrical pores, interstitial pores (near the narrowing area of the grain to grain contact), crack pores, dissolution pores, and shrinking pores (see Fig. 3.6).
- (iii) A different pore space classification is using the location of the pore in relation to the solid particles (grains, minerals). Pores between the solid particles are called “interparticle porosity”, whereas pores in a solid rock particle are called “intraparticle porosity” or “intercrystalline porosity” at the crystal lattice scale (Fig. 3.5).
- (iv) Another classification scheme applies the criterion of how well fluids or gases can penetrate through the pore system of a rock. Good transmission is provided through a system of interconnected pores with channel pores, ink-bottle pores, and telescope pores, a dead-end transmission by dead end pores and pigeon hole pores. No transmission occurs with isolated pores (see Fig. 3.7).
- (v) Without using other attributes, the pore size alone is often used for the classification of pores.

Using the effective porosity value von Moos and Quervain (1948) developed a classification scheme for rocks, with <1% compact, 1–2.5% a few pores, 2.5–5% slightly porous, 5–10% significantly porous, 10–20% many pores, and more than 20% means a lot of pore space (Table 3.2).

The size of a pore is given by its pore radius, which is determined from the radius of a cylinder representing the ideal form of a pore. However, natural pores usually exhibit a more complex geometrical form than a cylinder. Therefore, the determination of the pore size only reflects the smallest diameter or radius of a pore, which is usually the throat (or entry) of a pore or crack, and thus it defines the pore throat radius. Various pore size classifications have been published over time, with some shown in Table 3.2, mainly dividing the pore space into micro-, meso-, and macropores. However, it is quite difficult to get a consistent classification scheme as the boundaries between the classes vary.

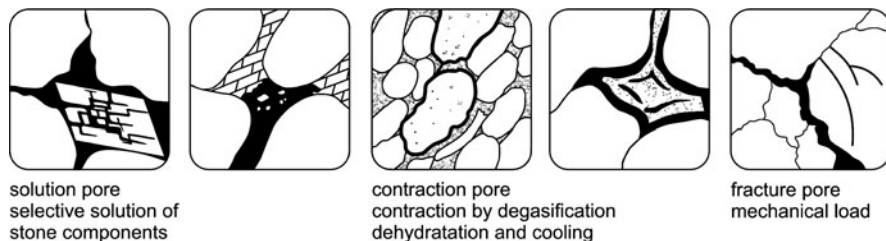
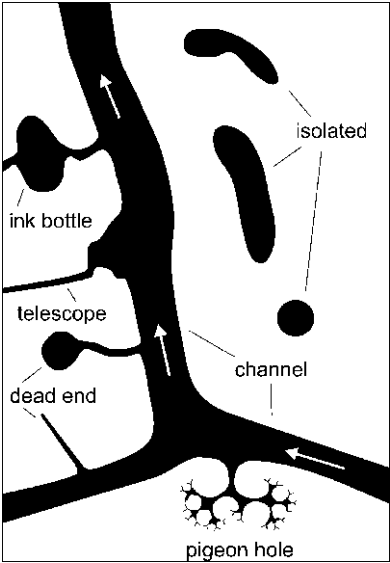


Fig. 3.6 Pore space types (after Fitzner and Basten 1994)

Fig. 3.7 Pore space classification using the transmission or penetration criterion (after Fitzner and Basten 1994)



The method for the determination of the pore (throat) radius distribution (see Brakel et al. 1981) is based on the Washburn equation (Washburn 1921), which is derived by combining Poiseuille’s Law for viscous flow and the Young-Laplace Equation for capillarity:

$$r_p = -2\gamma \cos \theta / P \tag{3.6}$$

Table 3.2 Comparison of some often used pore size classification schemes exhibiting variations in the ranges depending on the authors

	Micropores (μm)	Mesopores (μm)	Macropores (μm)	Large pores (μm)
De Quervain (1967)	<5	5–200 (fine pores)	200–2,000 (coarse pores)	>2,000
Dubinin (1979)	0.003 to 0.0032– 0.0012 to 0.0014	0.2 to 0.4– 0.003 to 0.0032	<0.2–0.4	
IUPAC (Gregg and Sing 1982)	<0.002	0.002–0.05	>0.05	
Klopper (1985)	<0.1	0.1–1,000 (capillary pores)	>1,000	
DIN 66131 (1993)	<0.002	0.002–0.05	>0.05	
Kodikara et al. (1999)	1 to 30		10–1,000	

with r_p the pore throat radius of a capillary pore, γ the surface tension at the liquid interface, θ the liquid-solid contact angle, and P the applied pressure.

The equation is based on the principle that every capillary pore radius is characterised by a certain pressure value that is needed to push a fluid into the pore (Doveton 1987). This relationship holds only for non-wetting fluids with a contact angle of $\theta > 90^\circ$, like mercury, which is a non-wetting fluid for almost all substances. Therefore, laboratory work mainly uses the mercury (intrusion) porosimetry technique that allows the pore radius determination for the whole range of possible radii found in rocks. This method is based on pressure-related mercury intrusion and extrusion, and can be realised with commercial equipment, which can measure the porosity and the pore size radii distribution in a range from ca. $0.0020 \mu\text{m}$ to $200 \mu\text{m}$.

Fitzner and Kownatzki (1991), Fitzner and Basten (1994), Meng (1993) or Heinrichs (2005) showed that with a combination of different methods a quite realistic image of the pore system can be achieved (e.g. Figs. 3.8 and 3.9). Between ca. $0.001 \mu\text{m}$ and $0.1 \mu\text{m}$ the pore radii distribution and pore volume can be determined from the nitrogen sorption method based on the Brunauer-Emmet-Teller (BET) theory (see Brunauer et al. 1938). The pore radii distribution between $0.1 \mu\text{m}$ and $4.0 \mu\text{m}$ can only be reliably determined with the mercury porosimetry, whereas for pores with a radius $>4.0 \mu\text{m}$, the macropore class, a combination between mercury porosimetry and image analysis based on thin sections was more successful. The determination of the total porosity can be achieved with different methods with the buoyancy method based on the Archimedes principle as a reasonable and efficient method (see Sect. 3.1).

For sandstones (see also Sect. 2.11), for example, the pore space or porosity is developed during sedimentary deposition with the packing density of the grains as one of the main parameters (Tucker 1985). The packing density is mainly influenced by the grain size, the grain shape, and the grain sorting as the result of the

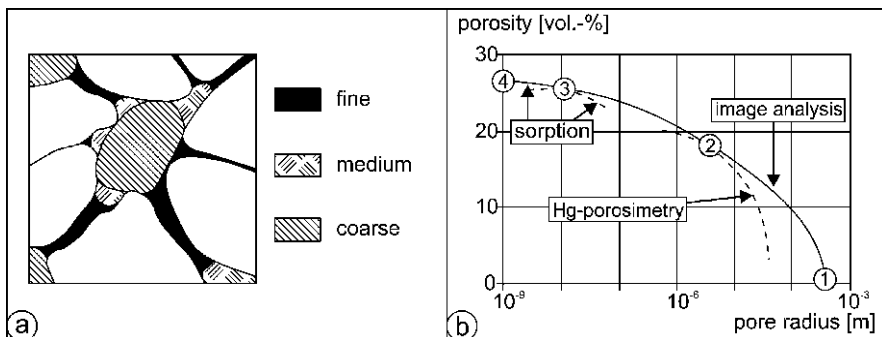


Fig. 3.8 Pore space in geo-materials: (a) Schematic diagram of the different pore sizes, (b) scale dependency of different methods for pore space characterisation (after Meng 1995)

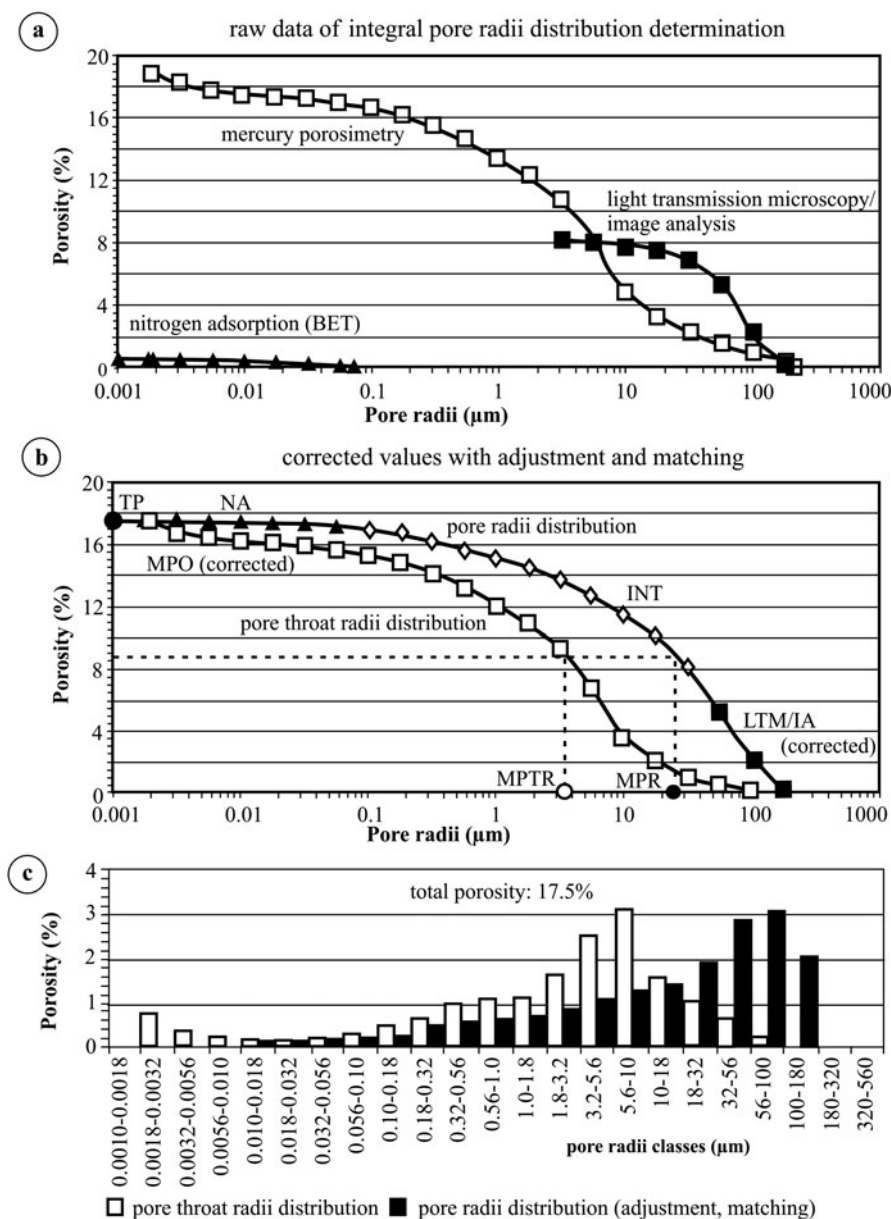


Fig. 3.9 (continued)

depositional environment. During later diagenetic processes, the primary porosity will change through compaction and water loss, through pressure solution at grain boundaries, and through cementation. The alteration of existing minerals and the precipitation of new minerals lead to a progressive decrease in porosity with deeper

burial. However, local fluids in the subsurface can also dissolve primary or even secondary minerals resulting in higher porosity values.

The porosity in each of the main sedimentary rock types can vary significantly (Fig. 3.1 or Table 3.3). Using 360 sandstone varieties Mosch and Siegesmund (2007) have shown that the porosity values range from 0.1% to 29.0% (Fig. 3.10). The reason for this lies in the variability of geological processes and variations in the depositional environment leading to the same rock but to different rock types. For example, beach and barren sands usually exhibit a good sorting with a narrow grain size distribution, whereas river and deltaic sands show a much more heterogeneous distribution of the grain sizes as well as a diversity in the mineralogical composition.

The observed pore size distributions of sandstones can be classified according to Ruedrich and Siegesmund (2006) into three types (Fig. 3.10): (A) unimodal distribution over a narrow pore size range, (B) uneven distribution, and (C) bimodal distribution or a plot showing one sub-maximum. For the capillarity and fluid transport in the pore space the pores with a radius in the range of <1 mm to ca. $0.1 \mu\text{m}$ are of great significance. In pores smaller than those, the micropores, the moisture transport is done by surface and solution diffusion processes. Micropores are important for questions related to the drying of rocks and for the penetration of substances with a defined molecule size for conservation and impregnation. In the macropores, however, the water loses its interconnectivity and thus cannot move freely any more.

During salt crystallisation experiments Fitzner and Basten (1994), Fitzner and Snethlage (1983) and Ruedrich et al. (2005b, 2007) could clearly show that sandstones with higher porosity are more sensitive to salt crystallisation than sandstones with lower porosity values (see Chap. 4). Also for lower tensile strength values, a higher sensitivity could be seen here. However, these trends show exceptions that are mainly related to the pore radii distribution. According to Ruedrich et al. (2005b), sandstone samples with a unimodal pore size distribution (Type A) and a small percentage of micropores could complete more cycles during the salt crystallisation experiment than sandstones with a larger percentage of micropores (Type C).

Fig. 3.9 Porosity values based on image analysis, mercury porosimetry, and the nitrogen adsorption BET method for the high porous Petra Sandstone. (a) raw data, (b) corrected values with adjustment and matching, and (c) differential distribution of the pore radii distribution and pore throat distribution after data correction (after Heinrichs 2005). NA – porosity part related to the lower radii classes/measurement data from the nitrogen adsorption method (BET), LTM/Ia (corrected) – porosity part related to the upper radii classes/corrected measurement data from light transmission microscopy/image analysis, INT – interpolated porosity parts of the middle radii classes, MPO (corrected) – corrected measurement data of the mercury porosimetry = corrected pore throat radii distribution, TP – total porosity/data adjustment and matching, MPTR – mean pore throat radius /corrected values from the mercury porosimetry, MPR – mean pore radius/data adjustment and matching

Table 3.3 Porosity, mean pore radius, water uptake at atmospheric conditions, degree of saturation, and vapor diffusion resistance factor for various dimension stones with commercial name, country of origin, and rock type given (Strohmeyer 2003, Mosch 2009, Hoffmann 2008, Ruedrich and Siegesmund 2007, Morales 2011 and unpublished data)

Commercial name	Country of origin	Rock type	Porosity (%)	Mean pore radius (μm)	Water uptake at atmospheric pressure (weight-%)		
					Degree of saturation (%)	Vapor diffusion resistance factor	
Ben Tak White	Thailand	Granite	0.59	0.046		0.83	2,815.8 (5.2)
Muang Tak Orange 1	Thailand	Granite	0.76	0.054		0.73	3,869.6 (10.2)
Kösseine	Germany	Granite	0.11	0.02	0.003		
Antrona	Italy	Granite	1.03	0.09	0.19		
Rojo Dragon	Argentina	Granite	0.45	0.088		0.81	1,416.1 (18.6)
Padang	China	Granodiorite	0.25	0.03	0.08		
Ben Tak Blue	Thailand	Quartz-monzonite	0.91	0.065		0.60	3,132.6 (51.1)
Salmon Red	Uruguay	Syenite	0.60	0.042			
Artigas	Uruguay	Syenite	0.72	0.077			
Nero Assolutto	Uruguay	Dolerite	0.05	0.097			
Anzola	Italy	Gabbro	0.31	0.10	0.02		
Nero Impala	South Africa	Gabbro-norite	0.04				
Negro Riojano	Argentina	Gabbro	0.04	0.007		0.53	2,321–5,927
Balmuccia	Italy	Peridotite	0.03				
Löbejün	Germany	Rhyolite	2.30		1.44	0.77	
Stardust Grey	Argentina	Rhyolite	0.60	0.010		0.85	1,187.0 (69.1)
Monte Merlo	Italy	Trachyte	11.76				
Drachenfels	Germany	Trachyte	11.92				
Weibern	Germany	Phonolite tuff	43.08		16.45		
Rochlitz	Germany	Rhyolite tuff	28.27	0.11–0.30	3.54	0.77	8.8–18.2
Eger-Dejnen	Hungary	Dacite tuff	34.76		8.54		
Phanom Sarakham Grey	Thailand	Gneiss	0.71	0.036		0.81	3,132.6 (51.1)
Azul Tango	Argentina	Gneiss	0.16	0.097		0.95	1,442.9 (15.4)
Calanca	Italy	Gneiss	0.73	0.16			

Table 3.3 (continued)

Commercial name	Country of origin	Rock type	Porosity (%)	Mean pore radius (μm)	Water uptake at atmospheric pressure (weight-%)	Degree of saturation (%)	Vapor diffusion resistance factor
Verde Andeer	Italy	Gneiss	0.89	0.08			
Serizzo Monte Rosa	Italy	Gneiss	1.01	0.16			
Wang Nam Kiew Black	Thailand	Hornblendite	0.55	0.163		0.83	2,813.9 (17.9)
Franco Vetado	Argentina	Migmatite	0.33	0.204		0.51	811.0 (29.4)
Rosa Estremoz	Portugal	Marble	0.14	0.06	0.02		
Carrara	Italy	Marble					
Phran Kratai Grey	Thailand	Marble	0.60	0.013		0.92	
Azul Cielo	Argentina	Marble	0.41	0.658		0.75	1,252.1 (49.0)
Azul Imperial	Brazil	Quartzite	0.18	0.01	0.02		
Camine Rufo	Uruguay	Slate	0.16	0.056			
San Luis Slate	Argentina	Slate	0.13	0.177		0.82	486.3 (73.0)
Theuma	Germany	Slate					
Globegerina	Malta	Limestone	34.59	0.56	14.96	0.76	
Caliza Amarilla	Argentina	Limestone	11.4	0.225	2.02 (40.4)	0.75	156.2 (20.2)
Dolomita Dorada	Argentina	Dolomitic limestone	8.51	0.968	0.95 (31.8)	0.44	1,070.16 (10.2)
Mae Phrik Yellow	Thailand	Limestone	0.52	0.07		0.74	
Soskut	Hungary	Limestone	20.08			0.65	33–40
Jerusalem	Israel	Limestone	3.32			0.92	187–554
Kuacker	Germany	Limestone	3.73	0.37			
Thüster	Germany	Limestone	21.86			0.68	15–28
Bad Langensalza	Germany	Travertine	9.12	8.41			
Baumberger	Hungary	Travertine			8.10	0.80	19–27
	Germany	Sandstone-like	21.28	2.16			
	Germany	Limestone					
Rüthener	Germany	Sandstone	25.57	3.2	20.4	0.79	9–11

Table 3.3 (continued)

Commercial name	Country of origin	Rock type	Porosity (%)	Mean pore radius (μm)	Water uptake at atmospheric pressure (weight-%)	Degree of saturation (%)	Vapor diffusion resistance factor
Bad Bentheim	Germany	Sandstone	23.33	9.15	17.9	0.72	11–13
Schleerithier	Germany	Sandstone	14.15	0.105		0.77	22–26
Weser	Germany	Sandstone	6.29	0.06	3.2	0.63	40–45
Sikhiu Brown	Thailand	Sandstone	10.52	0.144	6.56 (73.4)	0.57	
Pakchong Green	Thailand	Sandstone	7.23	0.053	4.12 (15.6)	0.85	
Tacuarembó	Uruguay	Sandstone	22.81	12.09			

Numbers in brackets refers to the measured anisotropy

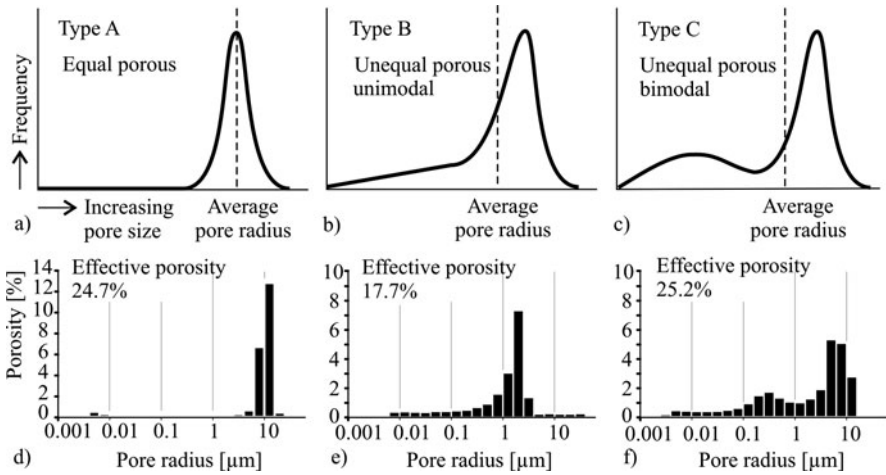


Fig. 3.10 (a–c) Schematic representation of ideal pore types and pore radii distribution in sandstones with Type A equal, Type B unequal unimodal, and Type C unequal bimodal pore size distribution. Examples for each type are shown below with (d) Bad Bentheimer Sandstone (for A), (e) Obernkirchner Sandstone (for B), and Cottaer Sandstone (for C) (from Ruedrich and Siegesmund 2006)

In carbonate rocks, various pore types can be distinguished (see also Koch and Sobott 2005). All of them are the result of the primary distribution of different components in a matrix influenced by secondary diagenetic processes (e.g. Clemens et al. 1990). During the early water expulsion stage, the mechanical compaction resulting from the overburden pressure is the dominant mechanism for porosity reduction (Schlanger and Douglas 1974; Kim et al. 1985). Archie (1952) was the first one who classified carbonate rocks by using their pore space. Choquette and Pray (1970) presented a fabric-selectivity concept that is based on the differentiation between primary and secondary porosity and to some extent on the degree of interconnectivity between the pores. Lucia (1983, 1995, 1999) added a more petrophysical view to the classification of the pore space: the pore size distribution controls the porosity, the permeability and the saturation, and it is related to the rock fabrics. These considerations and investigations led to two major pore space groups: (a) the interparticle pore space and (b) the vuggy pore space, which is divided into separate-vug pores and touching-vug pores. For the first group, Lucia (1995) characterised three rock fabric/petrophysical classes defined by certain permeability and water saturation fields. In the pore space classification of Lucia (1995) fractures are one type of touching-vug pores. The fractures can be separated from the pore space, because they can contribute to anisotropy as well as to porosity and permeability. The fracture intensity in dolomites is higher than in limy dolomites and in limestones (Stearns 1967; Sinclair 1980), and with decreasing grain size the fracture intensity in dolomites increases (Sinclair 1980). For the examination of fractures, van Golf-Racht (1996) presented a descriptive classification scheme

defining the following categories: (a) open/closed fractures, (b) macro/micro fractures and (c) natural/induced fractures.

A study by Mosch and Siegesmund (2007) has shown that carbonate rocks, separated in limestone, oolite, breccia, dolomite and travertine, cover a large range of porosity values, from very compact rocks with <1% to highly porous rocks with >20% as shown in Tables 3.1 and 3.3 and Fig. 3.2 (e.g. Siegesmund et al. 2010).

The largest variation in the porosity values is realised in the travertine group. Peschel (1977), for example, presents a value of 60% as the upper limit for the porosity of travertine, including lime tuff. Furthermore, travertine shows an extreme heterogeneity with respect to porosity and the distribution of the pores within the rock. For example, massive travertine has porosities of <10%, laminated travertine can show alternating layers with more and with fewer pores; and the phytothermal travertine can contain up to 40% plant fragments. Porosity measurements of the travertine *Romano Classico* show values between 7% and 15%, which results from the differences in the pore space distribution in different layers (Figs. 3.11 and 3.12).

Volcanic rocks, like andesite, basalt, rhyolite, dacite, trachyte, and tuff, also show a larger variability for the porosity values, from 1% to ca. 50% (Fig. 3.1, Table 3.3). This large range in pore space volume is the result of their geological origin and the environment at the time of deposition. One possible reason is that the degassing of a fast flowing magma combined with a rapid cooling might result in a pronounced bubble-like structure. Tuff, one of the few volcanic rocks used as a dimension stone, is mainly comprised of scoria, ash particles, and rock fragments, which are deposited during an explosive eruption and subsequently compacted and welded together (Chap. 2). As a result, the rocks show a greater variety in the porosity values, from 8% to nearly 50%.

Plutonic and metamorphic rocks only exhibit lower porosities due to their geological origin. The values for plutonic rocks range between 0.05% and 1.65%, with some outliers, whereas for metamorphic rocks the porosity values are not above 1%, except in a few cases (see Mosch and Siegesmund 2007). The majority of the pore spaces in plutonic and metamorphic rocks can be assumed to be characterised as crack porosity.

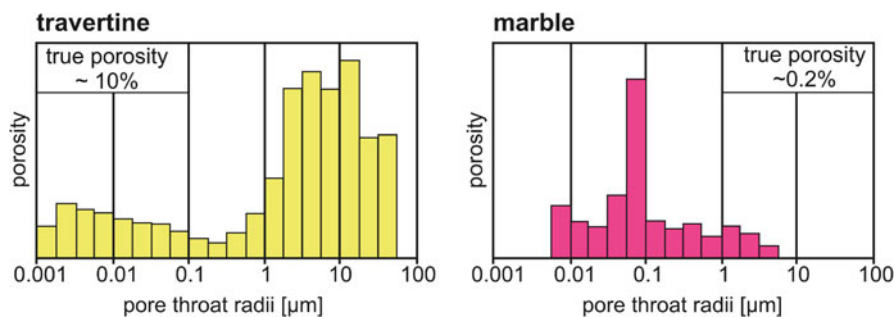


Fig. 3.11 Pore radii distribution of a travertine and a marble sample, both quarry-fresh. Note the differences in the true porosity

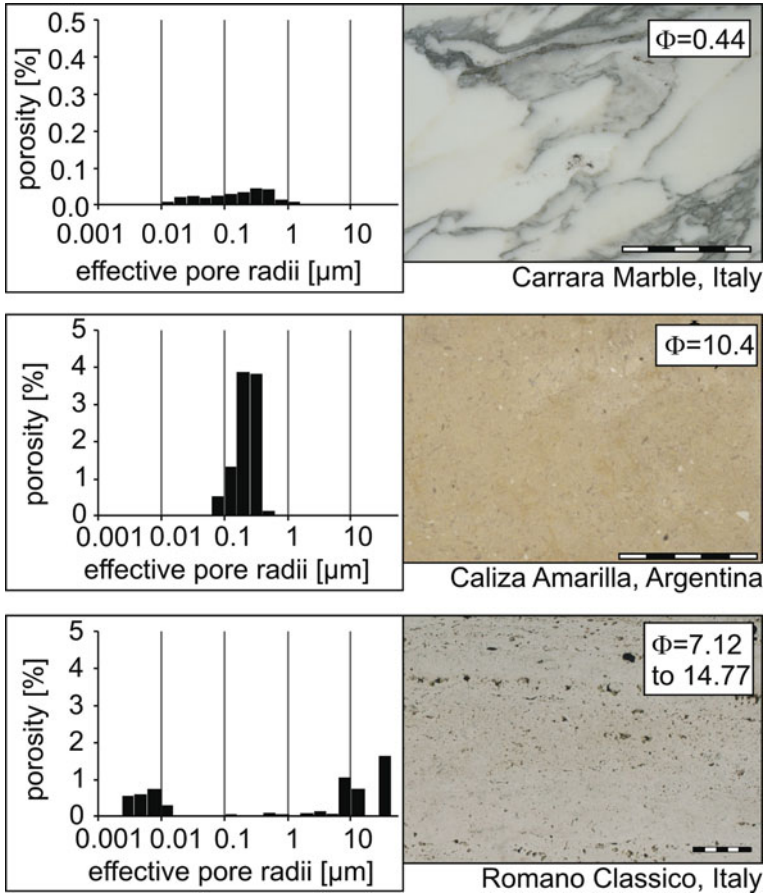


Fig. 3.12 Pore radii distribution for different carbonate rocks such as marbles, micritic limestone and a travertine (porosity is given in %)

Figure 3.13 demonstrates the relationship between the porosity and pore size distribution and the degree of bowing of marble panels (see Sect. 3.4.2 and Siegesmund et al. 2008b). The indoor exposed marble (sample S0) exhibits an initial porosity of around 0.51% (Fig. 3.13a). With increasing bowing, i.e. progressive deterioration, the porosity increases up to 2% (Fig. 3.13). This increase reflects an extreme value compared to porosities usually reported for marbles. Moreover, the pore size maximum moves towards larger pore radii with an increase in bowing. For example, marble sample S7 (3.13b) shows a significant difference in the upper pore radius range. The maximum pore diameter of this sample is around 1.0 μm, and therefore in the range of capillary pores. In contrast, the maximum pores in S0 are smaller than 0.1 μm, which cannot be filled by capillary water absorption. This phenomenon is also observed in general when comparing fresh and artificially weathered marble samples.

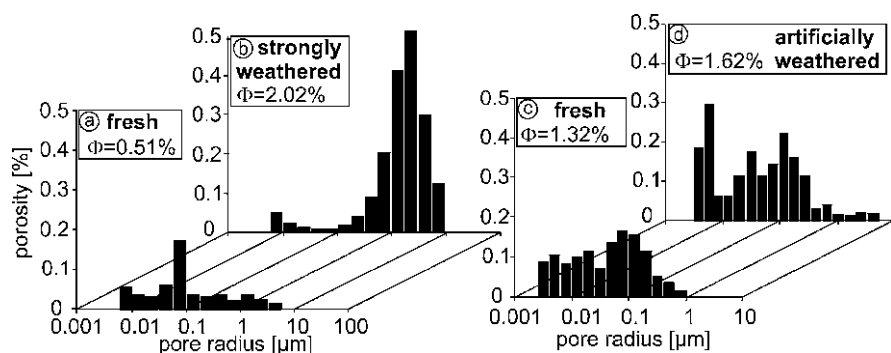


Fig. 3.13 Pore size distribution versus bowing of marble panels: (a) Marble sample S0 with a typical porosity of marbles. With increasing deterioration (b), i.e. bowing of the sample (marble sample S7) the porosity increases and a shift in the pore radii distribution can be recognised. The change in the effective porosity is also given for a fresh (c) and artificially weathered (d) marble sample

3.3 Water Transport and Retention Properties

Water is one of the main factors involved in most of the weathering processes (Snethlage 1984, Weiss 1992, Künzel 1994, Krus 1995, Künzel and Krus 1995, Mirwald 1997). Almost all of the weathering processes taking place at structures and buildings are controlled by the presence of water (e.g., frost and salt-related weathering (see Chap. 4); dry-wet cycles; hygric expansion, thermo-hygric processes), and thus the characteristics of water have an immense impact on the long term stability of dimension stones. The presence of water also has a significant effect on the petrophysical and mechanical properties of rocks. This is mainly influenced by the moisture content, which on the other hand is controlled by the pore space of the rock (see Sect. 3.2). The pore space and pore system relates to the amount of water absorption, the ability of the water transport, and the size of the surface area available for chemical interactions. The important so-called hygric properties here are adsorption, water vapor permeability, capillary water absorption, total amount of absorbed water, and drying characteristics.

Klopfer (1985) correlated pore sizes with the water transport and other mechanisms (see also Sect. 3.2). Three clear regions can be differentiated depending on the mechanism involved. For micropores, i.e., pore diameters $<0.1 \mu\text{m}$ water will condense at relative humidity values below 99% RH (relative humidity) (Fig. 3.14). Capillary suction is practically relevant to materials for pore diameters between $1 \mu\text{m}$ and 1 mm , the so-called capillary pores. For macropores, i.e., those with a diameter greater than 1 mm , fluid flow characteristics emerge.

The main moisture transport mechanisms are water vapor diffusion and the liquid water capillary absorption. Between these two mechanisms a surface diffusion of sorbed water has been postulated (Stockhausen 1981, Klopfer 1985, Künzel 1994, Krus 1995). The interaction between these three moisture transport mechanisms is illustrated in Fig. 3.15. The driving force for water vapor diffusion is the

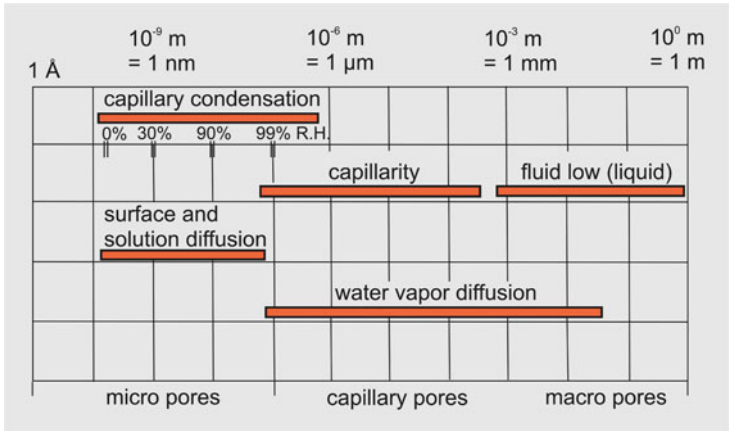


Fig. 3.14 Water transport mechanisms and the definition of pore size classes (adapted after Klopfer 1985)

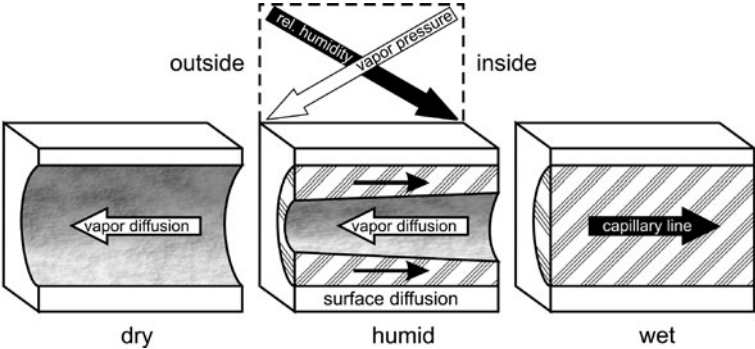


Fig. 3.15 Schematic illustration of different transport phenomena and their driving forces in porous rocks with different water content (after Holm 2001)

equilibrium tendency between a higher water vapor concentration and a lower one. With increasing moisture content in the air, i.e., relative humidity, the water vapor sorption process increases, leading to surface diffusion at the pore walls. This in turn will favor the eventual liquid capillary transport.

When all the macropores are filled with water, fluid transport will be the main mechanism for water movement in the rock. Furthermore, if hydrostatic pressure is present, such as when some centimeters high water puddle may collect over a surface so that its weight will increase the pressure on the pore system of the stone, then the capillary fluid transport changes into fluid flow processes (Darcy flow, e.g., Snethlage 1984, Klopfer 1985).

Among the different water uptake parameters determined, the water absorption by total immersion is that measured under atmospheric pressure or under vacuum conditions. From these specific values the saturation degree of a rock sample can

be obtained. Also important is the capillary water absorption, which depends on the capillarity of the sample, when only one surface is in contact with liquid water. Complementing the total water absorption and capillary water absorption are the drying mechanisms of a porous rock, which relate to the movement of the fluid water and vapor out of the pore system.

The determination of the water vapor sorption is one of the standard measurements for the characterization of building materials, including dimension stones. For the specification and understanding of the hygric properties, i.e., those related to the transport of water vapor, two aspects are important, firstly the properties of the pore space with respect to shape, cross section, surface area, and the three dimensional connectivity of the pores; and secondly, the interaction between the different phases (air, water vapor, liquid water) in the pore space.

3.3.1 Capillary Water Absorption

A porous dimension stone experiences water absorption while exposed to rain when being part of a building masonry or cladding as well as when in contact with groundwater. In general, a porous medium in contact with liquid water will absorb it by capillarity. This is a spontaneous process related to the capillary absorption force originated by the pores in the material between the diameters of 10 μm and 1 mm. It is the result of a balance between the surface tension of the liquid water and the adsorption forces of the pore wall, usually a polar mineral surface. Water at these polar surfaces exhibits a characteristic wetting angle that finally results in the tendency of the water to enter the pore system, the so-called capillary action or capillary suction. The mechanism of capillary water absorption depends mainly on the pore size and the geometry of the pore system.

As shown in Fig. 3.14, capillarity is the main driving force for water absorption by pores between 1 μm and 1 mm.

Water can move upwards against gravity (ascension) in a capillary along hydrophilic surfaces (Fig. 3.16), such as a mineral surface, whereas in the case of a hydrophobic surface the water descends in the capillary (depression). This is the case when a dimension stone has been treated with a hydrophobization agent as discussed in Sect. 7.9, or when a sample is in contact with mercury during mercury porosimetry, to determine its pore size distribution. This method, as well as others presently used for pore structure analysis, is based on idealized shape models of the pores, because the real pore geometry is too difficult to put into an exact mathematical expression.

Both the capillary suction and capillary suction rate depend on the pore diameter, with smaller capillaries having a higher suction power (Fig. 3.16a, b) but a lower capillary suction rate (e.g. Kettenacker 1930, Cammerer 1954, Klopfer 1985). Larger capillaries, on the other hand, have higher suction velocities but a much lower suction power. For a natural pore system with capillaries of different sizes, this will result in a generally faster capillary water absorption by larger pores and slower absorption by smaller pores. If the water transport is disrupted or broken

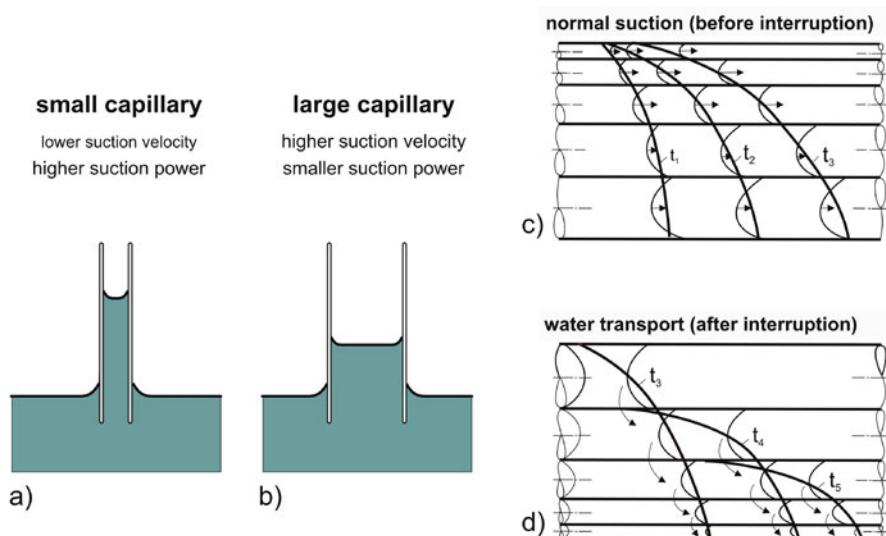


Fig. 3.16 Rise of the water level for capillaries of different diameter (**a**, **b**). The diameter has an influence on the suction behavior and the suction power in general, for an uninterrupted suction processes (**c**), and specifically for an interrupted suction process (**d**). The water front is presented in relation to the diameter of the capillary at different times, t_1 , t_2 , and t_3 . (**d**) Rearrangement of the water transport from larger to smaller pores after disruption of the water supply, also over different times, t_3 , t_4 , and t_5 (after Krus 1995)

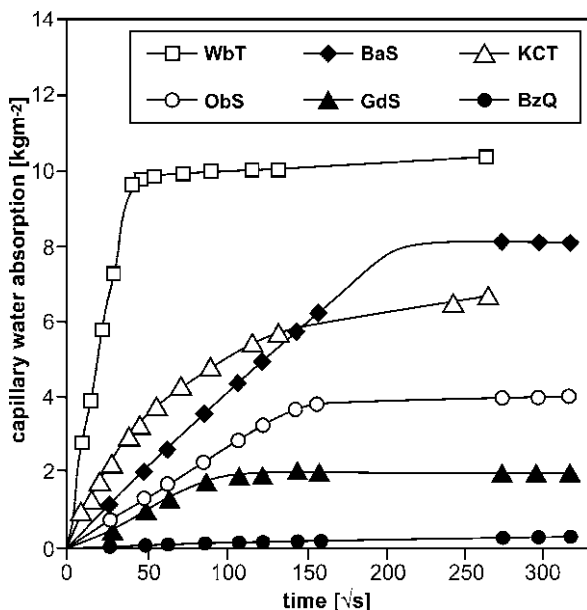
at one point a rearrangement of the processes involved occurs. Smaller pores will then increasingly absorb water from the larger capillary pores due to their higher suction power. This is schematically illustrated in Fig. 3.16c, d for capillary pores with increasing diameter. The water front is presented in relation to the diameter of the capillary over different times, t_1 , t_2 , and t_3 . Figure 3.16d shows the rearrangement of the water transport from larger to smaller pores after disruption of the water supply, also over different times, t_3 , t_4 , and t_5 .

The capillary water absorption of porous materials can be described through the water absorption coefficient (w-value). In the laboratory this can be determined using a cylindrical or cubic stone specimen placed with only its bottom side in a basin filled with water. The cube for this test usually has a 5 cm side length. Care has to be taken that the water absorption occurs only and constantly through the bottom side. For this purpose, the basin with the water has to be large enough so that no significant change in the water level occurs during the absorption experiment. The relationship of the capillary water absorption versus time is determined by periodically measuring the change in weight. The w-value is determined by:

$$\text{w-value [kg/(m}^2 \text{ h}^{0.5}\text{)]}: w = m_w / \sqrt{t} \quad (3.7)$$

where m_w is the surface-related water absorption (kg/m^2) and t is the absorption time (h).

Fig. 3.17 Capillary water absorption of some selected rock samples, Wbt - Weibner Tuff, BaS - Baumberger "Sandstone", a clastic limestone, ObS - Obernkirchner Sandstone, GdS - Grödener Sandstone, KCT - Kufstein Calc-Tufa, and BzQ - Bozen Quartz Porphyry. The capillary water absorption is given in kg/m^2 versus the square root of the time in seconds (after Franzen and Mirwald 2004)



The water absorption per unit area versus the square root of the time initially shows a linear behavior. The slope of this line corresponds to the w -value, the amount of water per unit area and the square root of time taken by capillary absorption into a sample (Fig. 3.17). This relationship only holds for some hours (depending on the rock type) or for dimension stones having good absorption properties until complete penetration (Klopfer 1974, Kraus 1985). According to Meng (1993), three conditions have to be fulfilled for the validity of the t -relationship: (1) The evaporation has to be low compared to the amount of water absorbed through the absorption process; (2) The material should not have very large pores (over 1 mm in diameter), i.e., where no capillary absorption occurs, nor very small ones (below 1 μm diameter) because the overall absorption rate is low; (3) The t -relationship only holds for vertical water absorption as long as it is below the theoretical maximum height.

On the other hand, the water permeation coefficient (b -value) can be calculated by using the following equation:

$$b\text{-value } [\text{m/h}^{0.5}] : b = z/\sqrt{t} \quad (3.8)$$

where z is the rise in height and t is the absorption time (h).

The b -value describes the penetration velocity of the water front through a material during the absorption process. Additionally, by dividing w/b the water capacity can be determined. Water capacity is generally defined as the water content per unit volume, which a rock is capable of absorbing by capillary action (Niesel and Schimmelwitz 1982, Weiss 1992).

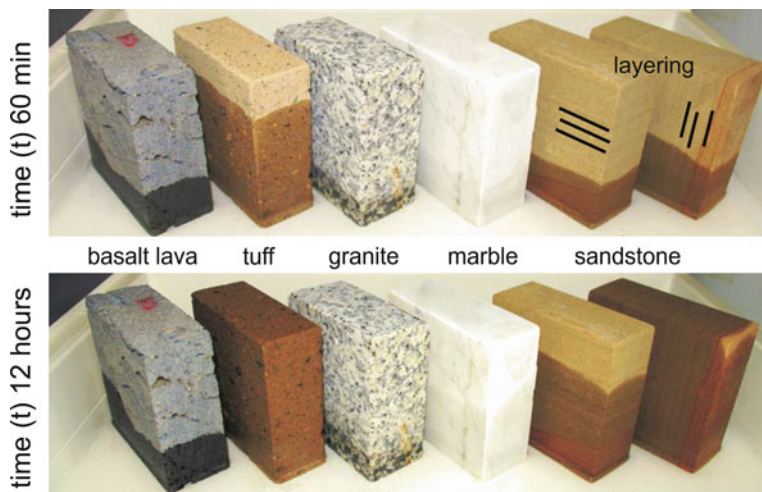


Fig. 3.18 Photos of the capillary water absorption of different rocks after 60 min (t60min) and twelve hours (t12hours). For the sandstone sample, two specimens were used, one with the sedimentary layering parallel and the other perpendicular to the water level

The determination of the capillary water absorption at an outdoor object is usually carried out using the “Karsten” tube (see [Chap. 6](#)).

As an example, the capillary water absorption of selected rocks presented in [Fig. 3.17](#) (see also [Fig. 3.18](#)), shows the typical trend that can be separated in two parts. The first part shows a linear increase of the capillary water absorption ([Kraus 1985](#)). In the second part the curve exhibits a plateau-style shape with the absorption value only slightly increasing with increasing t . The t -relationship can only be used as a first good approximation to describe the time-dependent trend of the initial capillary water absorption. From a large number of experiments, it is known that for many rock types a linear relationship of the capillary water absorption versus the square root of the time is valid. Using the w -value ([Snethlage 2005](#)) rocks can be classified into slightly absorbing ($w < 0.5 \text{ kg/m}^2\text{h}$), medium absorbing ($w < 0.5\text{--}3.0 \text{ kg/m}^2\text{h}$), and highly absorbing ($w > 3.0 \text{ kg/m}^2\text{h}$).

For a granite sample [Poschlod \(1990\)](#) reported a w -value of $0.41\text{--}0.03 \text{ kg/m}^2\text{h}$. A value of $0.41 \text{ kg/m}^2\text{h}$ means that during the first hour the rock absorbed 0.41 kg water per square meter, or 0.041 ml water per square centimeter, accordingly. Marbles samples, like Carrara or Laaser Marble, have w -values of about $0.10 \text{ kg/m}^2\text{h}$. For the Globigerina limestone from Malta, [Ruederich et al. \(2005a\)](#) determined w -values in the range of $7.78\text{--}14.96 \text{ kg/m}^2\text{h}$. [Szilagyi \(1995\)](#) reported for several often used dimensional building stones from Germany (sandstones and limestone) w -values between $0.1 \text{ kg/m}^2\text{h}$ and $9.4 \text{ kg/m}^2\text{h}$, which are in accordance with values determined on samples from the Cologne Cathedral ([Kraus 1985](#)). In the literature w -values of up to $70 \text{ kg/m}^2\text{h}$ can be found.

Capillary absorption also reportedly exhibits a strong directional dependency, which can be significant ([Poschlod 1990](#), [Weiss 1992](#), [Ruedrich et al. 2005a, 2007](#),

2010a). Anisotropy values of more than 70% are found quite often, and it can be assumed that the lowest capillary water absorption values are measured perpendicular to the sedimentary layering or metamorphic foliation. The b-value is also a critical number; especially when substitution materials have to be identified.

3.3.2 Water Absorption by Total Immersion

The water absorption is the difference between the weight of a sample totally immersed in water and its dry weight. In contrast to the capillary water absorption discussed above, the value of the total water absorption can be determined by different methods (Fig. 3.19). The usual methods are the total immersion under atmospheric pressure and under vacuum conditions, although the DIN-standard (DIN EN 1097–6 2005) recommends measuring the water absorption at 150 bar.

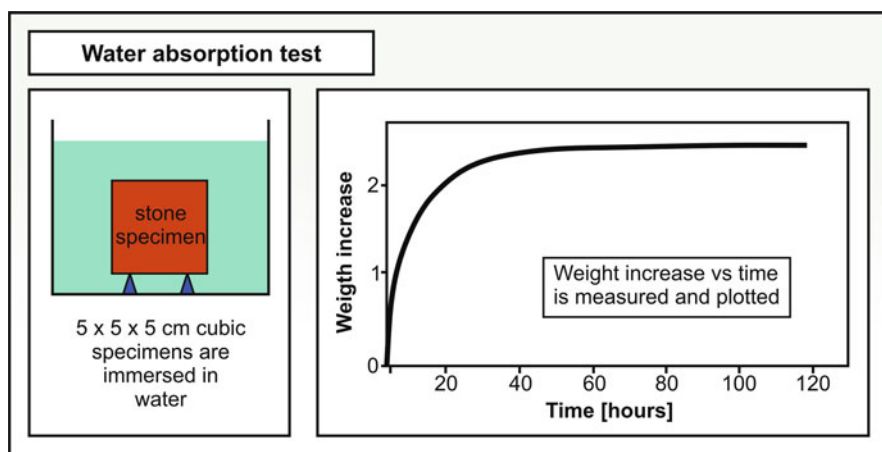


Fig. 3.19 Schematic illustration of the water absorption measurement with the sample totally immersed in water and a schematic curve of the sample's weight increase over time due to the absorption of water until a nearly weight constancy is reached (after I-Stone 2008)

The water absorption ability of a rock is closely related to its porosity. In general, the higher the value of the effective porosity the higher is the total water absorption. However, the total water absorption ability of a rock is not related to its total porosity, but to the percentage of pores accessible for the water under normal conditions.

The total water absorption value under atmospheric pressure conditions indicates how much water a rock can absorb over 24 hours when placed 3–5 cm below the water level. The subsequent weighting of the sample provides the wet weight (m_f , in g) and the following equation the relationship to the water absorption (W_{atm} in weight-%), where m_d (in g) is the dry weight of the sample:

$$W_{\text{atm}} = \frac{m_w - m_d}{m_d} \cdot 100 \text{ [weight-\%]} \quad (3.9)$$

with m_w the wet weight at $t = 48$ h (in g). However, the water absorption W_{atm} only provides a relative value that is only valid for the given experimental conditions (Chitsazian 1985).

The water uptake in relation to the dry weight of the sample is mainly influenced by porosity, pore size distribution and the mineralogical composition of the rock. Due to the dominance of quartz and feldspar the mineral density of sandstones is about 2.64 g/cm^3 , whereas calcitic limestone rocks have a matrix density of around 2.72 g/cm^3 . From that a cubic sandstone specimen with a 10 cm edge length and 20% porosity would show a weight of 2112 g in the dry state and 2212 g with 50% water saturation of the pore space. The related water absorption would be 4.73 weight-%. For a limestone specimen with the same dimension the dry weight would be 2176 g and 2276 g with 50% water saturation of the pore space, with the related water absorption to be 4.60 weight-%. Although both samples have the same water saturation value the water absorption varies. This shows the influence of the porosity in relation to the mineral density. For a porosity of 30% the water absorption of the sandstone is 8.12 and for the limestone 7.88. These values show that with increasing porosity the differences in the water absorption become more pronounced.

Furthermore, the pore size (see Sect. 3.2) has an influence on the total water absorption. A shorter immersion time over hours or days mainly fills the capillary pores to a large extent. Relatively small pores, which are widely found in sandstones, are filled only after a very long immersion time. On the other hand, cavernous mesopores, which are often found in limestone, will drain after taking the specimen out of the water.

Both phenomena are likely to have an influence on the determination of the total water absorption, either related to the volume of the sample, or its porosity or to the weight of the sample. Poschlod (1990) compares the total water absorption values of two granite and two marble samples, all measured under atmospheric pressure and water absorption determined in relation to weight-% and vol.-%; both values obviously differ in their numbers (which is finally due to the effect of the density): Kösseine granite 0.21 weight-% vs. 0.56 vol.-%; Nammering granite 0.76 weight-% vs. 1.97 vol.-%; Carrara marble 0.18 weight-% vs. 0.50 vol.-%; and Laas marble 0.15 weight-% vs. 0.40 vol.-%.

Fitzner (1970) showed that there is a relationship between the water absorption and the size and shape of the sample used in the experiment. The influence of the time period a sample was placed under water on the value of W_{atm} was discussed by Weiss (1992). He showed the saturation degree of the pores over the course of time in a logarithmic scale for several sandstones during the full submersion of the samples (see Fig. 3.19). Following Fitzner (1988) the resulting graphs show characteristic distributions depending on the pore radii distribution of the sample (see Sect. 3.2). If a sample is placed under water long enough, a “full” or 100% saturation of the accessible pore space can be assumed, as it was demonstrated in a very long, 800 days experiment by Fitzner (1970).

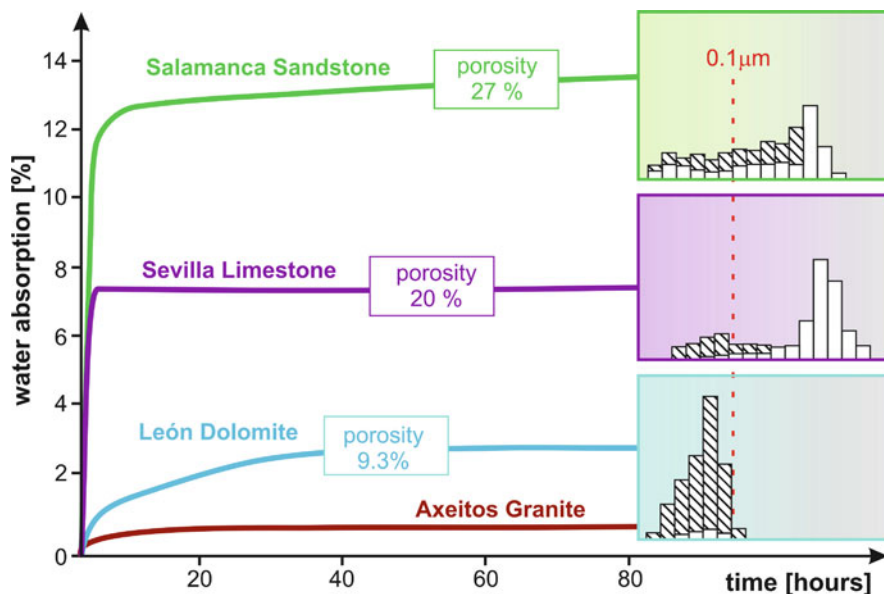


Fig. 3.20 Water absorption versus time for four different rock types from Spain. The pore size distribution is also schematically given (after I-Stone 2008)

The rocks shown in Fig. 3.20 comprise a granite and three sedimentary rocks (sandstone, limestone, and dolomite). They all show very significant differences in their pore space and, consequently, in their water absorption behavior. Three Hg-porosimetry histograms, total open porosity, and water absorption values are also included in the diagram. The sandstone shows initially over a short period of time a very fast water absorption, which is followed by a longer period with far slower absorption. This can be explained by the information provided by its pore size distribution, i.e., the range of pore sizes that allows water to access the open porosity (27%). The pore size distribution is very wide, from less than $0.01\ \mu\text{m}$ to almost $100\ \mu\text{m}$. In addition, the highest percentage of its total effective porosity is refilled through very wide conduits ($1\text{--}100\ \mu\text{m}$). Another small but significant percentage of the pore space is refilled through very narrow conduits (less than $0.01\text{--}0.1\ \mu\text{m}$). This explains the initial fast and easy water filling, reaching almost its total water absorption capacity, and the slow and difficult second period. This stone also exhibits swelling clays, smectites, located in intrapore and in intragranular positions that hinder water movement through the very narrow conduits. The limestone shows an expected behavior in this diagram as its pore space is characterized by very wide conduits ($1\text{--}100\ \mu\text{m}$), which give access to almost its total effective porosity (20%). Consequently, water absorption and water desorption are extremely easy and fast with an almost initial period that can be monitored. The behavior of the dolomite is quite interesting. Its total open porosity is 9.3%, but its increment in weight due to water absorption is only about 3%. As the Hg-histogram shows, only a very small fraction of the open porosity is effective and this small fraction is accessed only through small conduits in the $0.01\text{--}0.1\ \mu\text{m}$ diameter range. All these observations

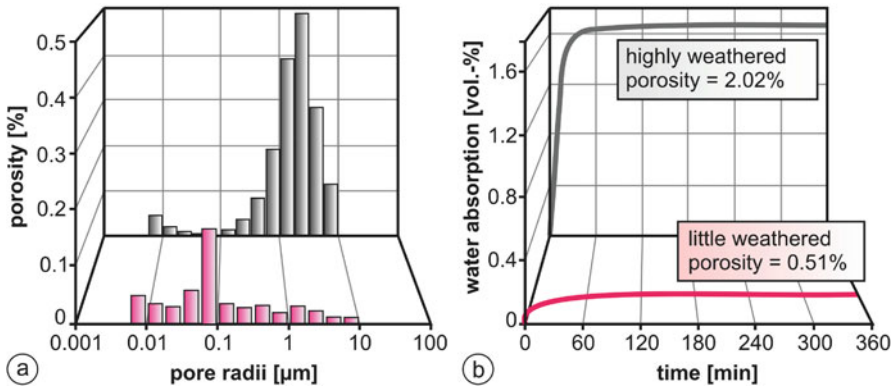


Fig. 3.21 (a) Porosity and pore size distribution for fresh and weathered Carrara Marble in comparison to the (b) water absorption for the same samples

explain the long period with slow water absorption capacity of this sample. Water movements in crystalline stones are in general difficult, as the granite in Fig. 3.20 shows. Water movement into the pores of this low porosity sample is entirely channeled through interconnected fissures and microcracks. Figure 3.21 shows the effect of the porosity for a quarry-fresh and strongly weathered marble sample on the water absorption. It can be clearly seen that with increasing weathering the pore radii distribution is moving towards larger pore radii. The weathered marble sample shows a spontaneous water absorption in the first minutes and after that a longer period of water absorption, which does not change with increasing time. As mentioned earlier, the sample has to be placed in the water for 24 or 48 hours.

For an evaluation of the water absorption Mosch and Siegesmund (2007) and Mosch (2009) compiled data from 1,650 dimension stones (Fig. 3.1, Tables 3.3 and 3.4). As shown in Fig. 3.22, there is a clear relationship between the water absorption, the bulk density, and the effective porosity, especially for the group of more porous rocks, like sandstones, carbonate rocks, and volcanic rocks. Despite some outliers the highest values for the water absorption can be found in the tuffs, which can be related to the relatively low compaction of these rocks during their pyroclastic deposition. This resulted in relatively high porosity values combined with an effective interconnectivity between the open pores. However, the porosity/ W_{atm} data pairs overall show a relatively large variation, which is the result of the heterogeneous character of the pore space (see Sect. 3.2). Here, the very large pores do not contribute to the water absorption ability of these rocks.

For the low porous rocks a linear relationship between the water absorption, the porosity, and the bulk density can be established. For plutonic and metamorphic rocks the correlation coefficient between the water absorption and the porosity is $R = 0.85$, whereas the correlation to the bulk density is only $R = -0.15$. The carbonate rock group exhibits in some way a similar picture. Regarding the water absorption, the carbonate rocks with a bulk density $\rho_{\text{bulk}} \geq 2.6 \text{ g/cm}^3$ are comparable to the plutonic and metamorphic rocks. With decreasing bulk density

Table 3.4 General values for water absorption

Group	Water absorption [weight-%]		
	Min	Max	Median
Plutonic rocks	0.01	0.60	0.23
Metamorphic rocks	0.01	0.57	0.18
Marbles	0.01	0.38	0.15
Gneisses	0.05	0.54	0.26
Carbonate rocks (all)	0.01	3.75	0.62
Carbonate rocks $\rho \geq 2.6 \text{ g/cm}^3$	0.01	1.75	0.30
Carbonate rocks $\rho < 2.6 \text{ g/cm}^3$	0.05	10.00	2.21
Sandstones (all)	0.04	10.60	5.11
Porosity class I ($\Phi < 13.4\%$)	0.07	7.08	4.85
Porosity class II ($\Phi < 20.5\%$)	2.64	9.32	5.25
Porosity class III ($\Phi \geq 20.5\%$)	5.12	8.65	7.04
Volcanic rocks (all)	0.05	21.9	3.51
Tuff	2.03	32.20	13.20
Volcanic rocks (without tuff)	0.05	4.40	1.67

The maximum and minimum values are the upper and lower extremes from the box plot data analysis, and therefore do not give the outlier values (after Mosch and Siegesmund 2007, Mosch 2009)

and increasing porosity the values and the distribution of the data pairs are more similar to the sandstone varieties.

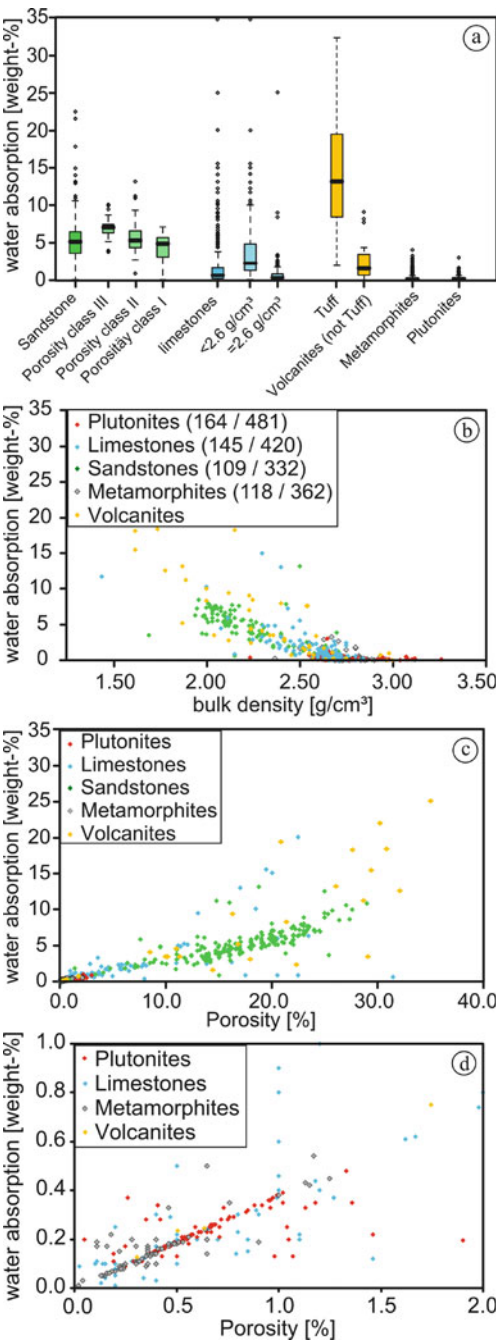
As shown before, the absolute values of the water absorption are closely correlated to the porosity of a rock. However, the data analysis of the sandstone varieties with respect to the porosity classes (as defined in Sect. 3.2) shows that the porosity is not the only factor that has an influence on water absorption. Though, the median value of the water absorption from the first (porosity $<13.4\%$) to the third porosity class (porosity $\geq 20.5\%$) increases, the central segments of the classes show significant overlaps (Fig. 3.22a).

Therefore, the porosity can only be used for a first evaluation of the water absorption ability of a rock. For rocks with a similar porosity value the pore radii distribution and the pore shape are become increasingly important parameters for water absorption, especially for sandstones, as these parameters are primarily related to the grain and particle size as well as to its sorting. Secondary factors are cementation and related diagenetic processes (see Chap. 2). For plutonic and metamorphic rocks, basically crystalline rocks, the porosity is mainly due to microcracks, either along grain boundaries, or as inter- and intracrystalline cracks (see Fig. 3.5f, g). For the capillary water absorption the important parameters are the state of the grain boundaries, as well as the width and interconnectivity of the existing microcracks.

A dataset of 188 sandstones from Germany is shown exemplarily in Fig. 3.23 to demonstrate the influence of two factors, grain size and sorting, on water absorption. Other factors that might have an influence are not taken into consideration here, e.g., the cement or the sphericity, or roundness, of the grains.

The rocks are primarily separated according to their grain size (fine-grained sandstones, fS: $<0.2 \text{ mm}$; coarse-grained sandstone, mS: $0.2\text{--}0.63 \text{ mm}$) and their

Fig. 3.22 (a) Water absorption for different rock types and some differentiation with respect to bulk density and porosity. Correlation between water absorption and bulk density (b) and porosity (c). (d) A detailed view of c for porosity values $\Phi < 2\%$ (after Mosch and Siegesmund 2007)



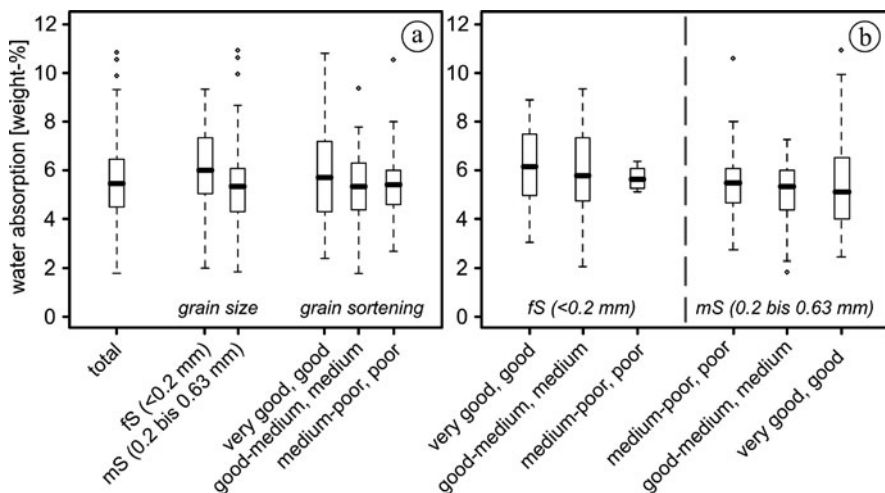


Fig. 3.23 Data analysis of 188 sandstones from Germany for a correlation between the water absorption and grain size and grain sorting (data after Grimm 1990, Wenzel and Häfner 2003, David 2006). The small range for the grain size group “fs”/moderate-poor, poor (in b) resulted from the small number of samples, $n = 4$ (after Mosch and Siegesmund 2007, Mosch 2009)

grain sorting (very good, good, good-moderate, moderate, moderate-poor, poor; see Fig. 3.23). Although it is generally assumed that a smaller mean grain size results in a lower porosity (see Chap. 2 and Sect. 3.2), the data here show that the median value of the absorption for rocks with a smaller grain size ($W_{\text{atm}} = 6.00$ weight-%) is slightly higher than for rocks with a larger grain size ($W_{\text{atm}} = 5.29$ weight-%). The data suggest that grain sorting has no direct effect on the rock’s ability towards capillary water absorption. However, the two datasets separated by grain size show some intrinsic trends. The water absorption of fine-grained sandstones (fs) increases with increasing grain size, or better sorting; whereas for coarser grained sandstones (ms) the same correlation shows a decreasing trend.

Therefore, the lowest median value of the water absorption can be expected for medium-grained sandstones with good to very good sorting, and the highest median value for fine-grained sandstones with very good to good sorting.

Water absorption is a relatively slow process resulting in the gradual filling of the remaining pore space based on capillary condensation mechanisms (Künzel and Krus 1995, Mirwald 1997). In the laboratory this state can be nearly achieved by evacuating the air from the effective pore space first until a vacuum is reached and then afterwards immersing the sample under water to saturate the pores. This process creates a significant absorption force or effect. However, Fitzner (1970) has shown that a really full saturation with nearly constant values can be achieved only after very long experiments with durations of up to two years.

The calculation of the forced water absorption is as follows:

$$W_{\text{vac}} = \frac{m_n - m_t}{m_t} \times 100 \quad (3.10)$$

with W_{vac} the water absorption under vacuum (weight-%), m_n the mass of the sample after forced water absorption (g), and m_t the dry mass of the sample (g).

For the evaluation of the frost resistance of a dimension stone, the quotient of the freely or unforced water absorption (W_{atm}) and the forced water absorption (W_{vac}), the so-called saturation degree or saturation coefficient S (Hirschwald 1912) is of importance. The S -value is calculated by the following equation:

$$S = \frac{W_{\text{atm}}}{W_{\text{vac}}} \quad (3.11)$$

The saturation coefficient describes how much of the total pore space is accessible to water absorption, thus providing a value for the frost resistance evaluation (Table 3.3). If the saturation coefficient is smaller than 0.9 or less than 90% of the pore space, the pores can be filled by capillary absorption and then the rock should be frost resistant as the volume increase related to the freezing of the water is 9%. For the frost resistance Hirschwald (1912) proposed the following guideline values using the saturation coefficient: when $S < 0.75$ the rock is weathering and frost resistant; for values ranging between 0.75 and 0.90 it is uncertain and further investigations are necessary; and, when $S > 0.90$ the rock is not weathering and frost resistant. Hirschwald (1912) himself favored a critical value of $S = 0.8$, with some exceptions, for a rock to be frost resistant. In the case of ice formation in a highly saturated pore space the remaining pore space can hardly compensate the volume expansion (Hörenbaum 2005). As the consequence, microcracks along the grain boundaries will be formed leading to a possibly significant damage of the rock and subsequently a decrease in the mechanical stability (e.g., Ondrasina et al. 2002, Ruedrich et al. 2010b).

However, for plutonic and metamorphic rocks as well as for carbonate rocks with a bulk density above 2.6 g/cm^3 it is assumed that the saturation coefficient has a limited validity and significance. With these rock types very small values of W_{atm} and W_{vac} can sometimes be found, resulting in relatively high values of S as the quotient of both. In most cases this is related to the measurement procedure and the result of the resolution limit are also associated with that. For example, the saturation coefficient of the Lichtenberg diorite, a medium-grained Bt-Hbl-diorite, is $S = 0.94$ (Grimm 1990), however its frost resistance should be certain because of the extremely small total porosity.

Sandstones and porous limestone show a higher variability of the absorption under atmospheric pressure conditions than of the forced water saturation at comparable porosity values. It becomes evident that the effectiveness of the atmospheric or free water capillary absorption depends on certain pore radii. Here, the variability in the pore size for basalt lava (Chap. 2) can be shown. With porosity values of 22.31% and 29.06% and atmospheric water absorption values of 2.36 weight-% and 3.50 weight-% the saturation coefficient give values of 0.24 and 0.25 (Grimm 1990). For a possible correlation between the S -value and the porosity a relative increase of S in the porosity range between 10% and 30% can hardly be seen (Fig. 3.24). However, a general trend based on this data set cannot be determined.

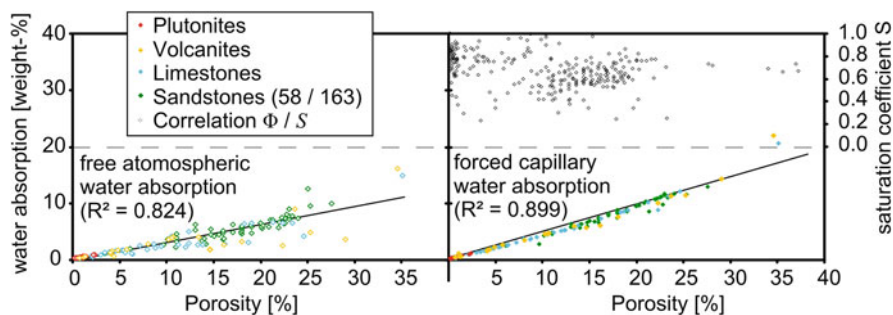


Fig. 3.24 Group specific point distribution of the correlation between the free atmospheric (*left*) and the forced capillary water absorption (*right*) as well as the saturation coefficient S . The R^2 for each linear equation is given (after Mosch and Siegesmund 2007, Mosch 2009)

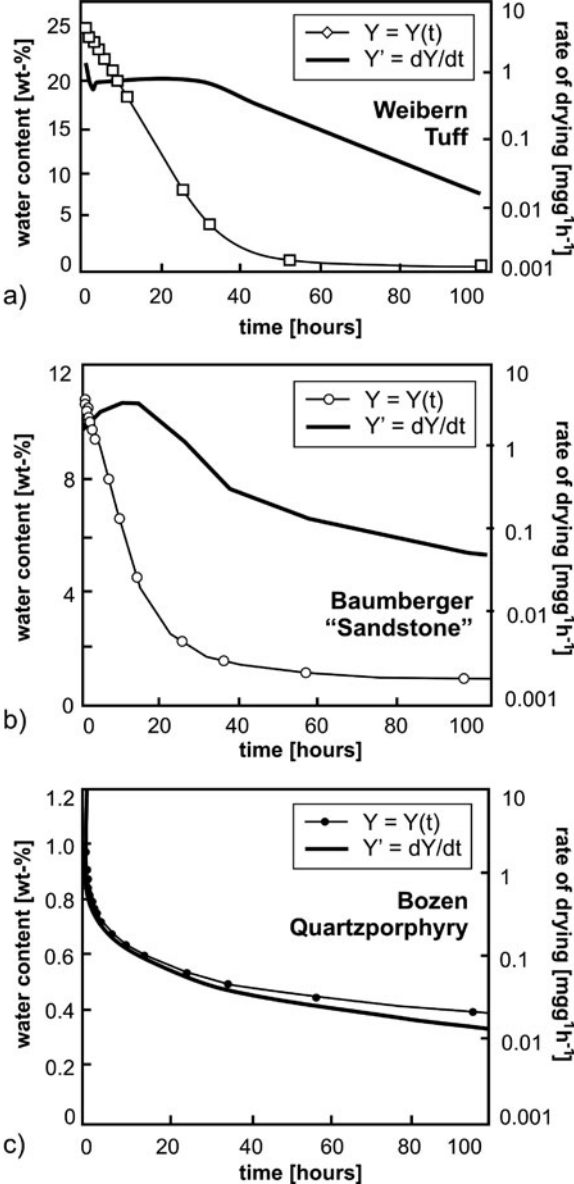
3.3.3 Drying Characteristics

The drying characteristic of a dimension stone is a very important factor that clearly controls the weathering resistance. Drying of wet building stones is a multistage process that is mainly governed by air humidity and movement. The faster and the more thoroughly a dimension stone is dried the lower is the possibility that the stone will experience frost or salt damage. The theory of drying of porous building materials is described by Vos (1978), Snethlage (1984), Klopfer (1985), Kraus (1985), and Snethlage et al. (1986) and examples are given in Fig. 3.25. In general, two phases can be distinguished; the first phase is characterized by a rapid decrease in moisture content, which shows a nearly linear trend. This drying path is defined as the capillary moisture transport from the inside of the sample towards the sample surface. After reaching the “critical moisture content” (Vos 1978), the water surface retreats back into the rock sample. During the second phase of the drying process, the capillary moisture transport is completely replaced and controlled by water vapor conductivity processes, which are related to evaporation mechanisms at the inner surfaces of the pores. The drying process stops when the moisture content in the rock sample and the adjacent air are in equilibrium.

Snethlage (1984) and Tournier et al. (2000) have defined three stages of drying, where Stage 2 and 3 corresponds to a level where the rate first decreases and then levels out at a small evaporation rate, i.e., after evaporation at a high drying rate. Franzen and Mirwald (2004) have subdivided Stage 1 into Stage 1a and Stage 1b: 1a corresponds to a sharp decrease in the drying rate, while 1b shows little change and eventually shows a tendency to increase. Stage 1b coincides with the almost linear decrease in the water content (see Snethlage 1984, Weiss 1992, Tournier et al. 2000).

Figure 3.25 shows the drying curves for different rock types: the Weiberner tuff, Baumberg “sandstone”, a clastic limestone, and Bozner quartz porphyry (see Franzen and Mirwald 2004; for the Baumberger “sandstone” also refer to Kraus 1985). Two curves are shown for each sample, the decrease in water content over

Fig. 3.25 Drying curves for the (a) Weiberner Tuff, (b) Baumberger “Sandstone” and (c) Bozner Quartz Porphyry. Thin lines with symbols represent water content vs. time; thick lines are the drying rates (note log-scale in right y-axis) (after Franzen and Mirwald 2004)



time and the drying rate, which is the first derivative of the water content with respect to time. All samples follow the three steps described above. The critical moisture content for different rock types will be reached after different time periods. The Baumberger “sandstone” reaches the point after approximately 14 hours, the Weiberner tuff after about 30 hours. The increase in the drying rate for the

Baumberger “sandstone” is quite pronounced, indicating a large active evaporating surface, which is related to the surface roughness of the pore system, expressed by the specific inner surface value. The Weiberner tuff comprises mainly of pores with smaller sizes, likely to be responsible for the almost constant behavior at Stage 1b. The Bozener quartz porphyry shows a continuous decrease in the drying rate and water content and reaches a water content after 100 hours that is still 40% of the saturation (see discussion in Franzen and Mirwald 2004).

The drying characteristics are also controlled by the temperature. The above shown samples were dried in an oven at 58°C, because Krantz (1983) has shown that the critical temperature for granites, for example, is at 74°C, where thermally induced microcracks start to form. For marbles the critical temperature for crack development is much lower as shown by Battaglia et al. (1993) and Siegesmund et al. (2000a). Vietor (1993) had shown that the drying rate of an amphibolite can be very slow, and even after 1.5 months the sample still continues to lose water, although at a very slow rate. An increase in the drying temperature to 90°C did not substantially increase the drying rate. The fact that water remains trapped in the rocks over very long time periods is difficult to explain.

3.3.4 Moisture Adsorption

Porous media with their specific surface area are in constant interaction with moisture from the atmosphere surrounding them as exemplified by the sorption of water molecules at their surface. At the hygroscopic level the water adsorption of a rock is regulated by the moisture content of the air or humidity (Fig. 3.26) and it is separated into sorption (moisture adsorption) and desorption (moisture release). Here

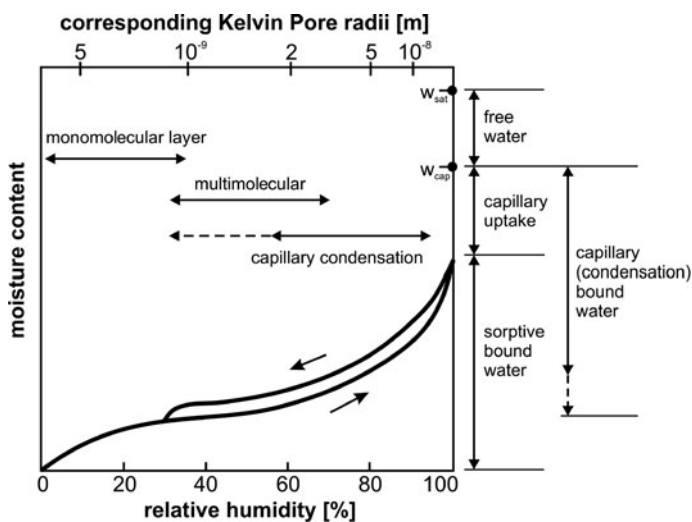


Fig. 3.26 Typical sorption isotherms of materials with small pores (after Kießl 1983)

the water molecules attach or detach themselves to or from the surface of the pore walls through the weak interactions of hydrogen bonds and van der Waals forces. At a given air humidity, a dynamic balance is established between the sorption and desorption of the water molecules, that is the sorption equilibrium, which thermodynamically is a function of the water vapor partial pressure, or the relative humidity of the air at constant temperature. The sorption moisture or hygroscopic range extends to equilibrium moisture content values of 95% relative humidity.

The determination of the equilibrium moisture content at a given relative humidity, RH, since the processes involved are temperature dependent, is the basis for determining the adsorption isotherms, which are characteristic and specific for each stone type. From the sorption and desorption isotherm determined over the whole humidity range, essential conclusions of the material properties of a stone can be drawn. The sorption isotherm describes the amount of water adsorbed in relation to the relative humidity of the surrounding air, while the desorption isotherm describes the water released as RH decreases.

Both isotherms can exhibit a hysteresis, which for most stones is not pronounced. Therefore, often it is enough to determine only the sorption isotherm (Künzel 1994). Garrecht (1992) describes as one cause of the hysteresis the presence of so-called bottleneck pores, large pores connected to a relatively small open pore for entry and access. Here, the partial pressure of the smaller pore determines the moisture condition in the larger pore. The larger pore can only release the water once the partial pressure conditions also allow the small pore to dry.

The often seen S-shape of the sorption isotherms (Fig. 3.27) reflects different steps in sorption or desorption. For the sorption isotherm, for example, initially one monomolecular layer of water molecules is adsorbed at the pore surface. With increasing relative air humidity further layers are added resulting in multimolecular-layer adsorption. At relative air humidity levels >85%, the sorption isotherm increases significantly due to beginning capillary condensation resulting in a gradual filling of the pores (Krus 1995). In the pores of <100 nm (10^{-7} m) the occurrence of capillary condensation results in a more or less full water saturation. The beginning of the capillary condensation varies, starting from 30% RH to 50% RH for most of the porous stones. At 95% RH the sorption isotherm exhibits a very strong increase (see Fig. 3.26). For this upper range no measurement methods are known for accurately determining the relative humidity in air humidity. It is the beginning of the capillary water range beyond the hygroscopic level (Krus 1995).

The sorption isotherms can be easily determined in a desiccator. The samples are placed at constant room temperature under different relative air humidity levels established by using different saturated salt solutions as listed in Table 3.5. Depending on their size, the samples need about one month to reach equilibrium between the relative air humidity levels inside the pores and the surrounding air in the desiccator. However, for some stones it can take up to several weeks or months to reach equilibrium, especially for the determination of the desorption isotherm (Meng 1993). Reaching a stable value for the relative air humidity in the desiccator is not enough to assume that equilibrium has been reached (Metz 1992). In general, it is quite difficult to determine the complete water vapor sorption isotherms for

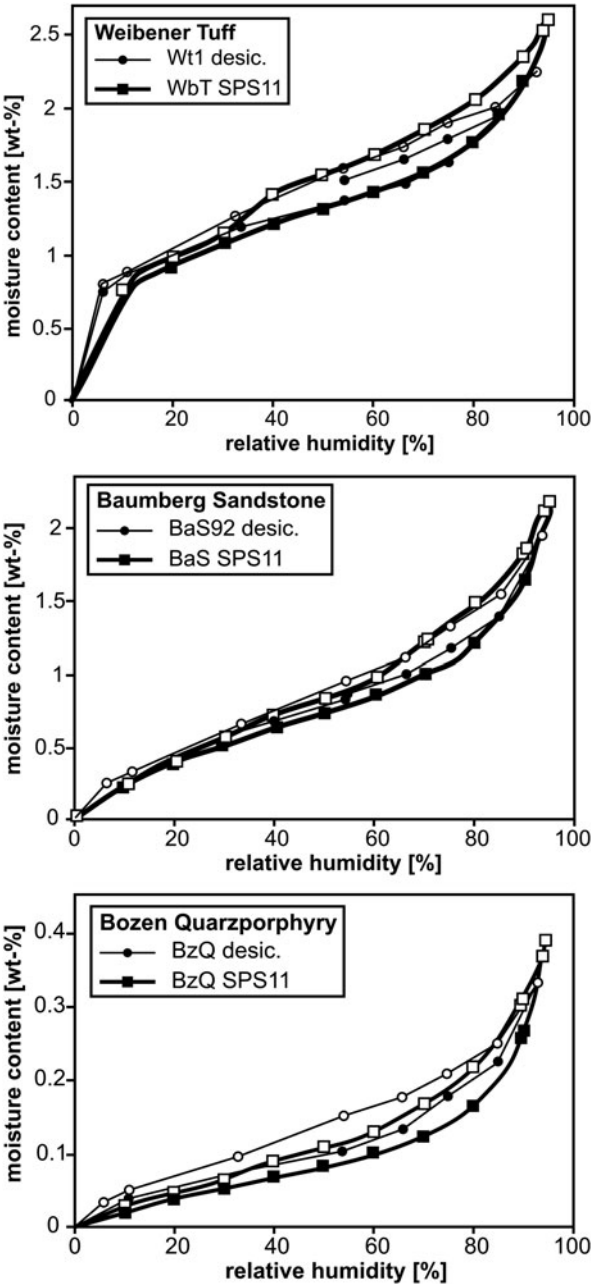


Fig. 3.27 Equilibrium moisture sorption isotherms. The filled symbols mark the sorption, and the open symbols the desorption data (desiccator method and SPS11 data determined by SPS 11), after Franzen and Mirwald (2004)

Table 3.5 Saturated salt solutions used for establishing constant relative humidities, their molecular weights and the relative humidity (RH) they generate in a closed environment (after Meng 1993)

Salt	Mol weight [g]	RH at 21°C [%]
NaOH	40	6
LiCl	42	11
MgCl ₂ ·6 H ₂ O	202	33
Mg(NO ₃) ₂	148	54
NaNO ₂	69	75
NaCl	58	75
KCl	75	85
KNO ₃	102	93
K ₂ SO ₄	176	97

stones, because of their heterogeneous composition and their often quite complex pore space geometry.

Franzen and Mirwald (2004) applied a new automatically working apparatus for the determination of the sorption isotherms, which may help to overcome the time-consuming desiccator method (see Griesser and Dillenz 2002). The data from Franzen and Mirwald (2004) further show that the hysteresis effect significantly starts to develop at around 30% relative air humidity; the level where the monomolecular layer of the initial adsorption is assumed to be completed. All sorption curves in the study exhibit a smooth course, whereas some desorption curves show a more non-continuous behavior near the 30% relative humidity level. All the samples showed hysteresis. For all the samples with a non-continuous behavior during desorption (Weiberner tuff, Baumberger “sandstone”, and Bozener quartz porphyry), the pore radii distribution mainly comprises pores smaller than 5 μm, whereas the samples with smooth desorption curves have a pore spectrum reaching into the range of macropores. This underlines the importance of the pore size distribution for the sorption and desorption of porous dimension stones.

3.3.5 Water Vapor Diffusion

The gas permeability of a rock is an important property for the drying characteristics of a wet pore surface. At the beginning of the drying process of a completely wet or soaked material capillarity plays an important role. Further drying, is characterized and determined by water vapor diffusion until the equilibrium moisture content, which depends on the environmental conditions at the site, is reached. Therefore, water vapor diffusion is of great importance for the durability of dimension stone (Schuh 1987). The diffusion transport of water vapor from the surrounding air through a porous material is controlled by two types of diffusion: (1) water vapor diffusion, based on the motion of water molecules in the gas phase through the pore space; (2) Surface diffusion, the fluid transport in the sorbate layer of the hygroscopic porous material. The sorbate layer on two sides of a same pore shows

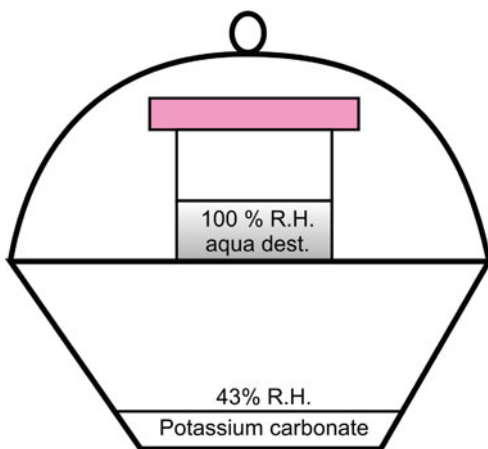
different thicknesses due to differences in the relative humidity. When a water molecule enters the thicker side of the sorbate layer than on the thinner side, another water molecule leaves the sorbate layer immediately. This transport is faster than through the gas phase (Klopfer 1985).

The dimensionless number of water vapor diffusion resistance (μ -value) provides information about the diffusion resistance of a porous material towards the moisture of the adjacent air. Besides the capillary water absorption, the μ -value is the most important parameter, which is used in the evaluation of the effectiveness of the treatment or retreatment of rocks with hydrophobic agents. The μ -value is calculated from the quotient between the actual diffusion coefficient of a material and the diffusion coefficient of a static air layer of the same thickness at the same temperature. The number of water vapor diffusion resistance therefore provides a direct numerical value of how much higher the diffusion resistance of a building material is, in comparison to a static air layer of the same thickness. The μ -value of air is 1. The water vapor diffusion can be determined according to the DIN EN 12572 (2009) using the dry cup or wet cup method.

The experimental setup for the wet-cup method is shown in Fig. 3.28. A stone disk of 10 mm thickness is placed on top of a Teflon crucible filled with demineralized water and stored at 20°C (see Fig. 3.28). Over a period of 14 days this experimental unit is placed in a desiccator where the relative humidity is set at 43% by a K_2CO_3 saturated solution. The measurements are carried out at constant temperature. The water vapor transport follows the concentration gradient through the core disk outwards (wet-cup method). Changes in the weight are determined at regular time intervals. After steady state diffusion flow is reached, the weight change in the glass is proportional to time. From the linear part of the weight change the water vapor diffusion resistance factor can be determined by the following equation:

$$\mu = \delta L \cdot p_s \cdot \Delta a \cdot A \cdot t/s \cdot \Delta m \quad (3.12)$$

Fig. 3.28 Schematic laboratory setup (wet-cup method) for the determination of the water vapor diffusion resistance factor (μ -value). The air moisture content in the desiccator is set at 43% relative humidity using K_2CO_3 . In this atmosphere the wet cup is placed filled with distilled water and a relative humidity (RH) of 100%. A cylindrical core disk is placed tight on top of the wet cup



where δL ($\text{kg}/\text{Pa} \cdot \text{m} \cdot \text{s}$) is the water vapor diffusion coefficient of air (at 21°C : $1.96 \times 10^{-10} \text{ kg}/\text{Pa} \cdot \text{m} \cdot \text{s}$), p_s the partial pressure of the saturated air ($2.49 \times 10^3 \text{ Pa}$), Δa the difference of the relative humidity, A the effective sample area (m^2), s the sample thickness (m), t the diffusion time in s , and Δm the weight change in kg .

Typical values for some building stones are given in Table 3.3. From these values, the porosity and also the pore radii distribution have a great influence on the μ -value, because at elevated air humidity levels capillary condensation occurs, especially in the small pores of the porous dimension stones.

3.4 Thermal Properties

3.4.1 Thermal Conductivity

The thermal conductivities of the rocks used for buildings have an effect on the climate of the building and of its interior, especially in cases where the rocks are used as heat insulators. Porous rocks, for example, can delay the heat exchange between the inner and outer environment of a building, and as a result the rooms become equally warm inside. Dense rocks with a good thermal conductivity often show a tendency towards moisture condensation at their surface. In general, rocks are poor heat conductors. Their heat conductivity, however, depends on the rock-forming minerals and their specific heat conductivity. Minerals with low heat conductivity values transmit the heat from the rocks' surface to the inner part of the rock very slowly. For dimension stones this is of importance when rocks or some parts of a rock are exposed to direct solar radiation.

Three petrophysical values characterize the thermal properties of rocks: the thermal conductivity λ , the temperature conductivity (thermal diffusivity) κ , and the specific heat c . The thermal conductivity can be described as the coefficient of the steady state heat transfer, whereas the temperature conductivity is a coefficient that describes the transient heat transport. The latter is an expression of how fast a temperature change spreads through a specific rock or medium. Both physical values are related to the density ρ and specific heat c of the medium, with:

$$\lambda = \kappa \rho c \quad (3.13)$$

where λ is given in $\text{Wm}^{-1} \text{ K}^{-1}$. The thermal conductivity λ is a basic parameter (along with the heat production) for localizing geothermal fields in the Earth's crust, with application for example in developing geothermal energy. In building stones the exposition is an important extrinsic parameter for conductive heat transfer. The relationship between the temperature gradient ($\text{grad } T$) and the heat flow (q) with the thermal conductivity (λ) as a material property is given by the Fourier equation as following:

$$q = -\lambda \text{ grad } T. \quad (3.14)$$

The two equations above only holds true for an isotropic case, which is not realized in nature. However, for a rough estimation it may be a useful approximation. The thermal conductivity can be determined by using stationary and non-stationary methods. Buntebarth (1991) conducted these measurements by using the divided bar technique, while Pribnow et al. (1993) were using a line source. Siegesmund (1994) measured the same sample with both methods and had observed discrepancies that reached up to 25%. In a strong physical sense the line source does not allow for anisotropic media (pers. comm. Buntebarth). The in-situ thermal conductivity of rocks mainly depends on the mineralogical composition, rock fabrics, pressure, temperature, and the type, and degree of fluid saturation.

When considering the rock-forming minerals, a wide distribution of the average thermal conductivity values can be seen, from sillimanite with $9.10 \text{ Wm}^{-1} \text{ K}^{-1}$, over quartz $7.67 \text{ Wm}^{-1} \text{ K}^{-1}$, chlorite $5.14 \text{ Wm}^{-1} \text{ K}^{-1}$, dolomite $4.50 \text{ Wm}^{-1} \text{ K}^{-1}$, calcite $3.57 \text{ Wm}^{-1} \text{ K}^{-1}$, and garnet $3.31 \text{ Wm}^{-1} \text{ K}^{-1}$, down to feldspar $2.3\text{--}2.5 \text{ Wm}^{-1} \text{ K}^{-1}$, muscovite $2.32 \text{ Wm}^{-1} \text{ K}^{-1}$, and biotite with $1.17 \text{ Wm}^{-1} \text{ K}^{-1}$. These values already show that quartz-rich rocks should exhibit a higher average thermal conductivity, whereas for rocks with increasing feldspar contents the thermal conductivity values should decrease.

The thermal conductivity of rocks in comparison to metals is about one decimal power smaller, as for example copper, which has a valued of $371 \text{ Wm}^{-1} \text{ K}^{-1}$ (Landolt–Börnstein 1982). Extensive literature exists on the thermal conductivity data of rock-forming minerals and rocks (e.g. Horai and Simmons 1969, Wenk and Wenk 1969, Horai and Susaki 1989). However, the values cannot really be compared, since they were determined by applying different methods and methodological approaches.

The thermal conductivity of rock samples increases with increasing pressure. The most rapid increase with increasing pressure (Fig. 3.29) is observed in the low pressure range and was attributed to the closure of microcracks and voids (e.g. Woodside and Messmer 1961, Buntebarth and Rueff 1987, Horai and Susaki 1989). This phenomenon is less important for dimensional stones, except when the rocks are under loading stress orientations relevant to the statics. The experimental studies however show that above 200 MPa the thermal conductivity exhibits a nearly linear behavior reflecting the intrinsic properties of a rock. The thermal conductivity decreases with increasing temperature. Following a compilation given by Clauser and Huenges (1995) and Buntebarth (1992), it is obvious that the porosity and the pore fluid (air or air-water mixture) may decrease the thermal conductivity due to a decreasing bulk density as a result of an increasing porosity.

In Table 3.6 a compilation of thermal conductivity for selected rock values are given. Data from marine basalts from Hawaii (Fig. 3.30) show a considerable variation of the thermal conductivity due to changes in the porosity (between 5% and 75%) and due to differences in the pore fluid, either pores filled with water (between $2.5 \text{ Wm}^{-1} \text{ K}^{-1}$ and $1.0 \text{ Wm}^{-1} \text{ K}^{-1}$) or air ($1.7 \text{ Wm}^{-1} \text{ K}^{-1}$ and $0.5 \text{ Wm}^{-1} \text{ K}^{-1}$); see Robertson and Peck (1974). The effect of air-, oil-, and water-saturated conditions on quartz-rich sandstones also shows that the air-saturation has a much more pronounced effect on the thermal conductivity than the oil- or water-saturated ones

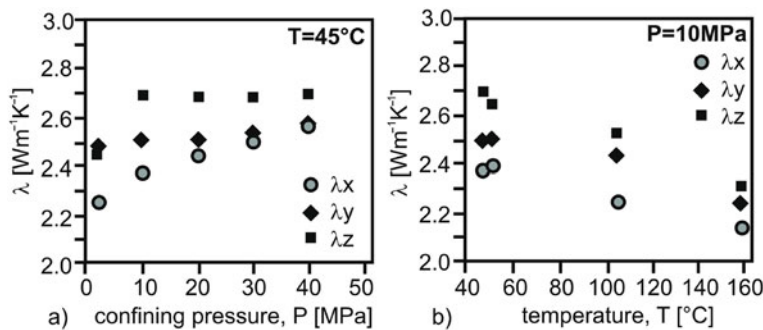


Fig. 3.29 Directional dependence of the thermal conductivity λ parallel to the x-, y- and z-direction (λ_x , λ_y , and λ_z) as a function of (a) confining pressure up to 40 MPa at constant temperature ($T = 45^\circ\text{C}$), and (b) as a function of temperature up to 160°C at constant confining pressure (10 MPa) (after Siegesmund et al. 1999)

Table 3.6 Thermal conductivity values of selected rocks together with density and porosity

Rock type	Density g/cm^3	Porosity %	Thermal conductivity $\text{W/m}^{-1}\text{K}^{-1}$
SANDSTONE			
Postaer Sandstone	2.06	22.7	0.93–1.13
Cottaer Sandstone	2.03	24.1	1.54–1.72
Ruhr Sandstone	2.53	6.1	0.65–1.40
Obernkirchner Sandstone	2.15	18.6	0.81–1.69
Sander Schilf-Sandstone	2.30	14.3	0.83–1.29
LIMESTONE			
Muschelkalk Halberstadt	2.18	20.1	0.76–0.83
Krensheimer	2.60	2.71	0.78–1.23
Muschelkalk			
Schaumkalk Naumburg	1.79	33.3	1.32–1.82
Anröchter “Sandstone” (clastic limestone)	2.51	7.00	1.33–2.04
VOLCANIC ROCKS			
Rhyolite	2.46–2.75	0.31–9.98	0.73–3.13
Basalt	2.04–3.20	1.75–29.06	0.51–2.03
Diabase	2.75–3.15	0.20–10.60	2.13–2.90
PLUTONIC ROCKS			
Granite	2.55–2.74	0.16–1.65	1.34–3.69
Syenite	2.55–2.84	0.33–0.72	1.74–3.9
Diorite	2.63–3.26	0.24–0.69	1.38–2.89
Gabbro	2.63–3.26	0.24–0.69	1.80–2.83
METAMORPHIC ROCKS			
Marble	2.64–2.82	0.02–0.87	1.59–4.00
Serpentine	2.55–2.89	0.25–1.44	2.31–2.87
Quartzite	2.58–2.75	0.18–0.46	2.68–7.60
Phyllite	2.64–2.88	0.28–1.44	1.67
Schists	2.64–2.88	0.28–1.44	1.03–4.93
Gneisses	2.57–2.88	0.11–0.71	0.94–4.86

From Langheinrich (1983), Schön (1983), Buntebarth und Rueff (1987), Siegesmund (1994), Szilagyi (1995), Peschel (1983), Schön (1996), Mosch and Siegesmund (2007)

Fig. 3.30 Variation of thermal conductivity with porosity for air and water filled basalt samples from Hawaii (after Robertson and Peck 1974)

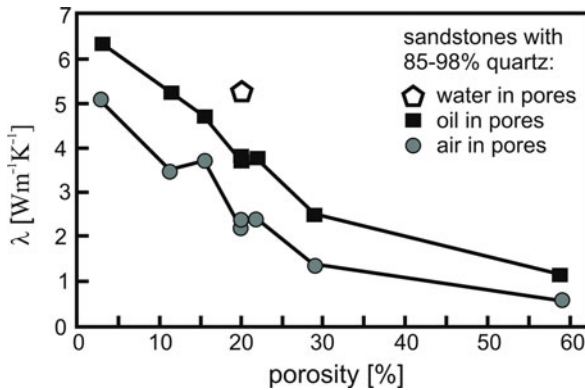
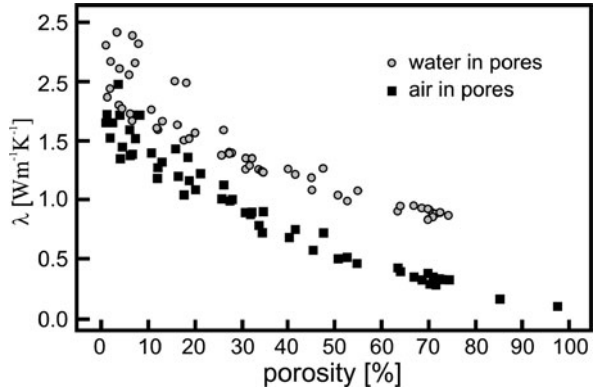


Fig. 3.31 Variation of thermal conductivity with porosity for water, oil- and air-filled pores in quartz-rich sandstones (after Messmer 1965)

(Fig. 3.31). Transferring these results to the dimension stones it can be assumed that water retention leads to a significant increase in the thermal conductivity. Examining the data in Table 3.6 it stands out that the mineralogical composition has a substantial influence on the thermal conductivity.

Different empirical methods exist for the calculation of the thermal conductivity of rocks (e.g. Birch and Clark 1940, Woodside and Messmer 1961, Horai and Baldrige 1972). The basic assumption of these methods is that they used a model of idealized parallel arranged layers with the number and thickness of each individual component related to the rock-forming minerals and their volume fraction. These relationships correspond to the averaging procedures of Voigt and Reuss in the case of the thermal conductivity parallel and perpendicular to the layer boundaries, respectively. Empirically derived averaging schemes have been described in the literature, which demonstrate a better fit between the experimentally measured and the calculated thermal conductivities (Schön 1983, Siegesmund 1996).

For minerals as well as rocks with a pronounced rock fabric, like gneiss, marble, or schists, the thermal conductivity can be strongly directionally dependent. Langheinrich (1983) found that the thermal conductivity in slates showed considerable differences in different directions. Slates with pronounced fabric anisotropy also show a relatively high anisotropy of the thermal conductivity. Clauser and Huenges (1995) showed for a gneiss sample and an amphibolite that the thermal conductivity may vary for the same sample from 100% parallel to the layering to about 60% perpendicular to the layering (foliation). Siegesmund (1994) determined values for a comparable paragneiss ranging from around $2.5\text{--}4.1\text{ W m}^{-1}\text{ K}^{-1}$ (Fig. 3.32).

For all metamorphic rocks investigated Langheinrich (1983) could show that the highest values of the thermal conductivity are observed in the direction parallel to the lineation. The lowest values were measured perpendicular to the foliation. These data are also in accordance with measurements from Wenk and Wenk (1969). The reason for this rock fabric related anisotropy of the thermal conductivity can be found in the preferred orientation of the mica minerals, mainly muscovite, with their mineralogical (001) planes parallel to the foliation (layering) combined with a strong directional dependence of the thermal conductivity of the mineral itself (Fig. 3.33). The single crystal thermal conductivity for muscovite, for example, is in the range from $0.84\text{ W m}^{-1}\text{ K}^{-1}$ (parallel c-axis) to $5.11\text{ W m}^{-1}\text{ K}^{-1}$ (parallel a- and b-axis) and an anisotropy of 83% (Fig. 3.33). This general explanation also holds for metamorphic slates, as here one of the rock-forming minerals are the phyllosilicates like illites/muscovites and chlorites. Quartz, which may also show some preferred orientation in these rocks, only plays a subordinate role with respect to the whole-rock anisotropy of thermal conductivity. However, the mineral itself exhibits a significant directional dependence of the thermal conductivity, with $11.3\text{ W m}^{-1}\text{ K}^{-1}$ parallel the c-axis and $6.5\text{ W m}^{-1}\text{ K}^{-1}$ parallel the a-axes, and having an anisotropy of 42%. Calcite, as the main mineral in limestone and

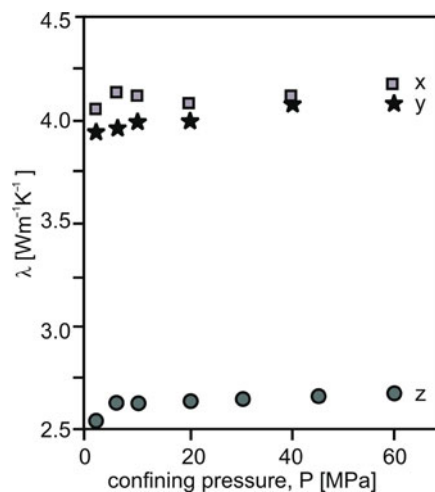


Fig. 3.32 Directional dependence of the thermal conductivity (λ) of a foliated paragneiss sample as a function of pressure up to 60 MPa (at room temperature) using the divided bar technique (x parallel to the lineation within the foliation plane, y normal to the lineation within the foliation, and z normal to the foliation and lineation), after Siegesmund (1996)

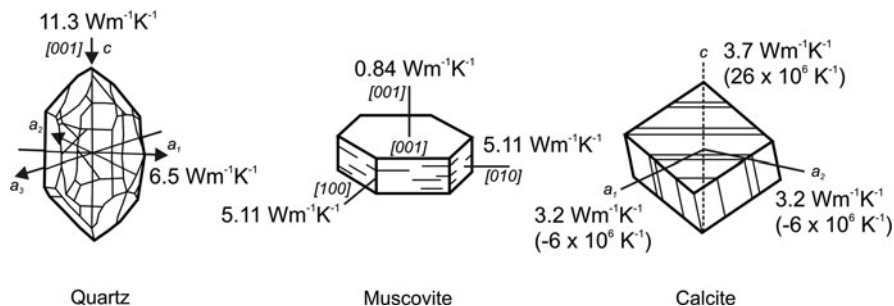


Fig. 3.33 Thermal conductivity values (in $\text{Wm}^{-1}\text{K}^{-1}$) of quartz, muscovite, and calcite in relation to the crystallographic axis. For calcite the thermal expansion coefficients are also given (after Siegesmund 1996; Siegesmund et al. 1997)

marbles, only shows a weak anisotropy (13%), with the highest value parallel the c -axis ($3.7 \text{ Wm}^{-1} \text{K}^{-1}$) and the lowest parallel to the a -axes ($3.22 \text{ Wm}^{-1} \text{K}^{-1}$). However, calcite exhibits a strong directional dependence of the thermal expansion, which is relevant for the application of marbles as dimension stones (see Sect. 3.4.2).

3.4.2 Thermal Expansion

Thermal expansion is an important property to characterize dimension stones, especially for their application and use in structural engineering, e.g., as cladding panels in exterior use or as flooring. For these applications a standardized evaluation of the expansion behavior during thermal exposure is essential and necessary (e.g., Peschel 1983). This includes the volume expansion of the rock as well as the anisotropy of the thermal expansion and the residual expansion, i.e., permanent length change after thermal treatment (related to thermal crack formation). The knowledge of the thermal expansion together with other petrophysical properties allows conclusions to be drawn concerning the suitability of a given dimension stone for certain applications. Changes in temperature and moisture are supposed to be important factors in stone degradation, as mentioned in Sects. 3.3 and 3.5 and further discussed in Sect. 4.2).

Comparable to other solid media a rock changes its volume when subject to a temperature change. The related thermal properties of a rock can be derived from its mineral content and composition. Most of the minerals show an anisotropic behavior of their linear thermal expansion, similar to many other mechanical properties, and so an anisotropic behavior for the rocks can be assumed. For rock samples with no or less pronounced preferred orientation of rock fabric elements (see Sect. 2.8) the thermal expansion shows more or less similar values in different directions. However, the linear thermal expansion coefficient of rock forming minerals is often only valid for a certain temperature interval. The coefficient should not be extrapolated into other temperature intervals, as the thermal expansion does not linearly increase with increasing temperature. For practical applications it is important to

know that after a rock sample experienced a temperature impact the sample length or volume will not be restored to its original value, which means that the temperature impact has resulted in a permanent form change.

Thermal expansion (millimeter/meter or mm/m) expresses the relative length change of a polycrystalline sample with changing temperature (Grüneisen 1926). The connection to the temperature is non-linear, i.e., the thermal expansion coefficient α (K^{-1}), which describes the specific length change per unit temperature depends on the considered temperature interval. The thermal dilatation coefficient α is calculated as the ratio between the length change of the samples ΔL and the original sample length L multiplied by the temperature interval ΔT in Kelvin ($\alpha = \Delta L/L \times \Delta T$). The residual strain (ϵ_{rs} in mm/m) is defined as the ratio between the length change of the sample after cooling down to room temperature ΔL_{rt} and the original sample length L as shown in the following equation:

$$\epsilon_{rs} = \Delta L_{rt}/L \quad (3.15)$$

ϵ_{rs} can only be related to one temperature level, in this case to room temperature before and after any temperature cycle.

So far no accepted international norm for the measurement and determination of the thermal expansion is available and it is not yet expected for the near future. Thermal expansion can be measured by means of a pushrod dilatometer (Fig. 3.34), which allows the measurement of six cylinders simultaneously (e.g., Strohmeyer 2003, Koch and Siegesmund 2004). The dilatometer consists of three main units: the heating unit, the specimen holder in the climate chamber and the displacement register. Taking room temperature as the base one, heating and cooling are tested at a $0.5^\circ\text{C}/\text{min}$ rate to ensure thermal equilibration of the specimen and to simulate conditions at a building as closely as possible. After reaching the designated temperature of each cycle, the temperature is kept at that level until the sample length remains stable (Fig. 3.35). This operating sequence allows detection of (a) the thermal dilatation by simulating temperature changes comparable to those observed under natural conditions; (b) the buffering effect of pre-existing microcrack systems due to former thermal degradation or pre-existing microcracks originating from the sample's geological history (Siegesmund et al. 2000b); (c) the effect of the increase of the thermal dilatation coefficients of rock-forming minerals; and, (d) the influence of intrinsic properties and thermal degradation (Zeisig et al. 2002).

Temperature changes, especially in thermally sensitive marbles, can cause a pronounced loss of relief structure due to granular decohesion. This weathering phenomenon is also known as insolation weathering if the main heat source is the sun (Bland and Rolls 1998). Evidences for this type of weathering are known from many hot rocky deserts (Roth 1955, Goudie 1974, Peel 1974, Ollier 1984). There diurnal temperature changes on the rock surface of approximately 42°C may occur (Peel 1974). High temperature changes are also reported from cold environments. Even under mid-European climates, however, temperature changes of about $35\text{--}40^\circ\text{C}$ on stone surfaces between day and night can be observed (Weiss 1992). The experimental simulation of this type of degradation has a long tradition. For example, Griggs (1936) heated and cooled coarse-grained granite through

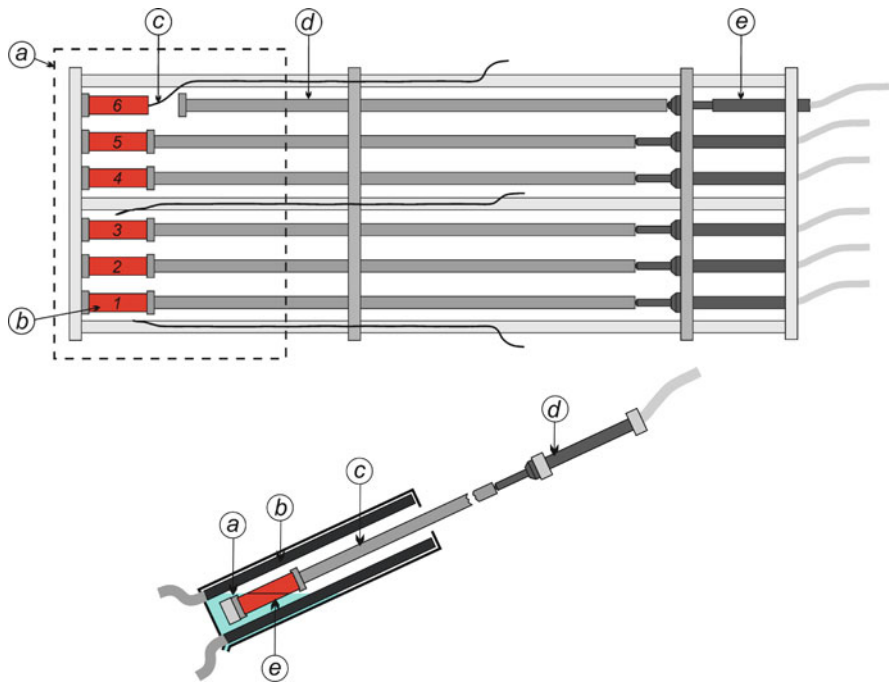


Fig. 3.34 Schematic diagram of a pushrod dilatometer: Top: (a) Position of the climate chamber, (b) Sample cylinder 50 mm in length and 15 mm in diameter, (c) Temperature sensor, (d) Quartz glass pushrod, (e) Display sensor. Bottom: (a) Water level, (b) Heat exchanger copper plate. (c) Quartz glass pushrod. (d) Displacement sensor, (e) Sample cylinder (Strohmeier 2003)

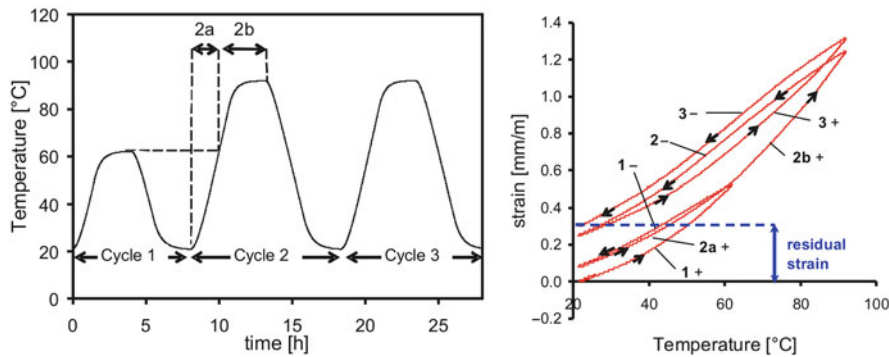


Fig. 3.35 (a) Temperature course of cycles 1–3 performed in the thermal dilatation measurements under dry conditions. Heating and cooling rate is 0.5°C/min. The destination temperatures are 60°C in cycle 1 and 90°C in cycle 2 and 3. (b) Specific example where the course of the expansion is indicated by arrows and numbers while heating (+) and cooling (–)

a temperature range of 110°C for a simulated period of 240 years corresponding to 89,400 heating cycles without evidence of any decay phenomena. In contrast, Aires-Barros et al. (1975) observed for comparable heating cycles a destruction of the rock fabric and a loss of material up to 4.26 weight-%

Studies from Ide (1937), Richter and Simmons (1974), Simmons and Cooper (1978), Bauer and Johnson (1979), and Glover et al. (1995), investigated the thermal expansion of magmatic rocks, like granite and norite, or quartzite (Lu and Jackson 1998). They showed that also for these rocks the occurrence of thermal cracks can be expected. However, most of these studies focused on the expansion behavior of these rocks under earth crust conditions with temperatures (up to 700°C) and higher pressures (Lu and Jackson 1998). Therefore, conclusions about expansion values and behaviors for central European surface temperature ranges and normal pressure have not been derived. However, very high temperature values can be seen on surface sites where fire impact played a role (see below).

When a stone is heated it expands and contracts when it is cooled. While the thermal expansion coefficient characterizes the materials response to heating or cooling, the residual strain after thermal exposure characterizes the thermal sensitivity of a rock. Figure 3.36 depicts very good examples of the thermal behavior of three different marble types. For each marble sample specimens were prepared cut in three perpendicular directions to each other in order to identify possible directional dependencies of the thermal properties. The graphs of thermal dilatation as a function of temperature show the following characteristics: both samples of Carrara Marble (CA1 and CA2) have a very similar lattice preferred orientation of the calcite grains and similar grain size (see Chap. 2) but exhibit a completely different magnitude of the residual strain after heat treatment up to 120°C. Sample CA2 (Fig. 3.36b) exhibits a large residual strain while the sample CA1 (Fig. 3.36a) has a small one. According to the weak texture of both samples, the directional dependence of the thermal expansion coefficient is small. The corresponding curves are almost linear with the temperature when it is raised for CA1, but for CA2 the slope increases when reaching about 60–70°C (dotted line in Fig. 3.36b). This indicates that (i) thermally induced cracks are generated at a certain critical crack initiation temperature; or, (ii) the total thermal dilatation coefficient is buffered by pre-existing critical crack systems at temperatures lower than the above mentioned critical temperature. The Kauffung Marble (Poland) is another exceptional example showing a strong directional dependence of the thermal expansion. However, no residual strain even after heat treatment up to 120°C can be observed and the dilatation increases almost linearly with temperature. Accordingly, the slope of the thermal dilatation curves is quite linear (Siegesmund et al. 2000a, Weiss et al. 2000). Any residual strain left in a stone after heating is the beginning of its deterioration.

For marbles, an average thermal expansion value of around $11 \times 10^{-6} \text{ K}^{-1}$ was calculated based on a larger sample set (Zeisig et al. 2002). In summary, up to now all experimentally determined polycrystalline thermal expansion behaviors of marbles as a function of heating and subsequent cooling cycle can be classified in four main categories: (a) isotropic thermal expansion without residual strain; (b) anisotropic thermal expansion without residual strain; (c) isotropic thermal

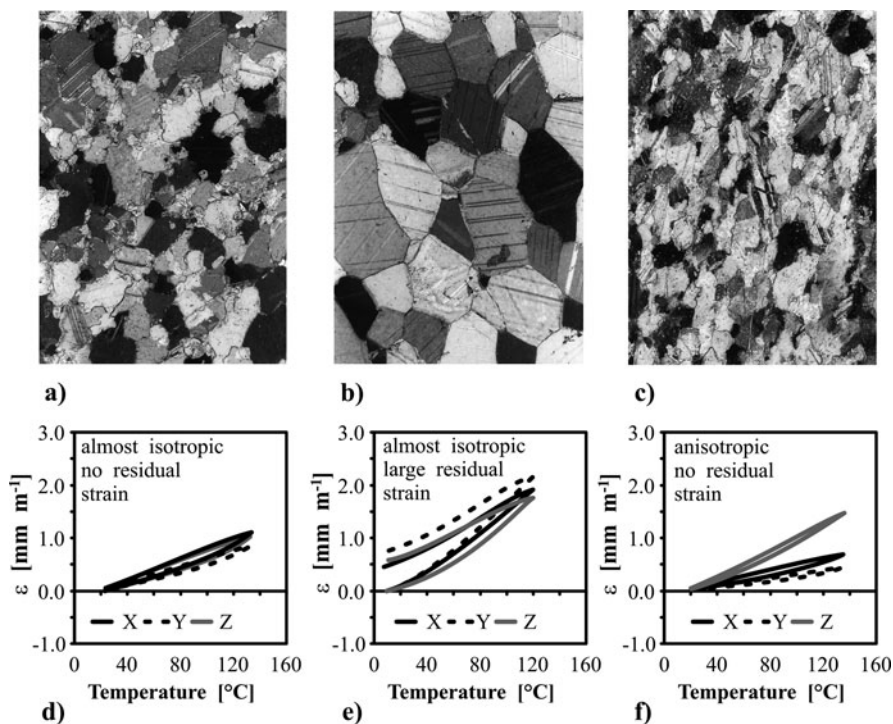


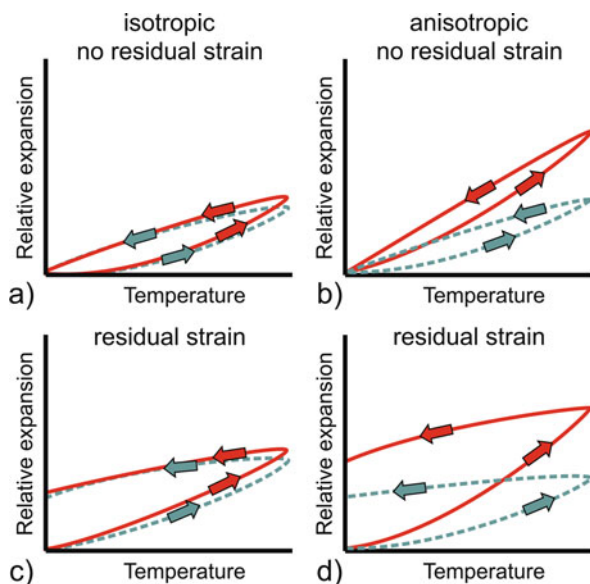
Fig. 3.36 Microfabrics of different marbles (long side of the photograph corresponds to 700 μm) with (a) Carrara Marble (CA1) with irregular grain boundaries, a bimodal grain size distribution with small recrystallized grains and large relic grains, (b) Carrara Marble (CA2) with straight grain boundaries (so called *foam structure*), (c) Kauffung Marble (KA) with elongated grains showing irregular grain boundaries. The macroscopically visible foliation is parallel to the long side of the photograph. Experimentally determined dilatation as a function of thermal treatment up to 120°C is shown in (d) Carrara Marble (CA1), (e) Carrara Marble (CA2), and (f) the Kauffung Marble (KA). Note the considerable residual strain of the CA2 marble, while no residual strain is observed for the other Carrara Marble. The different curves for each sample correspond to different directions of the investigated sample cube (after Weiss et al. 2001)

expansion with residual strain; and, (d) anisotropic thermal expansion with residual strain (Fig. 3.37).

The thermal expansion coefficients for a number of rocks are compiled in Table 3.7. They have been calculated for the temperature interval between 20°C and 90°C. Since some of the thermal expansion curves are not linear, this approach has to be considered as an approximation (see discussion in Zeisig et al. 2002 or Weiss et al. 2004b).

For magmatic rocks (Table 3.7) the thermal expansion coefficient varies between $5 \times 10^{-6} \text{ K}^{-1}$ and $10 \times 10^{-6} \text{ K}^{-1}$. Samples with extraordinary low values (like the gabbro-norite) contain a high amount of plagioclase with extremely small volume expansions. Typical expansion curves show a straight increase and decrease in length due to temperature (Weiss et al. 2004b). In some cases granites may also

Fig. 3.37 Schematic illustrations of the thermal expansion behavior of marble samples. The arrows indicate directions of temperature change (heating and subsequent cooling) (after Siegesmund et al. 2004a)



show a residual strain, which is also evident for those rocks that may show a bowing of thin slabs. For the Cezlak Granite (Slovenia) the thermal expansion is significantly anisotropic ($A = 0.25$) with $\alpha = 7.89 \times 10^{-6} \text{ K}^{-1}$ parallel to the z-direction, i.e. normal to the foliation plane (xy), and $\alpha = 5.89 \times 10^{-6} \text{ K}^{-1}$ parallel to the y-direction (Fig. 3.38; Siegesmund et al. 2008a).

Gneisses and schists vary between $6 \times 10^{-6} \text{ K}^{-1}$ and $9 \times 10^{-6} \text{ K}^{-1}$. Most gneiss samples show linear expansion behaviors similar to the magmatic rocks. An exception is the Knotenschiefer Slate from Theuma (Germany), which predominantly contains quartz and mica and shows high thermal expansion coefficients (Fig. 3.39). These minerals have a high volume expansion coefficient. Feldspar (with a low volume expansion coefficient) is lacking. The thermal expansion curve of Theuma is also characterized by a dehydration reaction, which is most pronounced in the direction perpendicular to the foliation (residual strain -0.3 mm/m at 20°C) and almost negligible parallel to the foliation.

Quartzitic rocks lie within the range of quartz-rich stones. The thermal expansion coefficient for quartz lies between $9 \times 10^{-6} \text{ K}^{-1}$ and $14 \times 10^{-6} \text{ K}^{-1}$. Quartzite shows almost the entire range of this variation as a consequence of the strong lattice preferred orientation of quartz. In general, the thermal expansion coefficient for sandstones is in the range of $10 \times 10^{-6} \text{ K}^{-1}$. A rare exception is the Baumberger “Sandstone” (clastic limestone) whose average thermal expansion coefficient is around $6.5 \times 10^{-6} \text{ K}^{-1}$. Even if the directional dependence in this rock is weak, its magnitude is directly controlled by the mineral composition, that is, the relative frequency of highly expanding calcite and the lesser expansion in feldspar and quartz.

Table 3.7 Thermal expansion coefficient α (10^{-6} K^{-1}) for different directions (x-, y-, and z-direction are perpendicular to each other) for various dimension stones with commercial name, country of origin, and rock type given (Strohmeyer 2003, Mosch 2009, Hoffmann 2008, Ruedrich and Siegesmund 2007, Weiss et al. 2004a, Morales 2011 and unpubl. data)

Commercial name	Country of origin	Rock type	Thermal expansion coefficient α				
			x-direction (10^{-6} K^{-1})	y-direction (10^{-6} K^{-1})	z-direction (10^{-6} K^{-1})	Average value (10^{-6} K^{-1})	Anisotropy (%)
Ben Tak White	Thailand	Granite	7.20	7.99	7.51	7.57	9.9
Muang Tak Orange I	Thailand	Granite	9.07	8.14	8.60	8.60	10.3
Kösseine	Germany	Granite	9.37	8.12	10.20	9.23	20.4
Antrona	Italy	Granite	7.02	6.70	6.68	6.80	4.8
Rojo Dragon	Argentina	Granite	9.65	9.72	10.00	9.79	3.5
Padang	China	Granodiorite	6.73	6.99	8.32	7.35	19.1
Ben Tak Blue	Thailand	Quartz-monzonite	7.97	7.58	7.24	7.60	9.2
Salmon Red	Uruguay	Syenite	6.50	6.12	7.59	6.74	19.4
Artigas	Uruguay	Syenite	4.07	7.53	5.96	5.85	45.9
Blue Pearl	Norway	Larvikite	5.42	3.98	9.63	6.34	58.7
Nero Absoluto	Uruguay	Dolerite	6.66	6.79	6.68	6.71	1.9
Anzola	Italy	Gabbro	6.89	7.23	8.02	7.38	14.1
Nero Impala	South Africa	Gabbro-norite	5.14	4.68	5.23	5.02	1.7
Negro Riojano	Argentina	Gabbro	7.93	7.85	7.42	7.73	6.4
Balmuccia	Italy	Peridotite	8.16	8.69	8.74	8.53	6.6
Löbejün	Germany	Rhyolite	7.27	6.47	6.99	6.91	11.0
Stardust Grey	Argentina	Rhyolite	7.58	8.00	8.62	8.07	12.1
Monte Merlo	Italy	Trachyte	7.14		4.03	5.59	43.6
Drachenfels	Germany	Trachyte	4.66		7.22	5.94	35.5
Weibern	Germany	Phonolite tuff	5.72	3.23	6.09	5.01	47.0
Rochlitz	Germany	Rhyolite tuff	8.00	8.28	7.74	8.01	6.5
Eger-DEMjen	Hungary	Dacite tuff	4.64		4.89	4.77	5.1
Phanom Sarakham Grey	Thailand	Gneiss	8.95	8.82	9.00	8.92	2.0

Table 3.7 (continued)

Commercial name	Country of origin	Rock type	Thermal expansion coefficient α				
			x-direction (10^{-6} K^{-1})	y-direction (10^{-6} K^{-1})	z-direction (10^{-6} K^{-1})	Average value (10^{-6} K^{-1})	Anisotropy (%)
Azul Tango	Argentina	Gneiss	7.09	8.35	8.25	7.90	15.1
Calanca	Italy	Gneiss	6.93	6.54	6.81	6.76	5.6
Verde Andeer	Italy	Gneiss	7.63	6.41	8.97	7.67	14.9
Serizzo Monte Roas	Italy	Gneiss	7.65	6.87	7.88	7.47	2.9
Wang Nam Kiew Black	Thailand	Hornblende	7.29	7.20	6.64	7.04	8.9
Franco Veteado	Argentina	Migmatite	9.01	8.88	9.11	9.00	2.5
Rosa Estremoz	Portugal	Marble	4.37	11.28	10.23	8.63	61.3
Carrara	Italy	Marbe	9.72	10.34	12.46	10.84	22.0
Carrara	Italy	Marble	4.81	6.64	6.90	6.12	30.3
Phran Kratai Grey	Thailand	Marble	-0.60	2.20	11.70	4.43	105.1
Azul Cielo	Argentina	Marble	1.18	9.70	1.67	4.18	87.8
Azul Imperial	Brazil	Quartzite	11.40	11.50	11.10	11.33	0.9
Camine Rufo	Uruguay	Slate	9.99	9.96	1.28	7.08	87.2
San Luis Slate	Argentina	Slate	9.50	9.89	1.17	6.85	88.2
Theuma	Germany	Slate	9.84	9.34	12.08	10.42	22.7
Globergerina	Malta	Limestone	4.72	4.57	4.24	4.51	10.2
Caliza Amarilla	Argentina	Limestone	5.14	6.89	6.17	6.07	25.4
Dolomita Dorada	Argentina	Dolomitic limestone	9.20	1.01	9.26	6.49	89.1
Mae Phrik Yellow	Thailand	Limestone	4.36	4.69	4.18	4.41	10.9
Soskut	Hungary	Limestone	3.59	4.54	4.39	4.17	20.9
Jerusalem	Israel	Limestone	3.89	4.61	4.55	4.35	15.6
Kuacker	Germany	Limestone	4.70	6.62	4.71	5.34	29.0
Thüster	Germany	Limestone	1.41	3.24	3.89	2.85	63.8
Bad Langensalza	Germany	Travertine	5.70	5.92	5.50	5.60	7.1

Table 3.7 (continued)

Commercial name	Country of origin	Rock type	Thermal expansion coefficient α				Anisotropy (%)
			x-direction (10^{-6} K $^{-1}$)	y-direction (10^{-6} K $^{-1}$)	z-direction (10^{-6} K $^{-1}$)	Average value (10^{-6} K $^{-1}$)	
Baumberger	Germany	Sandstone-like limestone	5.76	6.46	7.34	6.52	21.5
Rüthener	Germany	Sandstone	11.10	11.70	11.90	11.57	6.7
Bad Bentheim	Germany	Sandstone	11.90	12.00	12.50	12.13	4.8
Schleierthier	Germany	Sandstone	10.50	10.50	9.96	10.32	5.1
Weser	Germany	Sandstone	9.30	7.50	8.40	8.40	19.4
Sikhiu Brown	Thailand	Sandstone	11.93	12.66	12.50	12.36	5.8
Pakchong Green	Thailand	Sandstone	8.91	8.25	8.45	8.54	7.4
Tacuarembó	Uruguay	Sandstone	1.23	1.18	1.24	1.22	4.8

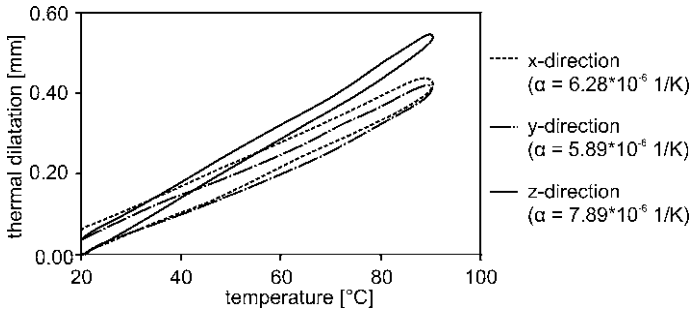


Fig. 3.38 Cezlak Granodiorite: Experimentally determined thermal dilatation as a function of temperature (after Siegesmund et al. 2008a)

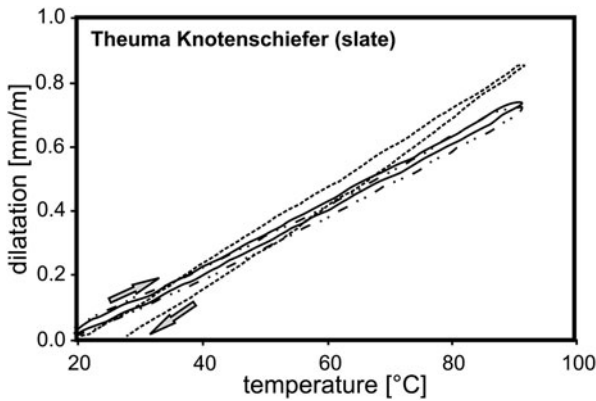


Fig. 3.39 Thermal expansion behavior for the Theuma Knotenschiefer (slate) that shows high coefficients and a dehydration reaction which is most pronounced in the perpendicular to the foliation direction. The thermal expansion path, i.e., heating and cooling is marked by arrows. Heating was performed from 20°C to 90°C

Its behavior is comparable to that of the Anröchter Limestone. Systematic investigations of limestone are still missing. The sparitic Kuacker Limestone has a thermal expansion coefficient of about $5.5 \times 10^{-6} \text{ K}^{-1}$, and has a residual expansion of 0.07 mm/m indicating a strong anisotropy (Fig. 3.40, compare x-, y-, and z-direction, Siegesmund et al. 2010).

Besides dehydration effects related to the mineral composition, samples may show a different thermal behavior when water is present in the pores. The German Weser Red Sandstone (Buntsandstein) may serve as an example for this behavior. When the sample is heated under dry conditions, the thermal expansion coefficient is quite linear and similar at all sample directions (Fig. 3.41). However, it is remarkably different when water is present in the pores (Fig. 3.41). Then, assumedly a kind of dehydration reaction occurs which is most pronounced in the z-direction, i.e., the direction perpendicular to the bedding. The second cycle clearly evidences the previous effect of the pore water, since after the dehydration reaction the thermal

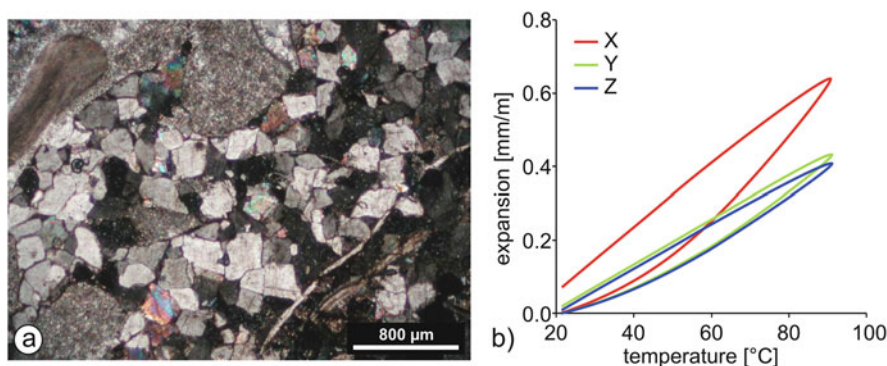


Fig. 3.40 Thermal expansion for the Kuacker Muschelkalk; (a) microfabrics showing a clear sparitic fabric and (b) thermal expansion and its anisotropy as a function of temperature. Note the residual strain values after the heating cycle corresponding to a permanent length change (after Siegesmund et al. 2010)

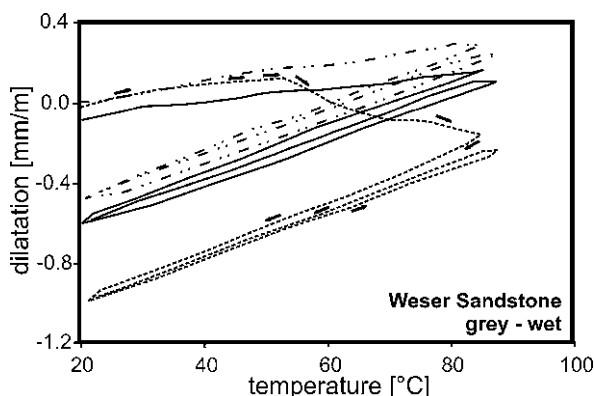


Fig. 3.41 Thermal expansion curve for a Weser Red Sandstone (Buntsandstein). Heating was performed from 20°C to 90°C in two cycles. The thermal expansion path is indicated by arrows. The first cycles clearly indicate a dehydration reaction, i.e. a contraction which is especially pronounced normal to the bedding. In the following cycles the Weser Sandstone behave as expected under dry conditions (after Weiss et al. 2004a, b)

expansion coefficient is the same in the dry condition. This example shows that samples sensitive to hygric/hydric dilatation may also have thermal expansion changes even if this is not the primary reason for these changes.

In summary, most marbles can clearly be considered as thermally sensitive materials (see also discussion in Sect. 4.2) because calcite causes temperature-induced damage already at comparably low temperature intervals (about 40°C; Battaglia et al. 1993).

Even if dolomite marbles are more resistant, the main cause for thermal degradation is the thermal expansion anisotropy of calcite and dolomite and the associated thermal stress generation (Siegesmund et al. 2000a, 2008a, Shushakova et al. 2010).

This effect is much less pronounced but not zero in dolomite marbles (Weiss et al. 2002a, b, Zeisig et al. 2002). Marbles are the only rock types where a preferred orientation of the main constituent minerals can cause certain directional dependences of the thermal expansion coefficient and of the residual strain. Accordingly, marbles show the highest directional dependence of the thermal expansion coefficient (Shushakova et al. 2010).

Sedimentary rocks show large variation of thermal properties. In most cases the thermal expansion curve is quite straight showing no residual strain after heating. However, a certain alteration of this behavior is to be expected when the samples are wet. This is particularly the case for rocks with a high porosity, large pore structure and high phyllosilicate content. Then, only the combination of heating and wetting can cause any remarkable alteration of the thermal expansion behavior (see Sect. 3.5). Even if the directional dependence of the thermal expansion coefficient is weak for sedimentary rocks, its magnitude is directly controlled by the mineral composition, i.e., the relative frequency of highly expanding calcite and weakly expanding feldspar and quartz. Very good examples for this behavior are impure sandstones or limestone, such as the Anröchter “Sandstone” (clastic limestone) discussed previously.

3.4.2.1 Deformation of Stone Slabs

Many authors (Sage 1988, Koch and Siegesmund 2004, Grelk et al. 2004, Siegesmund et al. 2008b) showed that the increase of residual strain stops after few heating cycles if moisture is absent. Therefore Bucher (1956) or Winkler (1996) point out the importance of moisture in the bowing process of stone slabs. To detect the anomalous weathering behavior of rocks the thermal expansion tests must also be carried out under a combination of dry and wet conditions. For this purpose the setup shown in Fig. 3.34 can be modified easily by tilting the specimen holder and the climate chamber. To discuss these effects more detailed measurements of progressive residual strain on a calcite marble were performed. Eight dry cycles up to 90°C were followed by 25 additional wet cycles, whereby the first six wet cycles were carried out in a way that at the end of a heating cycle the samples in the climate chamber were run until totally dry (Fig. 3.42). The findings from this approach can be summarized in two points. Firstly, under wet conditions there is a constant progressive residual strain that increases continuously even after 25 cycles (depending on the marble type). Secondly, the moisture content after heating cycles apparently influences the intensity of the marble deterioration.

The bowing behavior of marble panels can easily be tested in the laboratory (Koch and Siegesmund 2004, Grelk et al. 2004 or Siegesmund et al. 2008b). The test was performed on slabs (400 × 100 × 30 mm) so that each specimen was exposed to moisture on one side and infrared heating on the reverse side. The applied temperature ranged between 20°C and 80°C and a total of 40 cycles were performed. The continuous length change and the effect of anisotropic bowing with the applied cycles are shown in Fig. 3.43. The experiment confirmed that the bowing of the marble depends on the temperature and moisture conditions on the mineralogical

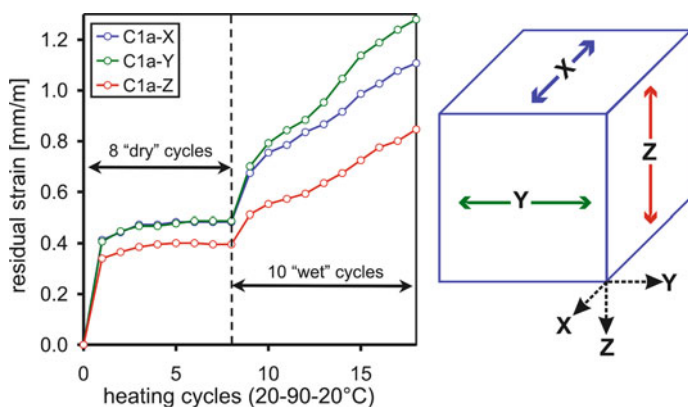


Fig. 3.42 Progressive increase of residual strain of a calcite marble (Carrara) as a function of 8 dry cycles followed by 10 wet cycles (9–18) in three different direction perpendicular to each other (from Koch and Siegesmund 2004)

composition of the marbles as well as on their fabrics (see discussion in Siegesmund et al. 2008b). Bowing deformation (Figs. 3.63 and 3.78) has also been observed on other rock types such as granite (Siegesmund et al. 2008b, Vázquez et al. 2010) or the conglomerate Nagelfluh (Grimm 1999).

3.4.2.2 Thermal Behavior at Higher Temperatures

With respect to thermal expansion most of the rock forming minerals (see Chap. 2) can be seen as anisotropic and therefore most of the dimensions stones also exhibit anisotropy depending on the degree of preferred orientation of the constituting minerals. As explained above, the thermal expansion coefficient depends strongly on the temperature. However, the relevant temperature range for the evaluation of temperature related properties for dimension stone is less than 120°C and may only exceed it for fire impacts. There are few historic and valuable buildings that did not suffer at least one fire during its existence (Sippel et al. 2007).

Thermal expansion measurements up to 1,000°C reveal that every rock shows a specific expansion behavior. Owing to the transition of low- to high-quartz all quartz-bearing rocks experience a sudden and remarkable expansion at temperatures just below 600°C (Fig. 3.44). The surface finishing of dimensional stones known as flamed surfaces is using the effect of higher temperatures to produce rough surfaces. Although the phase transformation of quartz is reversible, a striking positive residual strain can be recognized for the rhyolite and the granite after cooling from maximum temperatures of around 700°C (Fig. 3.44). Temperatures of 500°C result in much lower values of residual strain.

Consequently, all quartz-bearing rocks show varying degrees of intragranular microfractures within quartz grains. Fredrich and Wong (1986) referred to intragranular microfractures in feldspars of heated granites, whereas Hajpal and Török (2004) described intergranular microfractures in quartzitic sandstones for temperatures

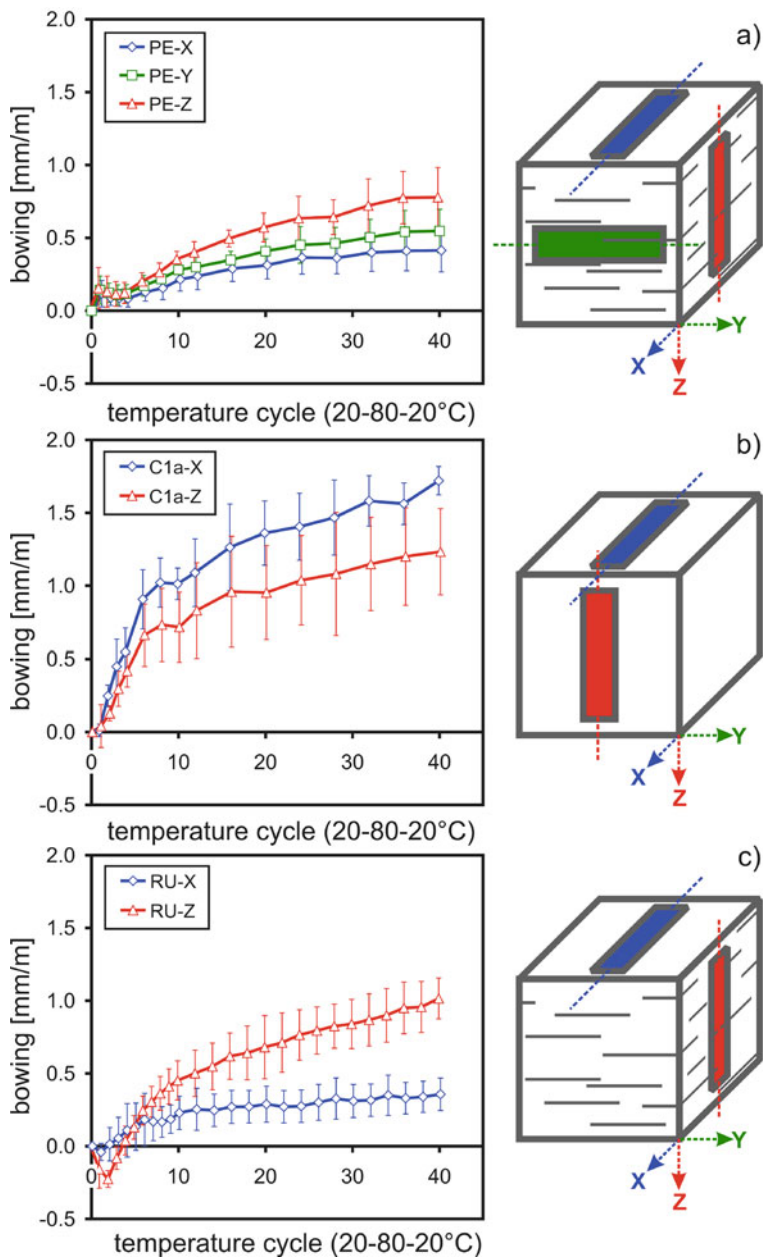
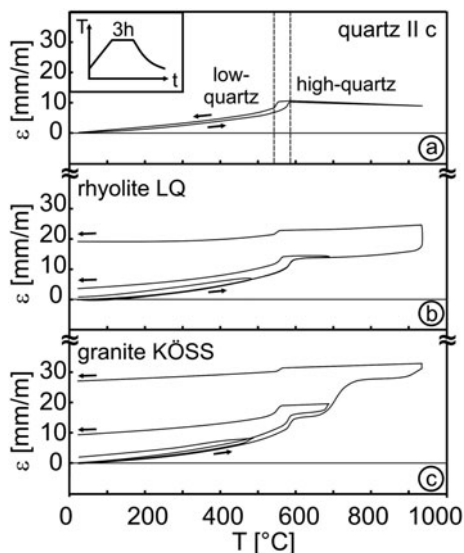


Fig. 3.43 Bowing of marble slabs versus number of heating cycles (left): Peccia (PE), Carrara (CA1), and Ruviana (RU) Marble. Each curve represents the mean bowing trend of three slabs (right). The colors of the curves are related to the cut direction displayed in the cube sketch (right). The colored rectangles represent the test pieces. x, y and z indicate the long axes of the test specimens. Positive values mean convex bowing (towards the heat side), negative ones concave bowing (see also Fig. 6.7a–b in Chap. 6)

Fig. 3.44 Thermal expansion of quartz parallel to the crystallographic c-axis (a) for comparison to the expansion behavior of the rhyolite from Löbejühn (b) and the granite from Kösseine (c) (after Sippel et al. 2007)



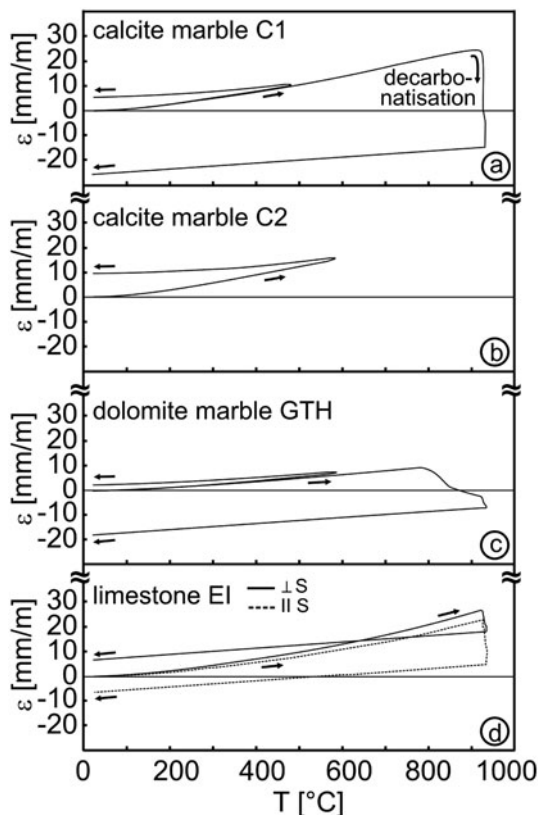
below 500°C (see discussion in Sippel et al. 2007). Residual strains in silicate rocks are significantly higher at temperatures above 700°C, except for very porous rocks (porosities over 15%) since the latter can accommodate the expansions.

The strong expansion occurring above 700°C is explained by a progressive crack growth and finally a high degree of transgranular microcracking. The impact of temperatures above 500°C causes an increasing total porosity and a remarkable shift to larger pore radii, whereas at temperatures below 500°C the pore space does not change significantly. Thermal expansion measurements up to 950°C for silicate rocks generally result in a higher expansion coefficient, α , associated with a higher residual strain and vice versa (Fig. 3.44).

Calcite marbles (Fig. 3.36) exhibit comparably high expansion coefficients and high residual strains when heated up to 100°C (Siegesmund et al. 2000a; Zeisig et al. 2002). This thermal sensitivity can also be observed up to 600°C. For example, the residual strain for the Carrara Marble CA1 (see also Fig. 3.36) is around 5.4 mm/m at 500°C, while the averaged residual strain for silicate rocks is less than 2 mm/m. The most intensive damages are shown by the calcite marble CA2 with a residual strain of 9.5 mm/m (Fig. 3.45). In this case, thermally induced stresses that originated at adjacent grains lead to a total decay, even before decarbonatization starts (Sippel et al. 2007).

In contrast to normal calcite marbles that are translucent since calcite crystals are transparent to translucent, decarbonatized rocks exhibit a rather dull white appearance. Fracture Surfaces of decarbonatized rocks show that the loss of cohesion after decarbonatization occurs along irregular surfaces and that the shape of calcite crystals survives the partial or total decomposition to CaO. The dolomite marbles appear to be more resistant to heat impacts even at temperatures up to 600°C (maximum residual strains of 2.1 mm/m).

Fig. 3.45 Thermal expansion of different carbonate rocks according to the temperature curve shown in Fig. 3.44 (after Sippel et al. 2007)



For the case of the gypsum bearing rock, two effects are crucial for its thermal behavior: (i) the dehydration of gypsum results in a limited expansion followed by a contraction within the temperature range of 180–300°C; and, (ii) the transformation of CaSO_4 (A III) to anhydrite (A II) reveals an intense contraction above 800°C expressed by a negative residual strain of about 20 mm/m (perpendicular to the bedding) and 35 mm/m (parallel to the bedding). SEM images show that the anhydrite crystals change their habit from prismatic to more isometric shapes when heated.

3.5 Hygric/Hydric Properties

The deformation of dimension stones can be related to different processes, mainly changes in the temperature and in the moisture content of the porous rocks. Quite important is the presence of moisture for the case of length or volume changes of dimensional stones (Fig. 3.46). This deformation phenomenon is well known as hygric (in the range between 0% and 95% RH) and hydric (in contact with water, also immersed) expansion and contraction as a result of moisture changes. An important physical weathering process related to this is the periodic or seasonal hygric dilatation of a rock due to environmental climate factors.

Fig. 3.46 Schematic diagram of the hygric dilatation as a function of time for a sandstone sample. Note that the hygric dilatation gradient changes over time (after Stück et al. 2010)

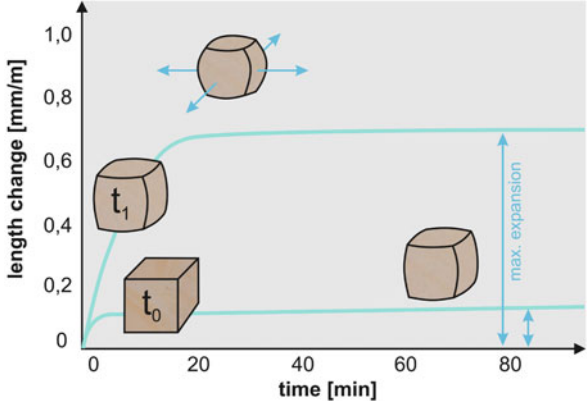
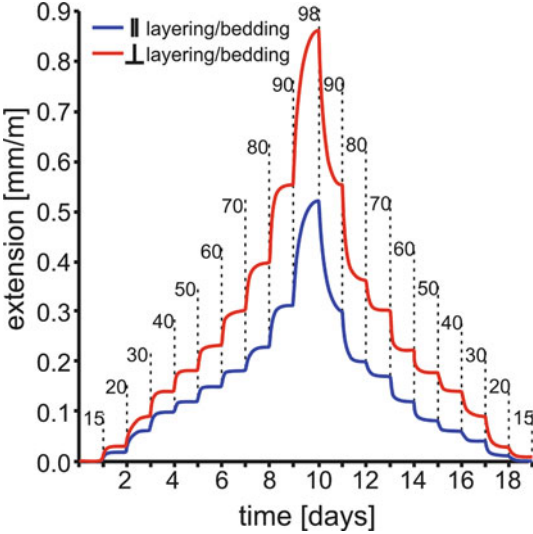


Fig. 3.47 Hygric expansion of dimension stone shown for the case of the Schöttmarer Sandstone. The dotted lines mark the humidity levels at which the sample equilibrated. A clear increase in the sample length occurs above 80% relative air humidity (modified after Ruedrich et al. 2005b)



The process of hygric dilation is related to changes in the relative humidity of the air (Fig. 3.47). For this case relatively high values of hygric dilation can also be found, but compared to direct wetting of the surface with liquid water, the values are much lower (Steindlberger 2003, Ruedrich et al. 2005b, Ruedrich et al. 2010a). The hygric expansion of the Schöttmarer Sandstone, shown in Fig. 3.47, reaches a value of 0.84 mm/m perpendicular to the layering at 98% RH.

The intensity of the hygric dilatation as a function of relative humidity shows large variations depending on the material. Blöchl et al. (1998) found for three sandstone varieties values that ranged between 10% and 28% for the total hygric dilatation at 95% RH. Furthermore, the dilatation of a sample is not equally distributed over a certain range of moisture content. There is a clear increase in the rock expansion at higher values of relative air humidity. The dilatation versus time of the

Schöttmarer Sandstone shown in Fig 3.47 can be seen as a representative curve, where the highest expansion occurs above 80% relative air humidity. At these high relative humidity levels, capillary condensation will increase and the expansions measured will approximate those obtained when the sample is in contact with liquid water for some time. The absolute hygric dilatation can also exhibit a strong directional dependency, where anisotropy values of over 50% are not uncommon (Fig. 3.47, e.g. Ruedrich et al. 2007, 2010a). These clear directional dependencies are often observed with rocks that have experienced additional deformation or compaction, for example mudstones, sandstones or slates.

When samples are exposed to direct contact with liquid water by capillary absorption from one face (mimicking the exposure of stones in buildings), the swelling perpendicular to the bedding was 1.13 mm/m (Table 3.8) in the case of the Schöttmarer Sandstone, which is one-third higher than when only exposed to high relative humidity levels (Fig. 3.47).

The combination of hygric or hydric swelling and thermal dilatation processes (see Sect. 3.4.2) is seen as an important factor for the weathering behavior of dimension stones. The result of these processes on dimension stones can be seen in the form of scaling and flaking parallel to the layering (see Chap. 6).

It is well known that the water impact of dimension stones can result in a volume increase of the rock. An overview of the hydric dilatation of different types of dimension stones is given in Table 3.8, while Fig. 3.48 provides hydric dilatation estimates that can be expected for certain rocks and rock types.

The hydric dilatation values of the three rock groups shown in Fig. 3.48 (Kocher 2005) differ from each other in two orders of magnitude. In the first group, granites, marbles, and carbonate rocks show values of up to 0.1%. In the second group sandstone and concrete show dilatation values of almost 1%, whereas the third group, consisting of mud and mudstones, reaches dilation values of up to 25%.

Besides the swelling of clay minerals, hydric dilatation of dimension stones may be the result of other critical processes (see Chap. 4.). This can be seen in the dilatation of rocks that only have a small amount of clays or are even clay free, such as basalt, granite, marble, flint or chert (Weiss 1992). In these cases, the hydric dilatation was mostly observed for rocks having a large percentage of micropores, thus enhancing capillary condensation processes (Stockhausen 1981).

When clays are present, pronounced values of hydric dilatation have been measured for mudstone, where changes of up to 20 mm/m are not uncommon (Madsen 1976, Madsen and Nüesch 1990). For sandstones, Schuh (1987) and Snethlage and Wendler (1997) reported dilatation values of up to 5 mm/m. However, Ruedrich et al. (2010a) could show that some sandstone varieties exhibit a pronounced hydric expansion, although these rocks do not have any swellable clay minerals.

Hydric dilatation is particularly pronounced in clay-bearing sandstones and volcanic tuff. Jiménez González and Scherer (2004) reported for subarkose sandstone from the USA, Switzerland, and Spain hydric expansion values of 0.5 mm/m and up to 2.0 mm/m (Scherer pers. comm., see also Sebastian et al. 2008, Wangler and Scherer 2008, Wangler et al. 2010). Snethlage and Wendler (1997) published hydric dilatation data for a set of sandstones covering a range up to 5 mm/m

Table 3.8 Hydric expansion of some selected dimension stones with density and porosity

Rock type	Density g/cm ³	Porosity %	Hydric expansion mm/m
SANDSTONE			
Portland Brownstone	2.39	8.7	0.45–0.17
Villarod Molasse	2.26–2.67	15.5	1.30–1.73
Tarifa Sandstone	2.55	8–11	1.20–2.90
Schöttmarer Sandstone	2.45	8.10	0.79–1.13
Meller Sandstone	2.30	13.22	0.63–1.18
Obernkirchner Sandstone	2.15	18.6	0.09–0.16
Sander Schilfsandstone (sandstone)	2.27	15.4	0.83–1.29
LIMESTONE			
Globigerinen Limestone	1.85	34.6	0.17–0.27
Thüste Limestone	2.12	21.14	0.02–0.07
Treuchtling Limestone	2.60	2.71	0.03–0.07
Travertine	2.40	7.38	0.02
Anröchter “Sandstone” (clastic limestone)	2.51	7.00	0.08–0.17
Baumberger “Sandstone” (clastic limestone)	2.09	20.00	0.13–0.62
VOLCANIC ROCKS			
Löbejün Rhyolite	2.52	4.50	0.03
Morshäuser Basalt	2.92	1.56	0.10
Niederbrechener Diabase	2.71	1.08	0.22
Londorfer Basalt Lava	2.86	10.88	0.00
Rochlitz Rhyolitic Tuff	2.01	23.70	0.96
Weibern Tuff	1.51	38.50	0.87–1.00
Laubacher Tuff	2.71	16.31	3.29
Trachyte Selters	2.32	8.00	0.60–1.10
PLUTONIC ROCKS			
Kösseine Granite	2.69	0.71	0.08
Padang Dark Granodiorite	2.80	0.25	0.01
Nero Impala Gabbro	2.92	0.04	0.01
Anzola Gabbro	3.09	0.13	0.03
METAMORPHIC ROCKS			
Calanca Paragneiss	2.73	0.73	0.06
Verde Andeer Orthogneiss	2.68	0.89	0.01
Prieborn Marble	2.70	0.77	0.06
Palissandro Marble	2.87	0.33	0.08
Carrara Marble	2.71	0.46	0.08
Quartzite	2.69	0.18	0.01
Serpentine	2.77	0.15	0.00
Theuma Knotenschiefer (Slate)	2.74	0.95	0.15–1.45
Roof Slate	2.67	0.70	0.26–4.80

Data from Poschlod 1990, Mirwald 1997, Jiménez González and Scherer 2004, Weiss et al. 2004a, Ruedrich et al. 2005b, Ruedrich and Siegesmund 2006, Sebastian et al. 2008, Felix (pers. comm.), Scherer (pers. comm.), and Steindlberger 2003

(for comparison see the compilation by Ruedrich et al. 2005b and Ruedrich and Siegesmund 2006). For volcanic tuffs, Steindlberger (2003) observed maximum values of up to 3.3 mm/m, while most of them exhibit values of around 0.5 mm/ and up to 1 mm/m, which is in agreement with observations made by Wendler et al. (1996), Ruedrich et al. (2005b), and Stück et al. (2008).

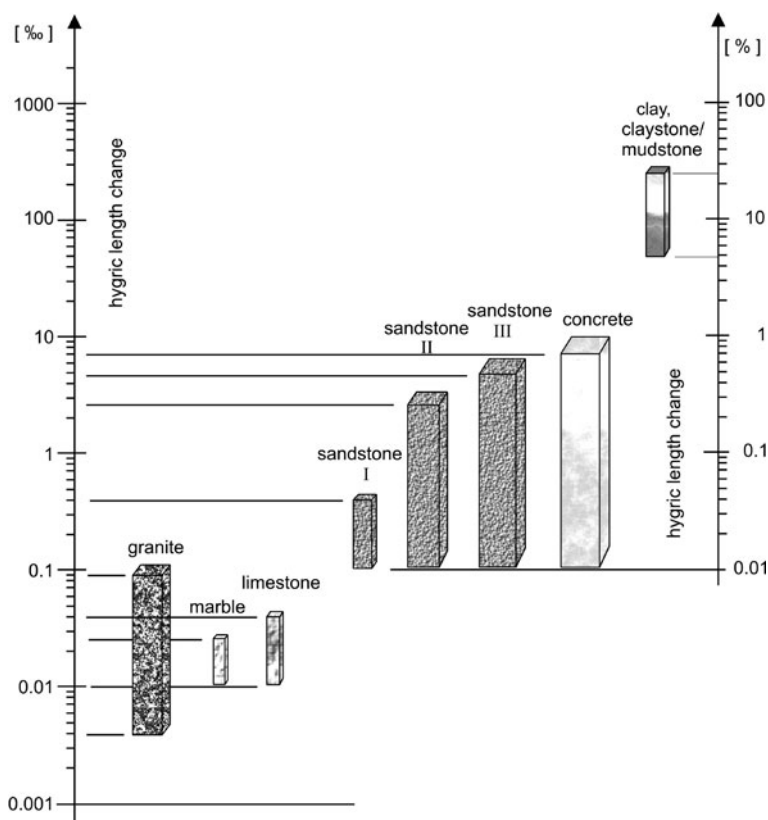
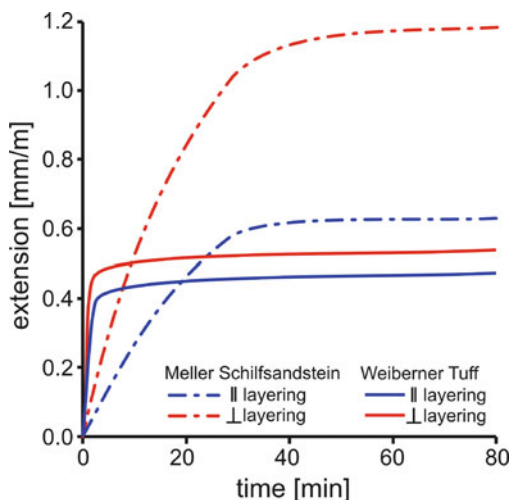


Fig. 3.48 Hygric dilatation values for different rocks and from different authors, granites, marbles, and sandstones after Hockmann and Kessler (1950), from Snethlage (1984), carbonates after Lukas (1990), Sandstone II after Schuh (1987), sandstone III after Snethlage and Wendler (1997), concrete after Wesche (1981), clay/mudstones after Madson (1976) and Madson and Nüesch (1990). After Kocher (2005)

Carbonate rocks usually show low values for hygric dilatation as in general these rocks have low clay contents. For most of the limestones, the hygric expansion is a subordinate factor. The Thüste Limestone (Table 3.8) shows a hygric expansion of 0.02 mm/m parallel to the bedding, while the value perpendicular to the bedding is around 0.07 mm/m (Weiss et al. 2004b). For the travertine of Bad Langensalza, the value is around 0.02 mm/m (Weiss et al. 2004b), whereas Katzschnmann et al. (2006) reported values of less than 0.01 mm/m. Comparable low values were reported for the limestone of the Unterer Muschelkalk (Terebratelkalk and Schaumkalk), where values between 0.03 mm/m and 0.07 mm/m are given by Katzschnmann et al. (2006).

In contrast, the Baumberger “Sandstone” as well as the “Sandstone” from Anröchter (both are clastic limestones containing 16% and 31% quartz, respectively, see Table 3.1) exhibits quite different hygric behavior with different time periods until the maximum expansion was reached, which is depending on the direction. The Baumberger “Sandstone” shows a remarkable swelling of 1.5 mm/m parallel to the bedding after 8 min and of 0.25 mm/m perpendicular to it after 12 min.

Fig. 3.49 Hydric expansion of dimension stones during capillary water uptake for the Meller Schilf Sandstone and Weiberner Tuff parallel and perpendicular to the layering. Note the strong directional dependency associated with the sandstone and the differences in the expansion speed of the samples



The Anröchter “Sandstone” exhibits a smaller expansion, about 0.08 mm/m parallel to the bedding and 0.16 mm/m perpendicular to it. The Globigerina Limestone from Malta shows a hydric expansion from 0.17 mm/m to 0.27 mm/m with a total anisotropy of around 37%.

As examples the hydric dilatation curves of the Meller Sandstone and the Weiberner Tuff are shown in Fig. 3.49. The Meller Sandstone exhibits a relatively large absolute hydric expansion perpendicular to the layering with a value of 1.18 mm/m. Parallel to the layering the hydric expansion during water uptake is much lower, with an absolute value of 0.63 mm/m, which results in an anisotropy of 46.6%. The Weiberner Tuff displays a similar hydric expansion curve with 0.53 mm/m perpendicular and 0.47 mm/m parallel to the weakly developed layering. The corresponding anisotropy is only 11.3%.

Another phenomenon of the hydric dilatation is the strong time dependence of the expansion for different rocks. The expansion of the Weiberner Tuff shown in Fig. 3.49 was completed within approximately 5 min in relation to the sample size, whereas the Meller Sandstone required 40 min for the main expansion. This time control mainly depends on the pore space properties of the rocks, so that rocks with a good connected pore network and higher porosities show faster hydric expansion than rocks with less connected pores and lower porosity values.

Laboratory investigations have shown that the hydric dilatation of dimension stones after a few exposure cycles is more or less a reversible process, where no residual expansion can be measured after the rock sample returns to the original moisture level. Detailed and extensive investigations of the effects of a large number of exposure cycles and long term exposure to moisture and changes in moisture content have not been carried out so far. However, Morales et al. (2007) discussed the decrease in rock strength induced by the impact of water and found a link between the compressive strength and the magnitude of hydric expansion in sandstones. This

connection has been verified for the tensile strength of sandstones in Ruedrich et al. (2010a). The results from Morales et al. (2007) on strength reduction (dependent on the degree of saturation) show that a low degree of saturation for all sandstones studied lead to a strong reduction in the compressive strength. At increasing degrees of saturation, sandstones with a slight hygric expansion and a small proportion of micropores only show a small increase in strength reduction. In contrast, sandstones with a pronounced hygric expansion and a large amount of micropores exhibit a progressive strength reduction at increasing saturation levels. This suggests that there is a relationship between the existent pore sizes, the saturation degree, the intensity of hygric swelling and the strength reduction, i.e. the rock softening of sandstones.

Most of the understanding and relationships related to hygric and hydric dilation discussed above come from laboratory experiments where demineralised water is usually used. In masonries made from dimension stones a variation in the salt concentration can often be found, which has a significant effect on the hygric/hydric dilatation, and therefore on the swelling pressure being formed. In comparison to pure water, the presence of an aqueous solution with a higher concentration of salt results in a lower swelling ability. However, if a salt is present a residual expansion can be observed (e.g. Wendler and Rückert-Thümling 1992; for further explanation see Chap. 4).

3.6 Strength Properties

The strength of materials is one of the basic parameters used to classify the quality of natural stones. It is one of the most important and reliable methods for classifying rocks and its application for construction purposes. The results can be directly used for the structural requirements considering an acceptable safety margin. However, the mechanical strength is often mainly controlled by the heterogeneity of the rocks and its fabrics instead of the individual properties of the rock-forming minerals. The strength values refer to the material's ability to withstand an applied stress without failure. Yield strength refers to the point on the engineering stress-strain curve (as opposed to the true stress-strain curve) beyond which the material begins deformation that cannot be reversed upon removal of the loading. Ultimate strength refers to the point on the engineering stress-strain curve corresponding to the maximum stress. The applied stress may be tensile, compressive or shear. Is the stress limit reached material failure occurs.

A material's strength depends on the rock fabrics and is influenced by its composition, the shape and size of the material sample and by the aging of the material and its storage conditions. The engineering processes to which a material is subjected can alter these microstructures. The variety of strengthening mechanisms that alter the strength of a material includes work-hardening, solid-solution strengthening, precipitation hardening and grain boundary strengthening and can be quantified and qualitatively explained. However, strengthening mechanisms are accompanied by the contradiction that some mechanical properties of the material may degenerate in an attempt to make the material stronger.

Strength is considered in terms of compressive strength, tensile strength, and shear strength, namely the limit states of compressive stress, tensile stress and shear stress, respectively. The effects of dynamic loading are probably the most important practical part of the strength of materials, especially the problem of fatigue. Repeated loading often initiates brittle cracks, which grow slowly until failure occurs.

However, the term strength of materials most often refers to various methods of calculating stresses in structural members, such as beams, columns and shafts. The methods that can be employed to predict the response of a structure under loading and its susceptibility to various failure modes may take into account various properties of the materials other than material (yield or ultimate) strength. For example, failure in buckling is dependent on material stiffness (Young's modulus).

3.6.1 Compressive Strength

Demands on compressive strength are made in cases where through a material a planar load to the subsurface ground has to be ensured. During an unconfined uniaxial pressure experiment, the test specimen that experiences longitudinal stress will be shortened until failure and final break (Fig. 3.50). The unconfined comprehensive strength (UCS) of a dry specimen expresses the value where the sample experiences total loss of cohesion along the fracture surface. A typical stress-strain curve is shown in Fig. 3.51. At the beginning of the experiment a consolidation of the specimen can usually be observed.

During this phase open pre-existing microcracks are being closed, preferably those that are oriented perpendicular to the loading axis. Through this the test specimen experiences an irreversible shortening but with relatively small values. At about 30% to 70% of the compressive strength value, the rock sample shows a nearly linear

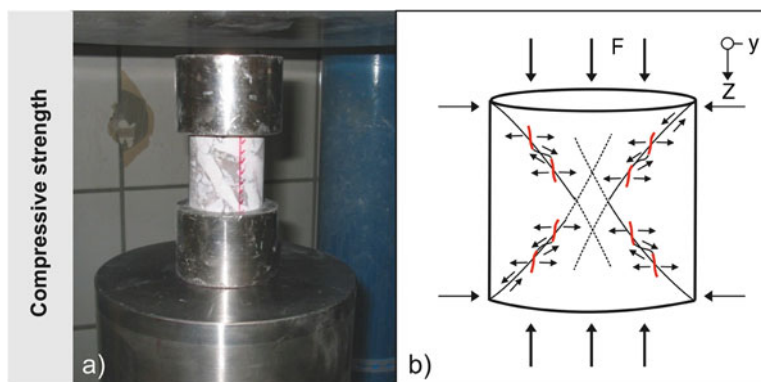


Fig. 3.50 Experimental setup (a) for determining the uniaxial compressive strength. (b) Rock failures occur through a combination of tensile and shear cracks. F = loading force

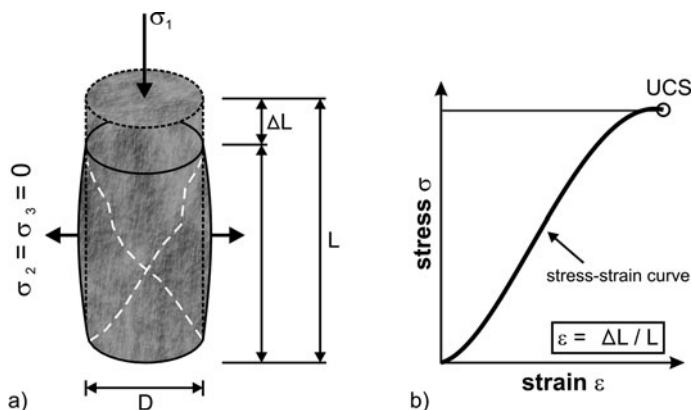


Fig. 3.51 Unconfined compressive strength (UCS) test: (a) a core with failure under unconfined compression, (b) a stress-strain curve where the UCS marks the failure of the specimen (after Thuro et al. 2001)

elastic behaviour. Beyond that an increase in microcrack formation occurs that ultimately leads to total material failure. The degree of consolidation depends strongly on the density of the open microcracks, whereas the latter linear elastic behaviour is mainly determined by the mineral composition and its elastic properties.

The question where new microcracks are formed in a test specimen depends on the one hand, on the geometry and density of the pre-existing microcracks, and on the other hand, on the mechanical stability of the minerals and the fabric-dependent interaction between mechanically weak and strong minerals (Segall and Pollard 1980, Dürrast et al. 2001).

For certain applications of natural dimension stones minimal values for the compressive strength are requested. This requirement shows that the experimental setups and specimen involved in the determination of the compressive strength must be comparable and equivalent. The uniaxial compressive strength is the most common performance measure used by engineers for the quality assessment of rocks. For the experimental compressive strength tests, several countries have standards for these tests (Germany, France, UK, USA, etc.), while in Europe the introduction of the European Standards may help to overcome the different approaches in use. Compressive strength is measured by tracking cylindrical, cubic or prismatic samples in a compression testing machine. Usually the compressive load is applied by a servo-hydraulic testing machine with a very stiff testing frame and a class 1 load range up to 300 kN. Most critical is the sample preparation, the sample dimension and the quality of the end-faces, which must be co-planar with an accuracy of 0.1%. The load is applied to the end-faces of the specimen with a strain rate of 10^{-5} s^{-1} until failure. The maximum load is defined as the uniaxial compressive strength. End-faces with a lower quality cause local stress and may result in a strength loss and low-quality data. The standards allow a large variation with respect to the samples sizes, while in geotechnical approaches standard specimens of a ratio >2 (length/diameter) are believed to represent the “true” UCS value. Bieniawski

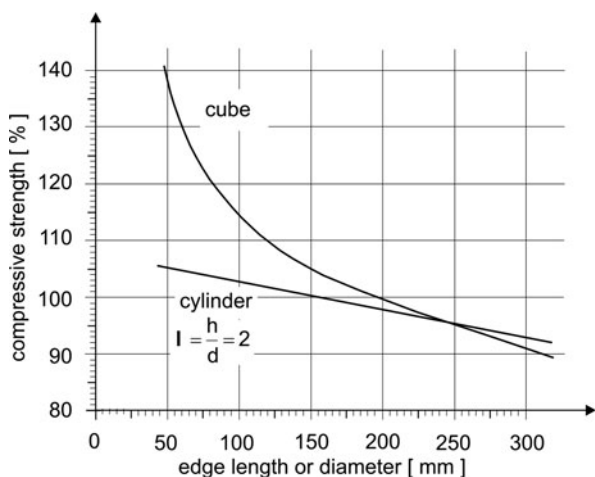


Fig. 3.52 Effect of the geometric shape of a test specimen (here a cylinder and cube) on its compressive strength (after Wesche 1981)

(1967) had shown that the compressive strength may differ up to 20% by using a length/diameter ratio of 1.

Following the DIN EN 1926 (1999), it is still possible to determine the compressive strength by using cubes with an edge length of 50 mm or cylindrical samples with diameter and height of 50 mm. The compressive strength determined with cylindrical samples particularly depends on the length-diameter-ratio (Fig. 3.52). However, with a length-diameter-ratio higher than 2.5–3 the influence of the specimen geometry is negligible (Wesche 1981). A still unresolved and discussed issue is the drying of the sample. On the one hand, it could mean that drying is carried out at 70°C until a constant mass has been reached, on the other hand, a not yet defined air dry state could be outlined. Rohowsky (2001) found that the compressive strength values determined following the DIN EN 1926 (1999) were above and below the values determined by following the DIN 52105 (1988). No systematic trend emerged. Experiments using the European standard showed an increase in the compressive strength values for samples with lower bulk density values and a decrease for samples with higher bulk densities.

A classification of hard rocks in relation to their compressive strength, which is mainly designed for engineering purposes, indicates and separates for dimension stones five (De Quervain 1967) to eight classes (Peschel 1974). In general, the following discriminations are made: hard rock with more than 110 MPa, medium hard rocks between 70 MPa and 110 MPa and weak rocks between 55 MPa and 70 MPa. Explorative data evaluation based on statistical characteristics with the construction of box plots was given by Mosch and Siegesmund (2007). The median value used has an advantage in comparison to the arithmetic mean as it is more robust towards statistical outliers, which means it is less affected by extreme values (Kürzl

1988). The relative position of the median value in the box also provides an impression about the skewness or symmetry of the distribution based on the data used. The boundaries of the box are the 25% and 75% quantile value. The box itself is designed to be independent from extreme values and provides a resistant dispersion measure of a given data set. The delineation of the horizontal lines, called “whiskers”, depends on various parameters. Usually, as also applied here, the length of the whiskers is a maximal 1.5 times of the interquantile distance (R Development Core Team 2005), whereas the end values are determined from values of the data set; values below and above the end values are outliers.

Mosch and Siegesmund (2007) found that the uniaxial compressive strength (e.g. Table 3.3 and Fig. 3.53) of plutonic rocks varies between 60 MPa and 292 MPa, a range that covers the majority of the published UCS values for plutonic dimension stones (Peschel 1983, Winkler 1994, Müller 2001). A much higher value with 427.7 MPa is given for the gabbro “African Blue” from South Africa; this value may be an extreme outlier and needs to be questioned. However, Morales et al. (2010) found in a more systematic study on dolerites from Uruguay (on rocks with commercial names such as Negro Absolut and Negro Oriental) compressive strength values from 400 MPa to 265 MPa. Volcanic rocks may also show a distinct heterogeneity in regards to the observed compressive strengths. They range between 50 MPa and 300 MPa. Rhyolites, dacites, andesites, and basalts have values that can be comparable. Trachytes and especially tuffs show lower values.

For the group of carbonate rocks, the UCS values vary between 4.4 MPa and 265 MPa. This range also covers the majority of the published UCS values for this rock group. Sandstones show a variation from 10.0 MPa to 257 MPa, while the highest value is related to a conglomerate from Brazil (Marinace Green). Metamorphic rocks show the highest variation with respect to the uniaxial compressive strength (see Fig. 3.53, Table 3.3).

The uniaxial compressive strength can show a strong directional dependence when determined in different directions of one rock. Moreover, the rock fabric elements, especially the lattice- and shape-preferred orientation of minerals as well as the degree of grain interlocking (i.e. grain boundary configuration), has been intensively investigated with respect to their control on petrophysical properties (e.g. Wenk 1985, Howarth and Rowlands 1987, Siegesmund and Dahms 1994, Siegesmund 1996, Brosch et al. 2000). In particular, metamorphic rocks like gneisses show a strong directional dependency of the compressive strength values. Strohmeyer and Siegesmund (2002) carried out a systematic study on a progressively deformed rock sequence, i.e. starting with a more or less undeformed granitoid and ending with a strongly deformed mylonite; this goes along with an increase in the intensity of the foliation. The sample sequence had a more or less similar chemical and mineralogical composition. The uniaxial compressive strength of the samples ranges between 128 MPa and 225 MPa. The development of a tightly spaced mylonitic foliation and in particular the preferred orientation of the micas may significantly control the anisotropic behaviour of the compressive strength. Micas show a perfect cleavage parallel to their (001) planes and often exhibit a strong preferred orientation in those deformed rocks. They are the weakest

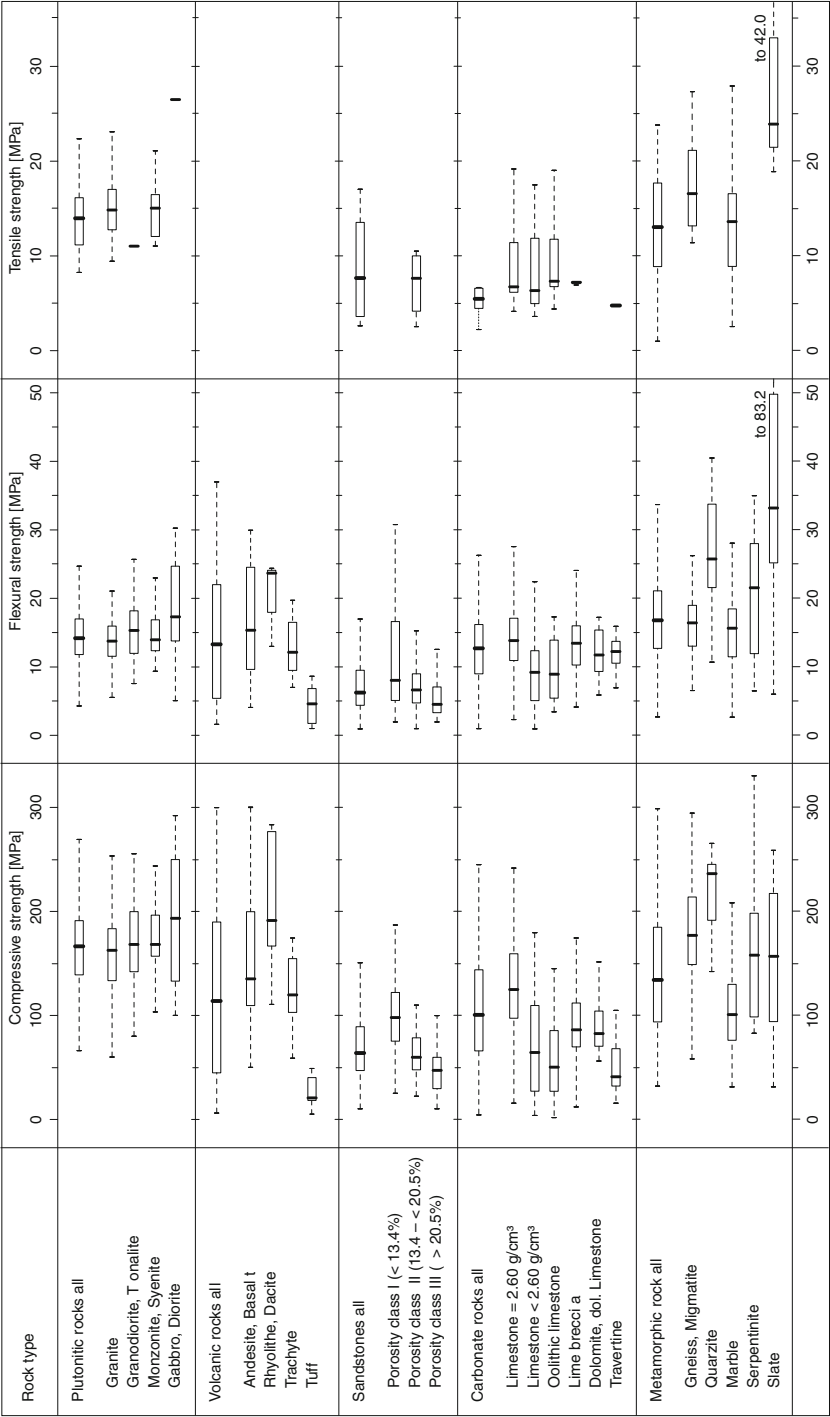


Fig. 3.53 Typical lithological value ranges with regard to the mechanical strength parameters (e.g. compressive, flexural, and tensile splitting strength) put together on the basis of a statistical analysis of natural stones. The boxplots describe the corresponding median value, the 25% and 75% quartile value as well as the upper and lower extreme value (after Mosch and Siegesmund 2007)

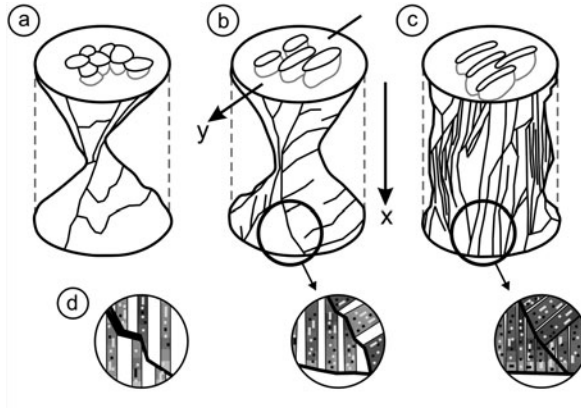


Fig. 3.54 Crack and failure pattern parallel to the x-direction during the UCS experiments in relation to the rock fabrics: (a) proto mylonite, (b) mylonite, and (c) ultramylonite; (d) shows the crack propagation during the experiment with the arrows indicating the displacement (after Strohmeyer and Siegesmund 2002)

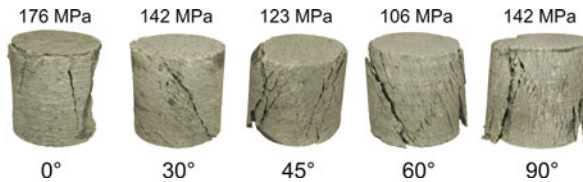


Fig. 3.55 UCS experiment on rock samples of a foliated gneiss (Verde Andeer) with values given: load perpendicular to the foliation ($0 = 176$ MPa), load 30° to the foliation (142 MPa), load 45° to the foliation (123 MPa), load 60° to the foliation (106 MPa) and load parallel to the foliation (142 MPa)

component in the rock with respect to their strength (as observed by Muegges 1898). Most importantly, the failure pattern also varies with regards to the mylonitisation, from a cone-shaped fracture pattern to a roof-shaped form (Fig. 3.54).

A more detailed study on a sample of the Verde Andeer Gneiss was performed where different load directions with respect to the foliation were tested (Fig. 3.55). It is well known that for foliated rocks the uniaxial compressive strength is highest in the loading direction normal to the foliation (e.g. de Quervain 1967, Brosch et al. 2000, Strohmeyer and Siegesmund 2002) and lowest at acute angles to these planes. The compressive strength values are controlled by the grain size, closely spaced foliation, mineralogical changes with respect to the layering etc. All these superimposed fabrics (see also Chap. 2) and especially the quantity and spatial arrangement of discontinuities forming zones of weakness characterise the resistance of rock to compressive fracture.

Hoffmann and Siegesmund (2007) reported an anisotropy value of 47% for a marble from Thailand (Phran Kratai Grey). The relatively low UCS values of crystalline marbles are related to the almost mono-mineralogical composition of this

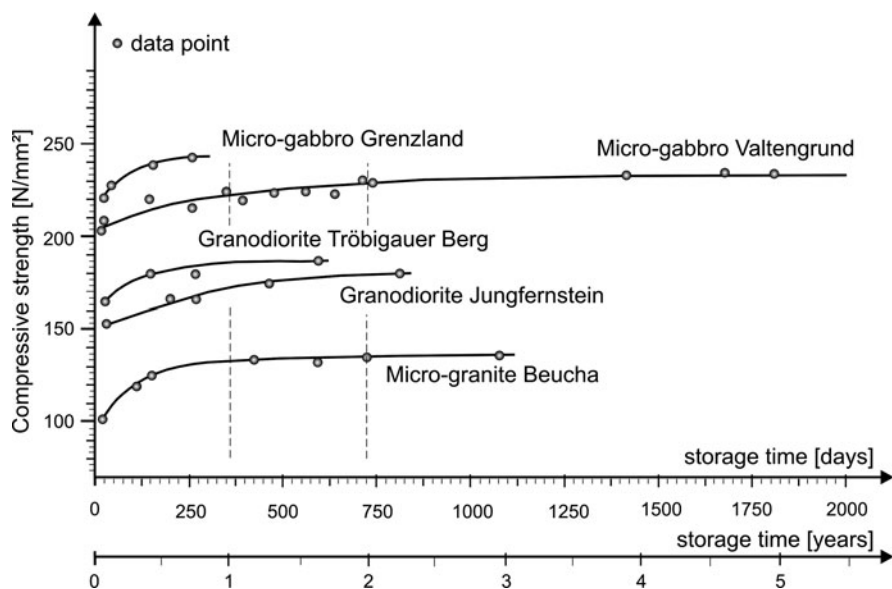


Fig. 3.56 Change in the compressive strength in relation to the aging of the test specimen (storage time between sample preparation and experiment), after Peschel (1974)

rock and the grain texture. Compressive strength values of Carrara marbles with polygonal grain boundaries (ca. 109 MPa) can be lower than those of Carrara marbles with stronger interlocked and interlobate grain boundaries (ca. 135 MPa). Strongly weathered marbles with polygonal textures can also show relatively low UCS values with about 40 MPa. Next to metamorphic rocks, dimension stones with pronounced parallel textural elements can also show a directional dependence of the uniaxial compressive strength, for example sandstones and carbonates with distinct layering feature (see Fig. 3.53).

Another factor influencing the compressive strength is the so-called “aging” of the testing sample. Peschel (1974) investigated a series of plutonic rocks and found that the compressive strength increases with time (Fig. 3.56). Freshly quarried stones first reached their maximum values in approximately 1.5–2 years and remain relatively constant after that. This can be explained by the relaxation reaction phenomenon of the rocks after they have been removed from the quarry and exposed to changing environmental conditions (pressure, temperature and moisture). The locked-in stress, which is related to the all-enclosing stress field of a mountain massif, can be lost by aging after the removal of the block from its surrounding.

Another problem is represented by the residual moisture, which is the result of long-term storage and progressive desiccation in the block (Greger 1930). Lentschig (1971) reported that the compressive strength of water-saturated magmatic and metamorphic rocks is lower than for dry samples, ranging 3–15% depending on the water absorption capacity. For different sandstones, Morales et al. (2007) could show that increasing moisture content strongly affects the uniaxial compressive rock

strength. However, the degree of rock strength reduction differs over a wide range. Morales et al. (2007) demonstrated that the effect of partial water saturation on rock strength is primarily related to the pore radii distribution and the matrix mineralogy. A reduction of the strength up to 50% has been observed. The results are partially in accordance with the conclusion of Hawkins and McConnell (1992), who had proposed that the degree of sensitivity to moisture content is controlled primarily by the proportion of quartz and clay minerals present, and to a lesser extent by the rock microfabrics.

The modulus of elasticity, which is usually quoted as Young's modulus (E), is based on the relationship between stress and strain. This value is also known as the static modulus obtained by the tangent, secant, or average method (Schön 1996). The so called tangent modulus refers to the 50%-value of the uniaxial strength, the secant modulus is taken as the gradient of the ultimate uniaxial compressive strength on the stress-strain curve from the origin to the 50%-value, and the average modulus represents an average slope of the straighter portion of the stress-strain curve.

Comparable to the compressive strength (Table 3.3) the main factors that have an influence on the static E -modulus are the mechanical properties of the minerals and the size of the grains. Török (2007) had compiled the modulus of elasticity and the compressive strength for some magmatic rock, which shows the general relationship between these two parameters (Fig. 3.57). However, these parameters not necessarily correlate with the dynamic E -modulus. Additionally, open microcracks are an important influencing parameter. During a uniaxial pressure experiment a rock specimen experiences elastic as well as additionally always plastic deformation. The latter one is not expected to occur during the determination of the dynamic E -modulus, as the time periods for the specimen under loading stress are relatively short (Schön 1983). Because of this the value of the dynamic E -modulus should be always higher than the one for the static E -modulus. After Schön (1983) this effect, respectively, the gap between both values, will increase with increasing porosity and fracturing of the rock sample.

3.6.2 Tensile Strength

The tensile strength of a material is the maximum amount of tensile stress that it can be subjected to before failure. Because this property does not compare well to other rock mechanical parameters it is not often measured on natural stones. In addition, its usability and relevancy is often not clear to many architects and others working with dimension stones. However, its understanding and determination is essential when referring to a complete rock mechanical concept of geological materials.

Tensile strength testing of rocks can be performed by direct tension tests (i.e. uniaxial or triaxial). The parallel end-faces of a sample cylinder, with a defined cross-sectional area, are typically glued onto the testing machine pistons. It is then pulled with a controlled, gradually increasing force until the sample breaks, or indirect tests are applied (e.g. Brazilian test, bending test, fracture toughness test).

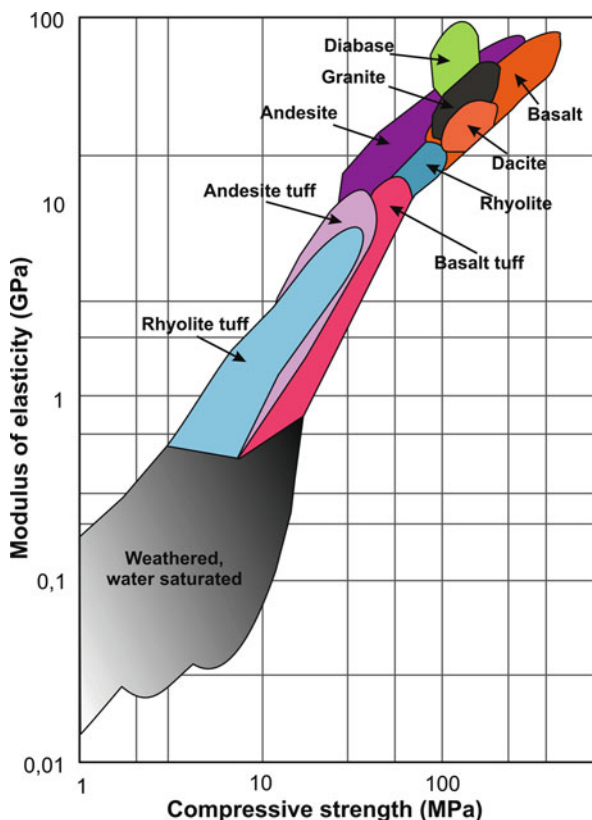


Fig. 3.57 Relationship between the modulus of elasticity and the compressive strength of selected rock types (after Török 2007)

The most usual testing procedure for the determination of the tensile failure strength (σ_z) is the Brazilian test (or indirect tensile strength test). A disc-shaped rock specimen is subjected to a compressional load in a uniaxial test setup, which results in a largely homogenous tensile stress field. This is a simple, relatively inexpensive, and fully standardized technique, which is well described by several standards, e.g. ASTM or ISRM.

The cylindrical test specimen (typically 20 mm or 40 mm in length with a diameter of 40 mm, following the recommendations of the various standards) is loaded diametrically across the circular cross section, where tensile deformation results perpendicular to the loading direction (see Fig. 3.58). Tensile failure occurs when the tensile strength of the experiment reaches the value of the rock tensile strength, σ_z . From the vertical load F and the specimen dimensions (see Fig. 3.58), the tensile strength can be calculated by the following equation:

$$\sigma_z = (2 \times F)/(d \times 1 \times \pi) \quad (3.16)$$

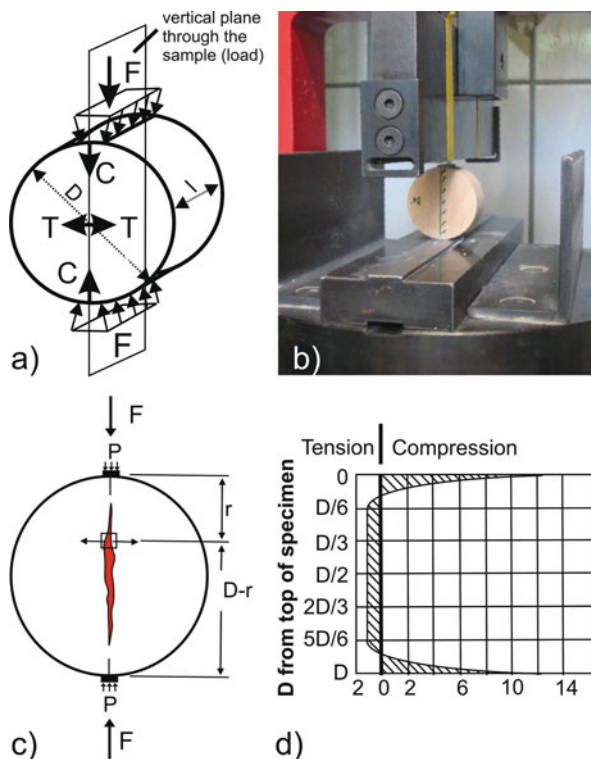


Fig. 3.58 Tensile strength test or Brazil method: (a) schematic diagram and (b) photo of the experimental setup, (c) schematic representation of the crack opening due to the tensile force, and (d) stress distribution inside a cylindrical test specimen under uniaxial load; abbreviations: C = direction of compression, T = direction of tension, P = compressional stress, F = loading force, D = diameter, l = length

with σ_z the tensile stress at failure (MPa), F the loading force (N), l the length of the specimen (mm), and d the diameter of the specimen (mm).

The tensile strength value of a rock results from the average of at least four tests using specimens prepared close to each other; usually the standard deviation should be given (e.g. Thuro et al. 2001). However, the tensional stress conditions within the sample are not homogeneously realized. Therefore, the tensile strength values determined by indirect methods such as the Brazilian test method do not reflect “true” tensile values. Peschel (1977) has shown that the Brazilian test and the “true” tensile strength data exhibit an almost constant ratio of 2:1.

To illustrate the tensile strength for selected lithologies existing experimental data sets derived on various rocks were compiled. Figure 3.53 and Table 3.9 show the tensile strength data of different rocks in respect to different measuring directions. The tensile strength data shows that the median value of the gneiss/migmatite group is with ca. 15 MPa slightly above the value of the marble data set (median value = ca. 12 MPa), which shows a wider scattering range. Magmatic rocks show

Table 3.9 Compressive, flexural, tensile, and abrasion strength and static Young's modulus for various dimension stones with commercial name, country of origin, and rock type given. Numbers in brackets are the anisotropic values

Commercial name	Country of origin	Rock type	Compressive strength (MPa)	Flexural strength (MPa)	Tensile strength (MPa)	Abrasion strength (cm ³ /50 cm ²)	Young's modulus (static) (GPa)
Ben Tak White	Thailand	Granite	183.7 (14.0)	19.7 (19.7)	10.6 (14.4)	4.7 (6.8)	28.0 (17.9)
Muang Tak	Thailand	Granite	184.8 (25.8)	16.9 (40.5)	8.9 (29.2)	4.0 (7.2)	28.1 (14.8)
Orange 1							
Kösseine	Germany	Granite	195.0 (2.6)	20.1 (20.3)	11.9 (12.7)	5.7 (2.8)	27.5 (17.8)
Antrona	Italy	Granite	208.0 (6.5)		12.4 (20.3)	7.4 (10.7)	15.9
Rojo Dragon	Argentina	Granite	194.4 (1.3)	13.3 (13.3)	6.5 (10.8)	4.2 (4.0)	32.5
Padang	China	Granodiorite	222.0 (6.5)	21.0 (11.9)	12.8 (8.9)	7.1 (3.6)	30.6 (19.3)
Ben Tak Blue	Thailand	Quartz-monzonite	174.5 (4.0)	28.3 (15.0)	13.8 (14.9)	8.4 (11.9)	28.5 (7.2)
Salmon Red	Uruguay	Syenite		14.38 (13.36)	8.9 (3.5)	2.81 (21.4)	
Artigas	Uruguay	Syenite			5.76 (20.6)	3.41 (4.1)	
Nero Absoluto	Uruguay	Dolerite	358.1 (2.9)	49.72 (16.4)	17.87 (15.7)	2.25 (10.6)	27.1 (35.6)
Anzola	Italy	Gabbro	208.0 (16.1)	14.9 (43.1)	9.6 (27.6)	7.8 (20.7)	28.5 (23.6)
Nero Impala	South Africa	Gabbro-norite	235.0 (14.5)	17.1 (5.9)	10.7 (8.0)	6.8 (12.9)	32.8 (35.3)
Negro Riojano	Argentina	Gabbro	200.2 (6.7)	24.6 (3.8)	10.9 (1.7)	3.96 (0.3)	
Balmuccia	Italy	Peridotite	243.0 (9.4)	24.6 (29.7)	13.1 (10.0)	5.7 (10.5)	37.7 (17.4)
Löbejün	Germany	Rhyolite	188.5	19.3 (17.6)	10.3 (31.2)	5.0	
Stardust Grey	Argentina	Rhyolite	207.4 (3.7)	21.5 (29.3)	10.3 (12.9)	7.3 (63.5)	29.6
Monte Merlo	Italy	Trachyte					
Drachenfels	Germany	Trachyte					
Weibern	Germany	Phonolite tuff	11.36		1.6		4.8
Rochlitz	Germany	Rhyolite tuff	23.6	2.7	3.8 (28.9)	11.8	16.7
Phanom	Thailand	Gneiss	194.6 (4.4)	19.1 (20.5)	10.9 (18.9)	4.5 (8.7)	29.2 (5.2)
Sarakham							
Grey							
Azul Tango	Argentina	Gneiss	212.3 (12.2)	23.5 (2.0)	9.7 (29.8)	3.9 (10.5)	36.6
Calanca	Italy	Gneiss	186.0 (22.3)	15.4 (73.4)	9.3 (54.8)	8.1 (15.9)	25.0 (10.8)

Table 3.9 (continued)

Commercial name	Country of origin	Rock type	Compressive strength (MPa)	Flexural strength (MPa)	Tensile strength (MPa)	Abrasion strength (cm ³ /50 cm ²)	Young's modulus (static) (GPa)
Verde Andeer	Italy	Gneiss	176.0 (28.0)	15.3 (72.6)	11.4 (52.8)	7.4 (22.9)	22.1 (7.9)
Serizzo Monte Roas	Italy	Gneiss	169.0 (8.0)		9.8 (54.1)	7.9 (13.7)	20.7 (6.5)
Wang Nam Kiew Black	Thailand	Hornblendite	125.5 (8.8)	11.5 (32.5)	6.8 (40.2)	7.1 (14.8)	28.7 (24.6)
Franco Veteado	Argentina	Migmatite	112.1 (16.0)	11.9 (17.3)	8.2 (16.6)		23.3
Rosa Estremoz	Portugal	Marble	81.2 (18.5)	16.4 (31.0)	6.8 (20.0)	24.9 (7.4)	16.4
Carrara	Italy	Marbe			4.5 (48.8)		
Carrara	Italy	Marble					
Phran Kratai	Thailand	Marble	122.1 (46.8)	25.3 (59.9)	8.9 (44.4)	15.0 (13.0)	30.4 (9.1)
Grey							
Azul Cielo	Argentina	Marble	50.2 (2.7)	5.9 (24.7)	3.5 (7.7)		30.8
Azul Imperial	Brazil	Muartzite	281.0 (5.8)	28.4 (41.9)	18.3 (21.9)	4.3 (8.5)	34.2 (14.3)
Camine Rulo	Uruguay	Slate			13.54 (93.1)	11.25 (19.5)	
San Luis Slate	Argentina	Slate	172.4 (33.8)	37.5 (99.3)	13.4 (62.7)	13.8 (67.2)	37.6
Theuma	Germany	Slate	89.7	3.3		31.3	
Globegerina	Malta	Limestone	26.4 (94.4)	1.1–4.7	3.03 (17.5)		21.0
Caliza Amarilla	Argentina	Limestone	84.0 (19.6)	11.4 (13.3)	4.5 (21.0)	24.8 (24.0)	
Dolomita Dorada	Argentina	Dolomitic limestone	194.1 (11.9)	21.3 (22.8)	10.2 (7.6)	21.1 (3.4)	
Mae Phrik Yellow	Thailand	Limestone	125.3 (6.4)	14.7 (11.0)	8.5 (18.9)	9.9 (9.1)	29.3 (23.3)
Soskut	Hungary	Limestone					
Kuacker	Germany	Limestone	82.5	12.6	5.5	23.8	
Thüster	Germany	Limestone	25.0		7.0		
Bad Langensalza	Germany	Travertine	55.0 (52)	7.9 (10.8)	5.6 (34.6)	22.3	47.2 (10.2)
Baumberger	Germany	Sandstone-like limestone	50.0	11.0	4.1	19.1	19

Table 3.9 (continued)

Commercial name	Country of origin	Rock type	Compressive strength (MPa)	Flexural strength (MPa)	Tensile strength (MPa)	Abrasion strength (cm ³ /50 cm ²)	Young's modulus (static) (GPa)
Anröchte	Germany	Sandstone-like limestone	85.0	21.9		19.8	
Bad Bentheim	Germany	Sandstone	50.3	4.2	2.8	16.4	
Schleierthor	Germany	Sandstone	76.8 (17.8)		6.8		
Weser	Germany	Sandstone	149.8 (5.5)	21.3	8.4	12.3	
Sikhiu Brown	Thailand	Sandstone	147.5 (20.5)	16.7 (35.5)	8.7 (7.7)	9.9 (9.6)	18.4 (5.1)
Pakchong Green	Thailand	Sandstone	103.2 (20.0)	10.7 (39.2)	5.7 (25.2)	12.9 (28.3)	13.5 (19.1)
Tacuarembó	Uruguay	Sandstone	32.7 (28)	4.07 (16.6)	2.4 (56.6)	56.4 (24.2)	5.3 (50)

tensile strength values of 9 MPa to ca. 27 MPa, which represent the group with the highest strength (Mosch and Siegesmund 2007). The number of data available for carbonate rocks is quite low, but the known strength range varies between 3.6 MPa and 19.1 MPa. The sandstone group exhibits similar characteristics also with comparable values. However, the fresh Solling Sandstone shows the highest tensile strength value due to its comparable high density and an interlocked matrix of the rock-forming grains (bulk density 2.34 g/cm^3 ; see Tables 3.1 and 3.9 (Ruedrich and Siegesmund 2007).

The samples and data shown in Fig. 3.59 providing a good example to illustrate how assumed similar rocks, following their appearance, have significantly different technical properties. The Schilf Sandstone samples shown exhibit a good correlation between porosity and tensile strength values. Choosing a dimension stone only by considering visual and aesthetic aspects might result in having rocks with quite different properties, for example, resulting in different weathering behavior.

One of the main factors affecting the tensile strength is the existence and spatial orientation of any foliation or sedimentary layering in the rock sample and the loading direction in respect to these rock fabric elements. In general, many rocks show the lowest tensile strength values perpendicular to the foliation or sedimentary layering. For sandstones relatively high anisotropy values of up to 40% related to the occurrence of sedimentary layering have been reported by Koch and Siegesmund (2001, 2005), Hoffmann and Siegesmund (2007), and Ruedrich und Siegesmund (2007).

The geometry of tension cracks and fractures in relation to the pre-existing rock fabric (e.g. intergranular cracks along grain boundaries and intragranular cracks in

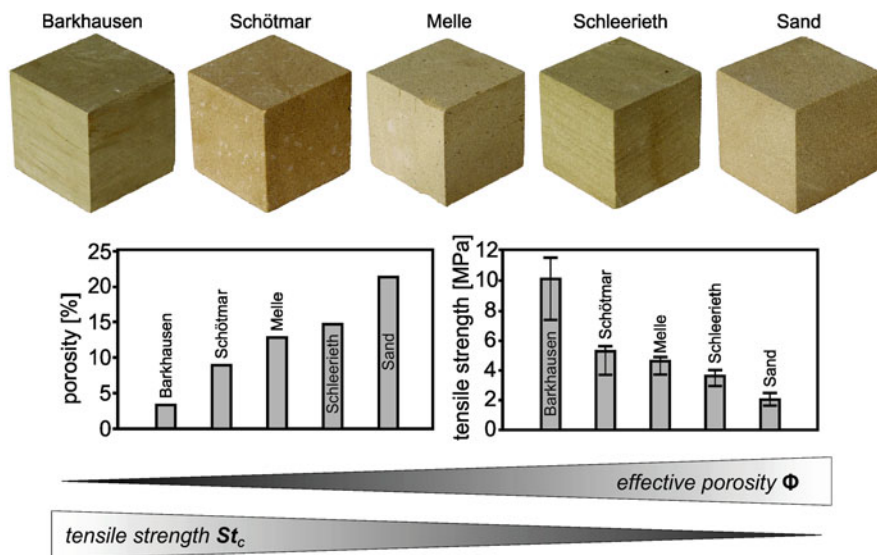


Fig. 3.59 Five different samples of the Schilf Sandstone (Middle Keuper) shown here are similar in color and structure, but their porosity and tensile strength values vary significantly (Mosch 2009)

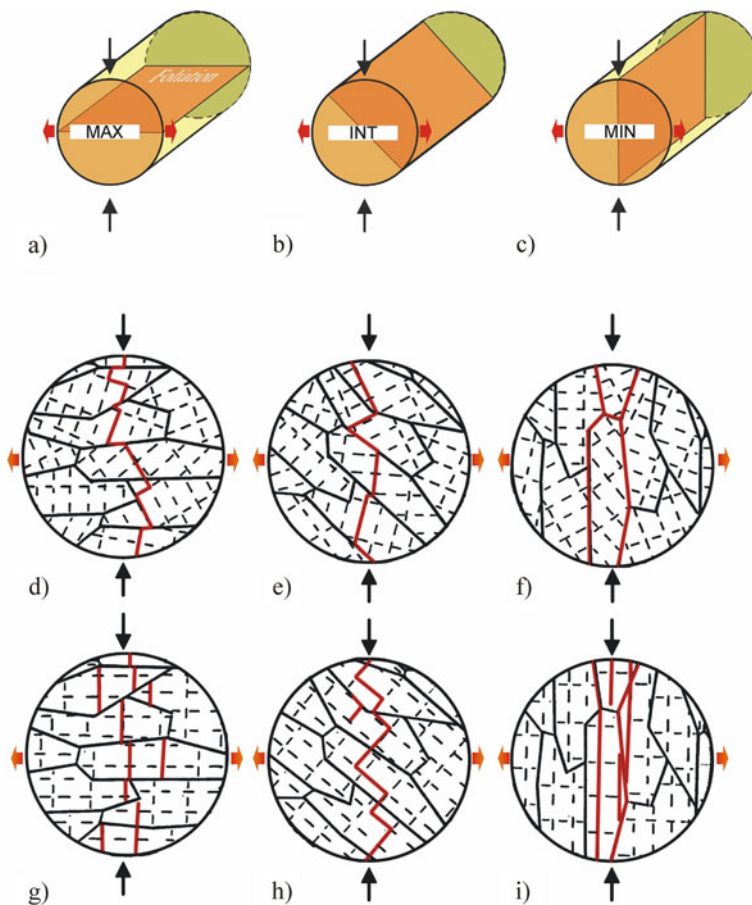


Fig. 3.60 Anisotropy of the tensile strength in relationship to the rock fabric and the orientation of the foliation with (a) maximal (MAX), (b) intermediate (INT), and (c) minimal (MIN) value of the tensile strength. (d–f) Rock fabric without any preferred mineral orientation (schematically shown by the orientation of cleavage planes) but with a clear preferred orientation of the grain shape. Depending on the orientation of the minerals in relation to the loading direction (black arrows) the sample would show higher (a, d) and lower (c, f) tensile strength values, i.e. the tensile strength is controlled by the grain shape. (g–i) Rock fabric with a clear preferred orientation of the minerals (schematically shown by the orientation of cleavage planes) and a clear preferred orientation of the grain shape. The tensile strength is controlled by the combination of the orientation of the minerals (cleavage planes) and the orientation of the grain shapes in relation to the loading direction (black arrows), with higher (a, g) and lower (c, i) tensile strength values. For all possibilities (d–i) the crack distribution is shown in red as the result of the tensile forces (red arrows) after Siegesmund et al. (2002)

the rock-forming minerals) are schematically shown as 2D-models in Fig. 3.60. Two different fabric types are compared, which have a different angle to the loading direction (indicated by the arrows). (1) The fabric in the upper row (Fig. 3.60d–f) shows a distinct and clear preferred orientation with respect to the grain shape, but no crystallographic preferred orientation. The tensile fracture orientation here is

mainly controlled by the orientation of the intragranular cracks. (2) In the lower row (Fig. 3.60 g–i), the rock has a preferred orientation with respect to the grain shape, as (1), and additionally a clear and distinct crystallographic preferred orientation of the rock-forming minerals, indicated by two cleavage planes oriented perpendicular to each other. One of these planes is oriented nearly parallel to the grain boundaries.

The six different test types illustrated in Fig. 3.60 showing the two different fabrics and the three different loading geometries result in extreme differences in the values for the tensile strength, with the maximal value for the fabric type in the upper left and the minimal value for the fabric type in the lower right. The maximal value results from the situation where the grain boundary cracks do not have the potential to become tensile cracks, since they are oriented perpendicular to the loading force. The minimal tensile strength value is the result of a fabric where the grain boundary cracks and the cleavage cracks are oriented parallel to the loading direction, and therefore both have the potential to become tensile cracks.

For a marble sample Siegesmund et al. (1999) observed a variation in the tensile strength from 5.5 MPa to 8.3 MPa. Related to this, Ruedrich (2003) has done detailed investigations on the Carrara and Sterzinger Marble. He found that the tensile strength values vary depending on the load direction, between 3.4 MPa and 5.6 MPa, and between 4.8 MPa and 7.9 MPa, respectively. His results support the concept of having a superposition of texture and load geometrical effects. As shown in Fig. 3.60, small tensile cracks were formed along the grain boundaries in the fine-grained Carrara Marble, which successively connect to each other thereby forming a growing network. In addition, Peck et al. (1985) and Brosch et al. (2000) could also demonstrate that open microcracks in rocks have a significant effect on the rock's tensile strength. For example, the Sterzinger Marble exhibits a clear interlocking of the grain boundaries, however the tensile forces induced by the uniaxial loading activated the crystallographic pre-existing intercrystalline microcracks in the rock.

Similar fabric types can be found in many metamorphic rocks as well as magmatic rocks, which are characterized by a distinct magmatic flow direction (see Chap. 2). Strohmeyer and Siegesmund (2002) found a variation of the tensile strength values, from ca. 7 MPa to 14 MPa, related to differences in the rock fabrics for a metagabbro from Anzola, where the hornblende shows a perfect cleavage. However, it is not clear if this phenomenon is superimposed by the orientation of the grain boundary cracks. On the other hand, the pyroxene, here a diopside, shows a much less developed cleavage.

Sandstone samples investigated by Koch and Siegesmund (2005) and Ruedrich und Siegesmund (2007) also show a significant directional dependence of the tensile strength related to the loading direction. Figure 3.61 shows several samples with an increasing degree of the preferred orientation of the grains and their tensile strength values in different directions. The anisotropy of the tensile strength increases with an increasing degree of the preferred orientation of the grains, with the Arholzener Sandstone shown having a value of 33%. Further, a sample of the Solling Sandstone, which is commonly used as a building stone, revealed a 33% higher tensile strength parallel to the sedimentary layering (4.5 MPa) than perpendicular to the layering (3.0 MPa).

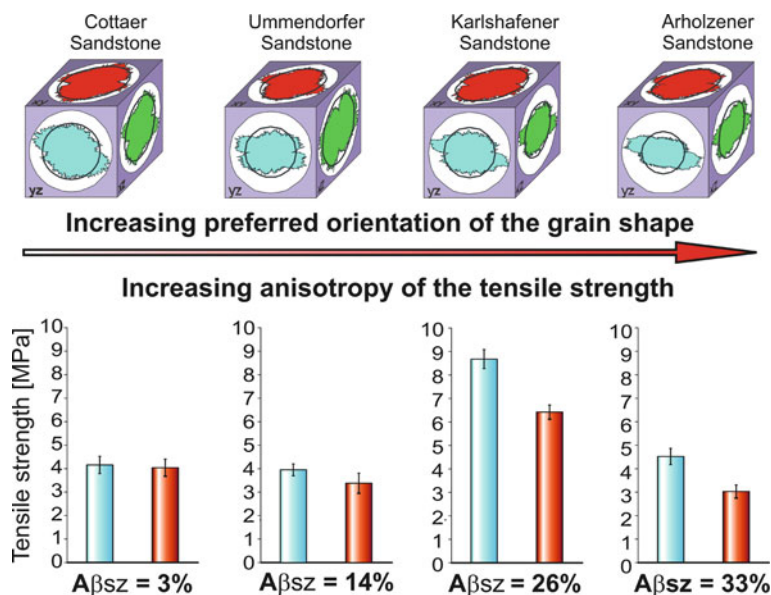


Fig. 3.61 Relationship between the preferred orientations of the grains related to grain shape and the anisotropy of the tensile strengths for different sandstones. The Arholzener Sandstone shows a strong preferred orientation of the grain shape and a strong anisotropy (modified after Koch and Siegesmund 2005)

Aiming at general correlations between various rock properties, Fig. 3.62 shows the relationship between the uniaxial compressive strength (using cylindrical samples) and the tensile strength for various rocks. Although a clear linear dependency for the various lithologies seems to be obvious, some restrictions have to be considered.

Peschel (1974) tested in an extensive experiment a large number of specimens from different rock types. Based on these data he postulated a nearly constant value of 1:10 for the ratio of compressive to tensile strength for the whole spectrum of dimension stones. However, in 1912 Hirschwald already published a nearly constant compressive/tensile strength ratio of 1:10–15. Strohmeyer (2003) found that when using the mean values of the compressive and tensile strength this ratio might hold. However, when taking anisotropy for both properties into account the ratio varies significantly. Then, on average the ratio between the compressive and tensile strength is 1:18, for maximal values of extreme anisotropic rocks 1:39, and for minimal values 1:10.

Finally, taking into account the uncertainties of such empirical relationships it is strongly suggested that if tensional strength values are required for any reason, e.g. for mechanical strength calculations, individual experimental investigations on the respective rock material should be performed.

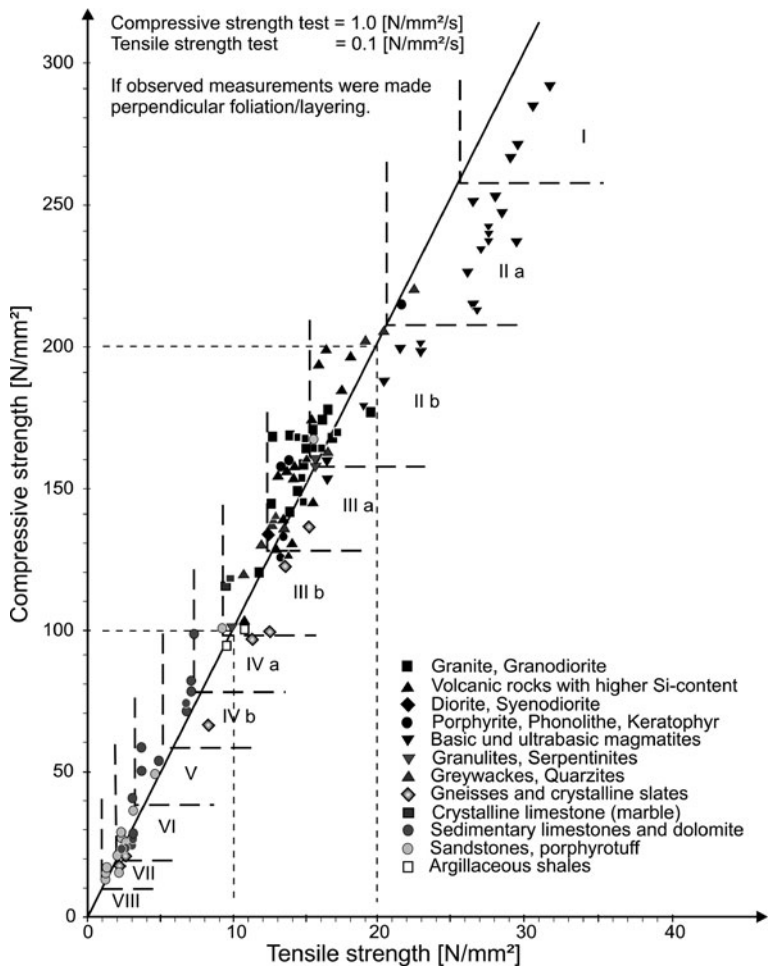


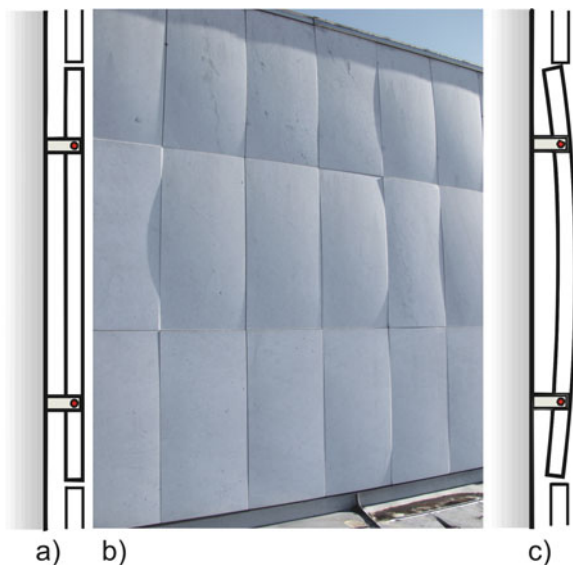
Fig. 3.62 Relationship between compressive strength values of cylindrical specimens and tensile strength values of various dimension stones (after Peschel 1977)

3.6.3 Flexural Strength

The flexural strength or bend strength is an important mechanical property of stones (Fig. 3.63). Failures due to bending stresses, both in structural and in decorative stone elements are more common than those caused by compressive or shear stresses. Typical examples for the flexural strength are the effect of wind suction and wind pressure of rear-ventilated stone facades, the load of stone panels built in stairways, or the static force of structurally loaded building parts like the architrave of a window.

In the rock specimen the bending load creates tensile stresses (Fig. 3.64). The flexural strength values are usually lower than the tensile strength values for a rock.

Fig. 3.63 Testing of the flexural strength is especially important for facade panels (a). For marble facade panels, which frequently show the characteristic bowing phenomenon (b and c), an increase in bowing with a distinct reduction in strength is the typical behavior (Koch and Siegesmund 2004, Siegesmund et al. 2008b)



The flexural strength is given by

$$R_{tf} = (3 F_{\max} l) / (2 b h^2), \quad (3.17)$$

where R_{tf} is the flexural strength (in MPa), $F_{\max}$ the applied force at failure (in N), l the length of the support span (mm), b the specimen width (in mm), and d the specimen height (in mm).

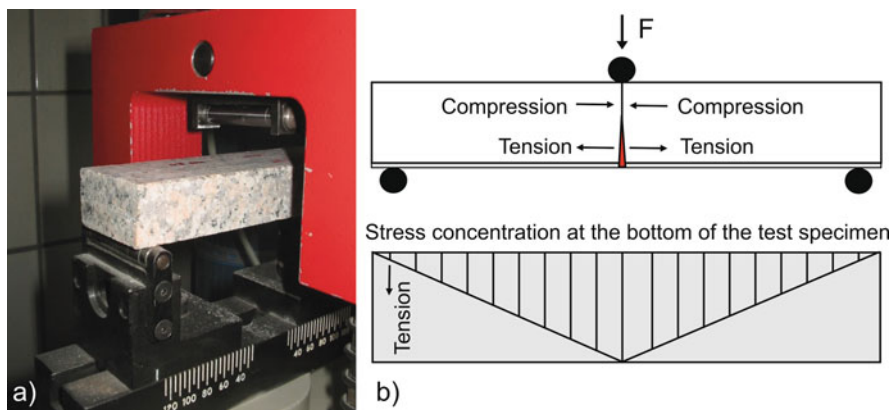


Fig. 3.64 (a) Photo of the experimental setup for flexural strength determination. (b) Schematic picture of the setup with regions of compression and tension indicated where F is the loading force direction

The flexural strength can be performed by different methods, i.e. three- and four-point load tests. There are several test methods according to the standard of the American Society for Testing and Materials (ASTM C880-89 1989) or the two European Standards DIN EN 12372 (1999) and DIN EN 13161 (2008). The comparability of the flexural strength values determined according to different standards is questionable, because the test after EN norm requires drying until mass constancy while the specimen dimensions can vary. The drying of the specimen results in slightly higher flexural strength values than determined according to the DIN. Since the EN norm allows variations in the size of the samples tested, a direct comparison of the values or for different rock groups is problematic. For example, in the Italian Standard UNI 9724/5 (1990) the width of the samples could be 30 mm/38 mm with a thickness of 20 mm/25 mm and the span of 100 mm/250 mm. Further, Blasi et al. (2000) could show that the four point load test provides somewhat less scattered results than the three point load test based on results from laboratory experiments of 12 selected marble samples.

For plutonic rocks Mosch and Siegesmund (2007) published values between 5.1 MPa and 30.2 MPa (see Table 3.9 Fig. 3.53). However, extreme outliers with maximal values up to 40 MPa were reported by Peschel (1983). For volcanic rocks similar flexural strength values were observed, but with a much wider range, because tuff and tuffitic rocks show relatively low values. Furthermore, carbonate rocks cover nearly the whole range of published flexural strength values for dimension stones, from 1.0 MPa to 34.0 MPa. Sandstones nearly cover the whole data spectrum as well. However, some authors, like Müller (2001), relate flexural strength values of more than 30 MPa only to crystalline schists.

Crystalline marbles can show relatively low values of flexural strength (Brosch et al. 2000). The fabric of the grains shows a great variety throughout the marble (see Chap. 2). The grain boundaries in marbles can be nearly in equilibrium, so that equigranular and polygonal grain fabrics can be observed. As a result the strength of these marble varieties can be significantly reduced compared to varieties with stronger interconnected and interlobate grain boundaries.

The serpentinite rock group also shows a larger range with respect to flexural strength values. However, this can be related to the heterogeneity and the various fabric patterns associated with this rock group. Many serpentinites contain relic minerals, which often reveal their origin as ultrabasic rocks, like peridotites, and additionally they are often highly deformed.

Metamorphic rocks can especially exhibit a significant directional dependency of the flexural strength (see Fig. 3.65). This anisotropy is pronounced in crystalline schists due to the preferred orientation of various fabric elements. Therefore, the data range of these rocks is quite large, with 5.9 MPa to 82.3 MPa, as given by Mosch and Siegesmund (2007).

The variability in the rock fabric elements is commonly quite pronounced in metamorphic rocks (but this holds also true for magmatic and sedimentary rocks depending on the rock fabrics, see also Chap. 2) and it is the main reason for the variability in the flexural strength values measured in different directions of one sample. Because of this the European Norm (EN) requires for dimension stones laboratory

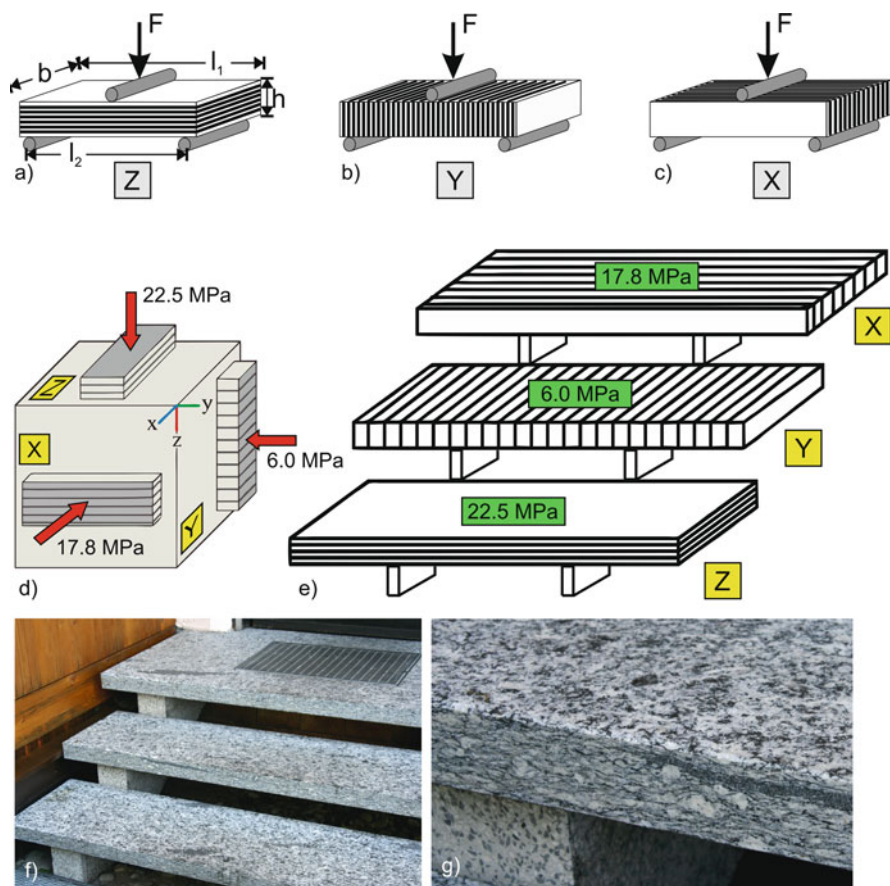


Fig. 3.65 (a–c) Schematic diagram illustrating the determination of the flexural strength in different directions with respect to a pronounced planar fabric element (layering, foliation) as indicated using three specimens of one rock sample, **d**, with **a** load perpendicular to the planar fabric, **b** load parallel to the planar fabric with the cylindrical axis parallel to the planar fabric and **c** perpendicular to it. **(d)** Differences in the measured values exhibit a directional dependency of the flexural strength (anisotropy), with a schematic diagram of a stairway illustrating the use of the different samples in **(e)**. **(f)** shows a real stairway with details of the planar element corresponding to the z-orientation in **(g)**

measurements of the flexural strength in different directions, which allows the characterization of the maximum and minimum values for a specific rock type.

The variation in the flexural strength values is also distinct in very heterogeneous rock types such as migmatites (see [Chap. 2](#)). Mosch (2009) reported for the migmatite “Franco Veteado” from Argentina a maximal anisotropy value of 17.3%. Figure 3.66 shows how the values can vary due to the occurrence of rock fabric elements, for example, only in the z-direction between 9.2 MPa and 15.6 MPa. The dominating fabric elements in the migmatite are the alternating layers of quartz-feldspar-rich layers (leucosome), layers of original rock material (palaeosome), and

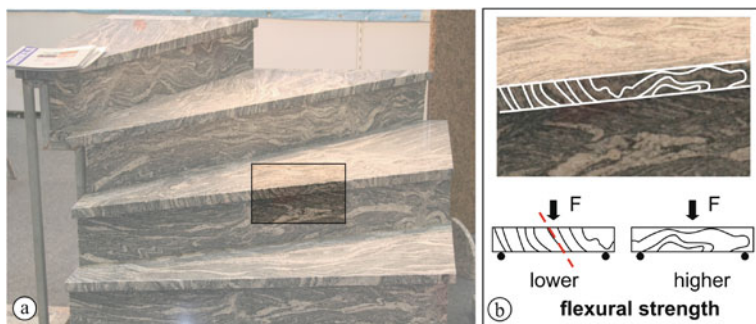


Fig. 3.66 (a) Variability of the rock fabric in gneisses and migmatites and (b) their influence on mechanical stability under conditions of difference loading paths (Mosch 2009)

layers with biotite minerals. The possible loading pathways for staircase treads should illustrate the problems of anisotropic tensile strength in construction and structural engineering applications.

The directions usually presented are the spatial orientation of the tensile vectors, which are perpendicular to the fracture surface (see Fig. 3.64). The directional dependency of the flexural strength can be relatively complex, which can result in a larger data range depending on loading path and fabric elements of the sample examined. A quite pronounced preferred orientation of fabric elements or the superposition of more the one element can result in significant anisotropy values of the flexural strength. Anisotropy values of more than 70% for strongly foliated rocks are possible. Sandstones and carbonate rocks exhibiting a strong layering can reach values of around 40 MPa. It comes as no surprise that the loading directions perpendicular to the layering or foliation show the lowest flexural strength values. Koch and Siegesmund (2004) and Siegesmund et al. (2008b) demonstrated that marble facade panels, which showed differences in weathering expressed by their degree in bowing, exhibited flexural strength values varying from around 14 MPa to 2 MPa (Fig. 3.63). With increasing weathering, which correlates and can be expressed with an increase in the porosity (see Sect. 3.2), the flexural strength decreases. This comes in addition to possible anisotropy related lower values of the flexural strength due to the existence of fabric elements in the rock.

3.6.4 Hardness and Abrasion Resistance

Rocks are often attributed by laymen with the words hard or soft. However, science and engineering have a more differentiated view on this mechanical property, but often derived from and developed for application purposes. In road construction the compressive strength of material (see Sect. 3.6.1), or the abrasion of the road construction material itself is used to describe the hardness of a rock. In the dimension stone and stone working industry a scale for the hardness often reflects and describes the difficulties in rock preparation. An overview of the hardness scales

can be found, for example in Hirschwald (1912), Peschel (1983) or Rösler (1991). A widely used hardness scale, especially in mineralogical and geological investigations, is the scratch resistance after Mohs, developed in 1812 by the German mineralogist Friedrich Mohs. Further hardness scales in use are the indentation hardness, for example, after Vickers, developed in 1924 by Smith and Sandland at Vickers Ltd., or after Knoop, developed in the USA in 1939. For polymers the Shore hardness is mainly used by applying a durometer. The Rosival scale is based on the abrasion hardness, and it is basically an improved Mohs scale mainly used in civil engineering today. For further details on hardness testing see Franklin and Dusseault (1989). Although the hardness is a material property the different measurement methods provide different values; but they can often be converted into each other. A direct conversion from the Mohs (MH) to the Vickers (VH) hardness is possible using the following equation after Picot and Johan (1977):

$$\text{MH} = 0.7 \cdot \text{VH}^{1/3}. \quad (3.18)$$

In general, it can be said that granite is a harder rock than sandstone, for example, as granite consists of minerals with the Mohs hardness 7 (quartz), 6 (feldspar), and 2-3 (mica), whereas sandstone consists of minerals with the Mohs hardness 7 (quartz as the matrix) and 2-3 (calcareous or siliceous cement). However, the micro-hardness is a highly variable property due to the heterogeneity of natural stones and the diversity in minerals. Primavori (1999) had shown that the micro-hardness based on Knoop could be a useful tool for the characterization of rocks with respect to the wear strength and processing performance, which is finally controlled by the hardness of the individual rock-forming minerals in contrast to the average hardness. For granite this could mean that micro-hardness measurements will produce a dataset of extreme variability, because the Knoop micro-hardness is determined by a series of measurements 1 mm apart from each other made by a diamond-tipped penetration device pressed at a given load against the surface.

Attempts were made to directly correlate the material hardness with other mechanical properties of interest; the shore hardness related to the compressional strength is one example (Peschel 1983). Materials with higher shore hardness values also show higher values in compressional strength.

A measure for rock hardness can also be the degree of abrasion, which is the resistance of a rock against the grinding force. The abrasion resistance of a rock mainly depends on the mineralogical composition and the rock fabric. Of further importance is the moisture content in the rock (Hirschwald 1912). The determination of the abrasion resistance basically means how much a rock can resist forces related to footsteps and walking (see Figs. 3.67 and 3.68). This, on the other hand, also depends on the frequency of the footsteps and walking as well as the material hitting the stone on the floor like bare-feet or shoes. Therefore, places and rooms are classified according to different degrees of exposure. Floors with light exposure are for example bedrooms or bathrooms, since these places are commonly entered bare-footed or with house shoes, which in general have a soft sole, giving them



Fig. 3.67 Carrara Marble stairs in (a) in Montevideo (Uruguay) and (b) in Buenos Aires (Argentina) clearly show a differential abrasion



Fig. 3.68 Staircases in the Geology Department at Princeton University (U.S.A.) showing significant abrasion. The staircases are made of an altered and metamorphosed ultramafic rock. The parts that wear down are talc plus magnesite. The more resistant lines and bumps are chlorite and/or pyroxene

these rooms the classification level “abrasion class 1”. This means that the dimension stones used as floor tiles should not produce any sand or dust when stepped on. Interiors with medium exposure are living and dining rooms, since these places are mainly entered with shoes having soft to normal soles, consequently only light abrasions can be expected in these rooms. Places with higher exposure levels are kitchens, house entrances, economic areas, foyers, receptions, showrooms and sales rooms, or in general more public places, like airports. These places are highly frequented, and therefore, highly exposed to abrasion and subsequent sand production from the stones used as floor tiles.

For the determination of the abrasion resistance different methods are applied, for example, the Amsler or Böhme abrasion testing machines, basically working with a wheel. Although other methods available apply the same principles, the standardization can be quite different. Therefore, a conversion of the data from different

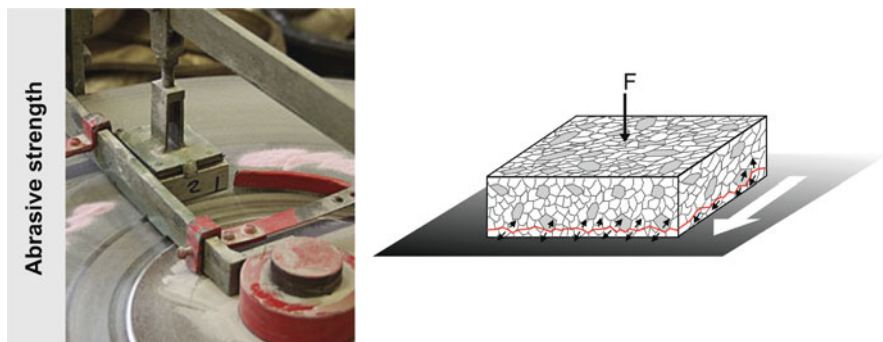


Fig. 3.69 Measurement of the abrasion resistance: Left: Photo of the setup with the cubic sample pushed on the rotating grinding wheel. Right: Schematic drawing of the abrasion at the contact where the rock sample is pushed with constant force F onto the moving grinding wheel

methods is not possible at the present. For the Böhme test, a rock specimen with a quadratic base is pushed with a defined weight on a grinding wheel, and then ground by using a defined grinding medium or abrasive (see Fig. 3.69). At the contact between the specimen and the grinding wheel, shear and tensional cracks occur resulting in a drastic softening of the rock fabric (Fig. 3.69). All minerals and fabric elements of the rock contribute to the abrasion resistance (Strohmeyer 2003).

The dimension of the specimens should be $71 \times 71 \times 25$ mm (length, width, height) because the contact surface between the rock and the grinding wheel has to be at least 50 cm^2 . The Böhme test value indicates the material loss of a specimen after a certain number of rounds on the grinding wheel. Therefore, higher values represent samples with a lower abrasion resistance and lower values equate to samples with a higher abrasion resistance.

Table 3.9 summarizes the abrasion resistance values of several rock types, whereas Fig. 3.70 shows the range of dispersion for various rock types. In general, it is desirable to have stone with a lower abrasion resistance value, indicating a higher abrasion resistance.

Granites are often seen as very hard rocks and advertised as robust material that shows almost no abrasion even at higher exposure or wear levels. However, Hoffman and Siegesmund (2007) and Hoffmann (2008) observed variations for granites and gneisses with mean values for the abrasion of $4.01 \text{ cm}^3/50 \text{ cm}^2$ and $4.81 \text{ cm}^3/50 \text{ cm}^2$, respectively. Any directional dependence, or anisotropy, of the abrasion resistance found in rocks is strongly controlled by the preferred orientation of the macroscopic fabric elements. However, in comparison to other mechanical strength values the abrasion resistance seems to be less sensitive to any preferred orientation of macroscopic fabric elements than other values.

Quartzite show the highest possible values of the abrasion resistance of rocks compared to other rock types. Because of its hardness a quartzite can only be processed by saw cutting and machine grinding and polishing. Basalt shares similar values in hardness and abrasion resistance, because it is also a relatively hard

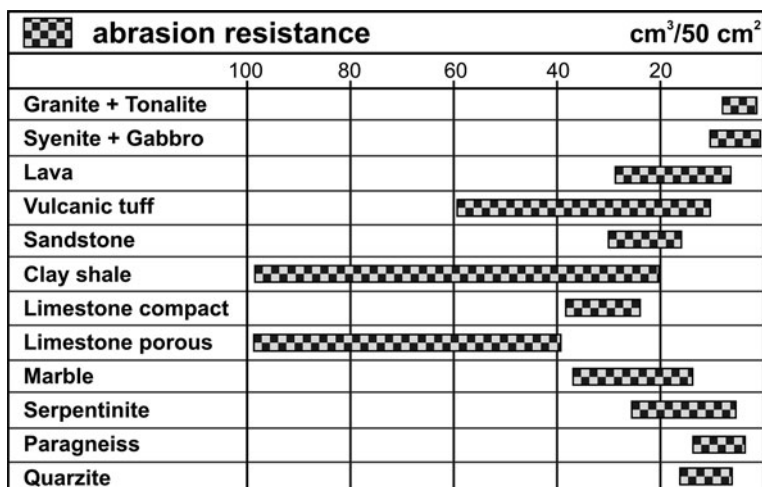


Fig. 3.70 Böhme test abrasion values for different rock types including their range of dispersion (modified after Müller 2001). Note, higher values (from right to left) indicate a rock having a lower abrasion resistance, lower values indicate a higher abrasion resistance

rock. However, Hoffmann (2008) could show that mafic rocks, which mainly consist of scissile minerals like pyroxene and hornblende, experienced higher mass losses during the testing, indicating a lower abrasion resistance of these lithologies. For a hornblendite, for example (see Table 3.9), Hoffmann (2008) determined abrasion values of around $7.10 \text{ cm}^3/50 \text{ cm}^2$.

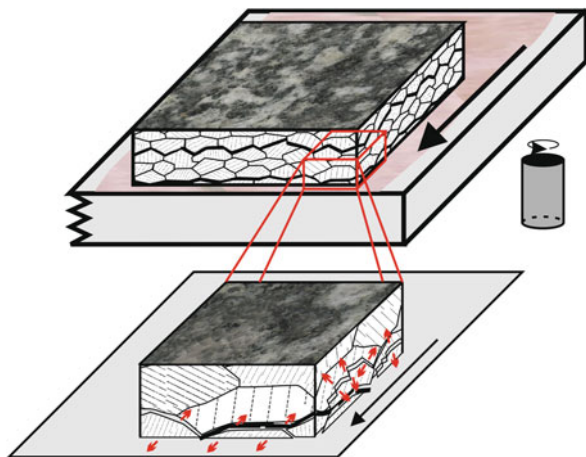
Schists are quite softer rocks, but they also can show a good abrasion resistance. This higher resistance is the result of relatively higher quartz contents in combination with a very low porosity. Schists exhibit a relatively strong anisotropy in all strength related properties. For example, the values of the abrasion resistance for a schist sample from Argentina are between 7 and $21 \text{ cm}^3/50 \text{ cm}^2$ (67% anisotropy). Significant higher values of the abrasion resistance are found for a schist from Lotharheil, Germany (see Fig. 2.40), due to the samples high quartz content.

The abrasive material loss values for carbonate rocks show a wide distribution range. Marble and dense limestone can show a similar range of values varying between $12.5 \text{ cm}^3/50 \text{ cm}^2$ and $32.5 \text{ cm}^3/50 \text{ cm}^2$ (Peschel 1983). Travertine samples as well as porous limestone can exhibit much higher abrasion values.

For sandstones the hardness and abrasion values mainly depend on the composition of the matrix and also of the cement. However, frost resistance is always given for this rock type. But on the other side it is quite difficult or often not really possible to polish sandstones, except some varieties with higher quartz content or siliceous cement, like one rock from Thailand; however, these rocks are rare worldwide.

Detailed investigations of the abrasion surface of different stones with a scanning electron microscope by Strohmeyer (2003) and Strohmeyer and Siegesmund (2002) have revealed different mechanisms of failure (Fig. 3.71). Astonishingly, after the abrasion test no scratch and stress marks were found at the mineral surfaces in

Fig. 3.71 Schematic illustration of failure mechanisms, which can occur during an abrasion test. The softening of the rock fabric is related to the occurrence of shear and tensile cracks that subsequently leads to the disintegration of the whole mineral assembly



contact with the grinding wheel, respectively grinding powder. Even the minerals that were weaker than the SiC grinding powder did not produce swarf, but rather experienced smashing and fragmentation of the crystal structure. This results in a softening of the rock fabric, which subsequently leads to disintegration of the whole mineral assembly. Open pre-existing microcracks parallel to mineral cleavages or along grain boundaries are preferentially activated during this process (Fig. 3.72). Therefore, these fabric elements are mechanically weak links in the rock, so that their frequency of occurrence is an important factor in determining the abrasion resistance.

In general, the mean values of the abrasion resistance are strongly related to the mechanical properties of the rock-forming minerals, especially the mean hardness. For mica-bearing granitic or metagranitic rocks the content of mica provides some

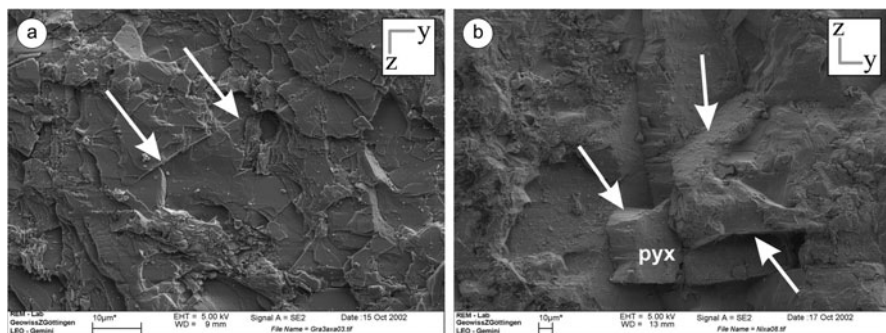


Fig. 3.72 SEM-figures from Strohmeyer (2003) of rock surfaces after an abrasion test: (a) Granitic ultramylonite with preexisting microcracks (arrows) opened during the test. (b) Gabbro norite sample with preexisting microcracks in pyroxene (pyx) activated during the abrasion test (arrows)

estimate of the abrasion values to be expected. The microcrack density as well as the grain size has a relatively smaller effect here compared to other mechanical properties, like compressive, tensile, and flexural strength. The anisotropy of the abrasion resistance shows a good correlation with the macroscopic fabric elements and the mica content, and the distribution and orientation of the mica minerals in the rock (see Sect. 2.8). For rocks with a significant preferred orientation of fabric elements (e.g. sedimentary layering, foliation), the abrasion resistance parallel or perpendicular to these elements can also be significantly different.

3.6.5 Breaking Load at the Dowel Hole

In recent years the breaking load at the dowel hole increasingly received attention due to new trends in architecture, especially the use of dimensions stones for rear-ventilated cladding or lining on buildings. The dowels have two main functions, first the load bearing, taking the weight of the cladding or lining units, and second the restraining, holding the cladding or lining units in position and resisting the cyclical loads due to wind pressure and suction (see Fig. 3.73a–d). Dimensional stones chosen for such an application have to be tested for the breaking load at the dowel hole according to the DIN EN 13364 (2002). These technical regulations

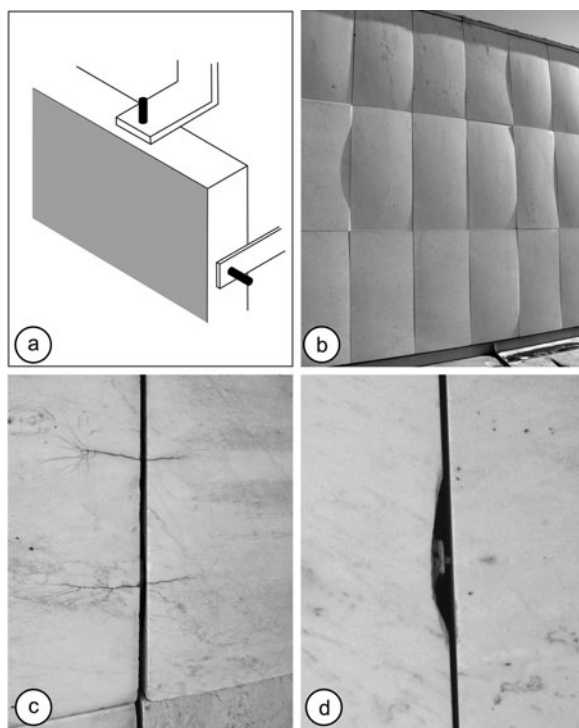


Fig. 3.73 Restrain fixings, usually ties or cramps with dowels (a); here at the top and at the side of a cladding to maintain the stone in position. Observed damages on marble panels: (b) Clearly developed convex bowing of Carrara Marble (university library, Göttingen, Germany) (c) Typical cracks often initiated at the dowels, and (d) breakout of larger marble fragments at the dowels

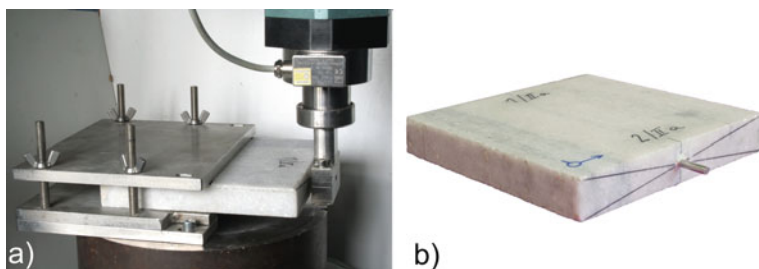


Fig. 3.74 Experimental setup for testing the breaking load at the dowel hole (a), and a dimension stone sample with the dowel implemented in the centre position of the face (b)

require a quadratic sample with a size of $200 \times 200 \times 30$ mm, or alternatively slabs of 300×300 mm (Fig. 3.74).

The breaking load at the dowel hole describes the force needed until a dowel breaks out from its dowel hole under vertical loading (Fig. 3.74). The required loading rate according to the DIN EN 13364 (2002) is 50 N/s. Furthermore, the norm requires breaking load tests in different directions in case the dimension stone exhibits any preferred orientation of rock fabric elements, resulting in planar structures like sedimentary layering or metamorphic foliation. Figure 3.75 provides an overview of the different directions, called types in the DIN EN 13364 (2002), to be tested in relation to the orientation of the planar elements.

For the testing dowel holes have to be drilled into each site of the cubic sample or slab, exactly in the middle and with not more than 2 mm deviation in relation to the shorter margins of the sample face (see Figs. 3.74 and 3.75). Rohowski (2001) compared the values of the breaking load at the dowel hole of 15 different dimension stone types from two different measurements following different norms, the older German norm, LGA (1988), and the 1999 draft version of the European norm DIN

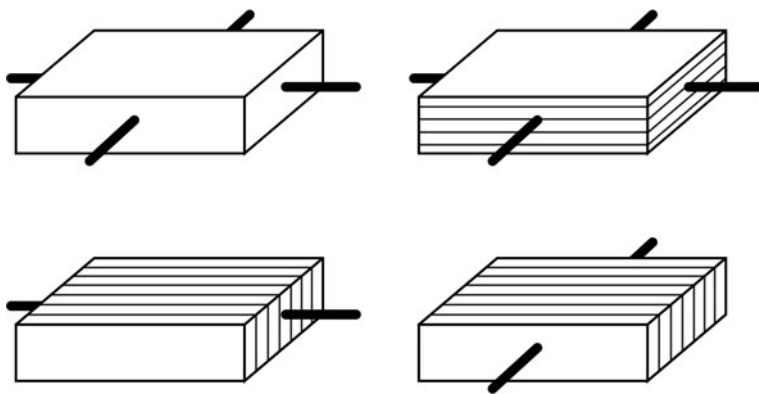


Fig. 3.75 Orientation of dowels in relation to the spatial orientation and distribution of planar fabric elements in the sample to be tested (after DIN EN 13364 2002)

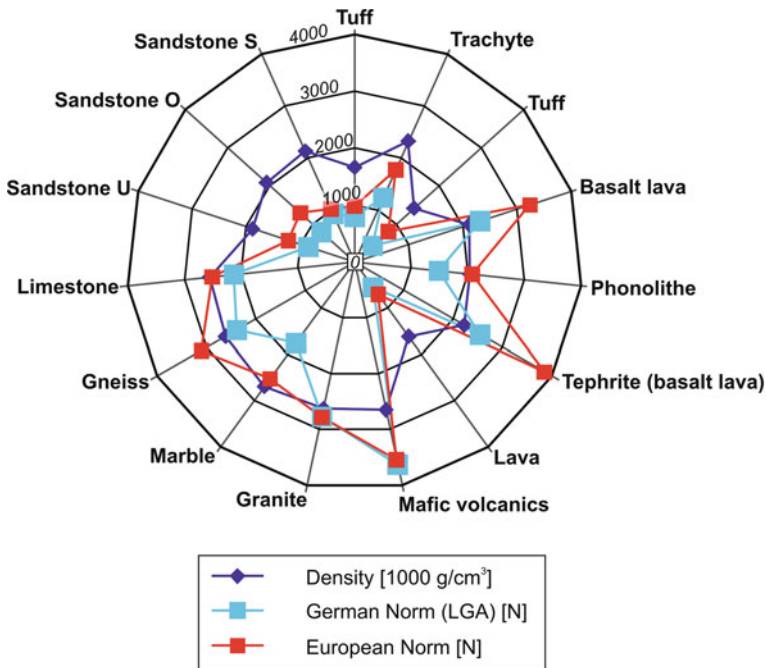


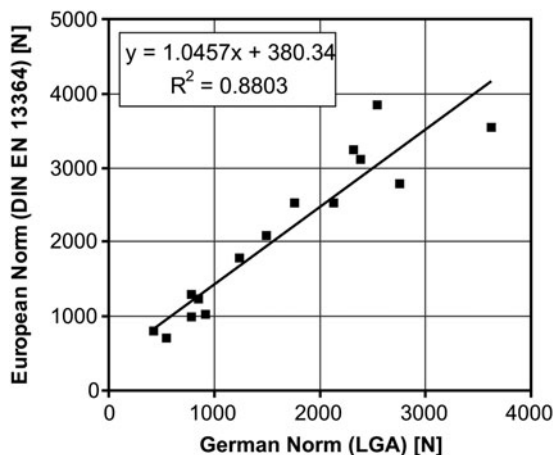
Fig. 3.76 Breaking load values at the dowel hole (in N) and density (in 1,000 g/cm³) for 15 samples determined after two different norms, the German norm, LGA (1988) and the 1999 draft version of the European norm, DIN EN 13364 (2002). Data from Rohowski (2001)

EN 13364 - it was introduced three years later. With the comparison of the values following both norms, Rohowski (2001) revealed differences of up to 88%, with higher deviations for the very high porous rock samples (see Fig. 3.76). Rohowski (2001) suggested there might be a correlation between the breaking load at the dowel hole and the bulk density as shown in Fig. 3.76. However, the mafic rocks and basalt samples, followed by the granite and the gneiss showed in both experiments the highest values compared to other rock types. These rock types usually have good grain to grain contacts, whereas marbles, for example, often have grain boundary cracks.

These differences, for some samples of significant amount, certainly have an effect on the direct comparison between values determined following different norms, although the data distribution in Fig. 3.77 suggests a systematic difference; but the sample number is too low, especially for samples with higher breaking load values. Some of the differences may originate from the new rules in sample drying, since the new European norms require drying at 70°C until mass constancy is reached.

The mean variation coefficient of the experimentally derived values for the breaking load at the dowel hole was 16% for each of the two data sets following the different norms. The data in Fig. 3.77 also show that for dimension stones

Fig. 3.77 Comparison of the breaking load values at the dowel hole (in N) for 15 samples determined after two different norms, the German norm, LGA (1988) and the 1999 draft version of the European norm, DIN EN 13364 (2002). Data from Rohowski (2001)



examined under the European norm higher values of the breaking load at dowel hole can be assumed. This provides more freedom in the construction of rear-ventilated cladding, especially with respect to the dimension and thickness of the cladding (larger and thinner plates), which subsequently is also a cost saving factor (Rohowski 2001).

The analysis of the breaking load at the dowel hole is of critical importance for the stability and structural safety of rear-ventilated cladding. The more recently discussed issue of dimensional stability loss (e.g. bowing) of dimension stones that are used as cladding is actually a very well known phenomenon, especially for marbles, but also for granites, limestone, and conglomerates (Fig. 3.78).

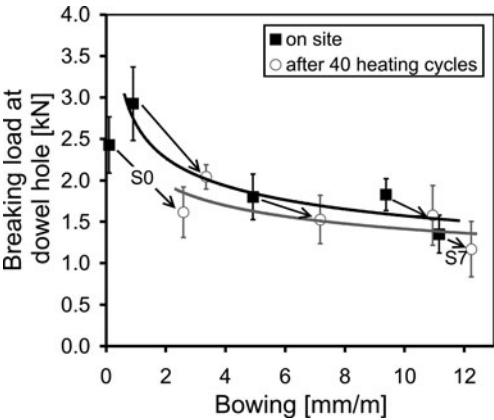
The bowing phenomenon of marble panels at buildings goes along with numerous visible damages like cracks and outbreaks (see Fig. 3.73c, d). Depending on the amplitude of bowing of the marble slabs, the intensity of damages increases significantly. In a detailed study, Siegesmund et al. (2008b) could show that the flexural strength (DIN EN 12372 1999) and the breaking load at the dowel hole (DIN EN 13364 2002) are strongly controlled by the bowing, i.e. deterioration of the marble. The decrease of the breaking load at the dowel hole versus bowing for a marble sample is shown in Fig. 3.79. The initial strength of 2.43 kN of Sample S0 with no bowing observed is reduced to 1.35 kN for Sample S7 that showed bowing of 11 mm/m. Consequently, the reduction in strength for the breaking load at the dowel hole is around 54%. Simulating the conditions of marble panels at the building in an extreme way, the bowing potential was tested in the laboratory on marble slabs (see Sect. 3.4.2). The specimens were exposed to moisture on one side lying on a mm-thick film of demineralised water, and additionally exposed to cyclical heat from a heating pad 3 cm above the slab surface on the reverse side. A total of 40 cycles were performed (see Koch and Siegesmund 2001, 2004; Grelk et al. 2004).

The bowing test was done to demonstrate how the increase in bowing of the panels controls the loss in strength on the breaking load at the dowel hole to



Fig. 3.78 Photos of cladding showing bowing: (a) granite (Siegesmund et al. 2008a), (b) Nagelfluh Conglomerate (after Grimm 1999), and (c) marble (Siegesmund et al. 2008b)

Fig. 3.79 Breaking load at the dowel hole versus bowing of the panels before (solid lines) and after 40 heating cycles (dashed lines). The standard deviations are indicated by error bars



constrain any risk management (Fig. 3.79). For example, Sample S7 exhibits the smallest increase in bowing after 40 cycles with 0.75 mm/m. The largest difference (3.62 mm/m) is observed for Sample S1 that corresponds to an irreversible length change of up to 1.1 mm/m. The observed reduction in the breaking load at the dowel hole varies between 13% (S7) and 33% (S0). In summary, the evidence clearly shows a loss in rock strength due to the bowing.

However, the general trend observed here also has exceptions. In one case an existing dowel hole was tested again after the sample was subject to a weathering simulation experiment. The test of the breaking load showed a very critical value, because microcracks that have been developed around the dowel hole caused the dowel hole to break immediately after the beginning of the loading.

3.7 Ultrasonic Wave Velocities and Young's Modulus

Recently ultrasonic measurements are gaining importance as non-destructive tools for evaluating structural damage on historical buildings and artworks. In these methods, P-wave velocity (V_p) measurements are correlated with the porosity increase due to fabric damages in rocks. Such correlations provide a method of measuring the degree of structural damage due to weathering and deterioration using indirect measurements. Damage prevention and repair planning can be made on the basis of such a V_p /structural damage classification.

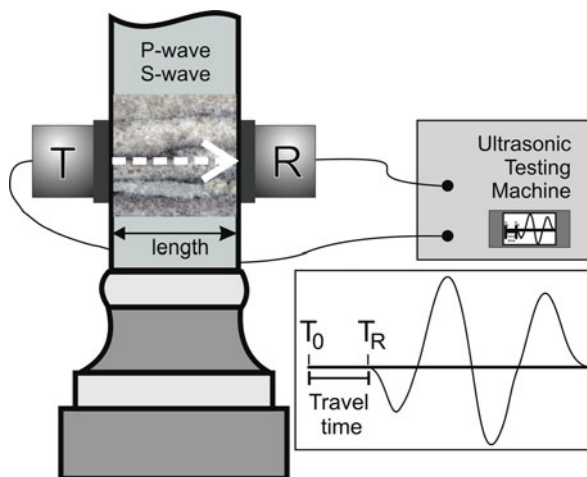
The elastic properties of a solid material and by this also the seismic or acoustic velocities are fully determined by the elastic constants (stiffness). These constants combine stress σ acting in a specified direction to the strain ϵ in a specified direction, where both directions are not necessarily the same. Stiffness is described by a fourth rank tensor and written in a 6×6 matrix. In case of one dimensional stress ϵ is proportional to σ through the modulus of elasticity or Young's Modulus E , described by the Hooke' Law as follows:

$$\sigma = E \epsilon. \quad (3.19)$$

This linear relationship is only valid for small and reversible deformations and as long as the material is below a specific strain. From the elastic properties the elastic wave velocities may be calculated by solving the Christoffel Equation (Crosson and Lin 1971, Siegesmund 1996).

In general, experimental velocity data are measured in the laboratory by the method of "pulse transmission" through rock samples of a few centimeters in length (Fig. 3.80). This is basically an application that was introduced by Birch (1960) in which an electrical pulse is applied to a transducer at one end of a specimen. The resulting deformation is transmitted through the specimen to a receiving transducer where the mechanical signal is converted into an electrical signal. The velocity is obtained from the travel time and the length of the specimen after correction for the system delay. This method is used in the same way as ultrasonic measurements in the laboratory as well as on-site; inside and outdoors, when portable equipment is

Fig. 3.80 Schematic diagram illustrating the ultrasonic measuring technique, with transducer (T) and receiver (R) in direct contact to the sample of known length (l in mm). The P- or S-wave travels the distance in a discrete travel time (t in μs) measured with an ultrasonic testing machine. From both data the ultrasonic velocity (V in km/s) can be calculated with $V = l/t$

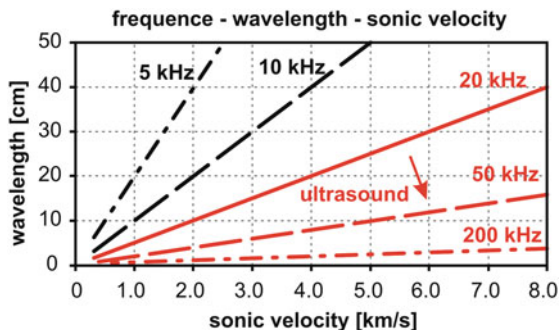


available (see Chap. 6). The transmitter and receiver transducers are then placed on opposite surfaces of the sample to be measured, for example a column or a sculpture (see Fig. 3.80).

In order to get reproducible ultrasonic signals and reliable velocity data, it is important to have a solid and smooth coupling between a rock sample and the transmitting and receiving transducers that allow an undisturbed transmission of the acoustic signal. In the laboratory as well as in-situ the velocities are generated and received by piezoceramic transducers with resonance frequencies in the range usually from 0.5 MHz to 2.0 MHz. However, other transducer frequencies can also be used. Köhler (2009) pointed out that for sample dimensions of less than 20 cm the ultrasonic frequencies applied should be larger than 20 Hz, when the ultrasonic velocities are around 4000 m/s. The relationships between ultrasonic velocities, applied frequencies and wavelengths are shown in Fig. 3.81. The waves generated propagate as different wave types, however, in practical use compressional waves (P-waves) and transversal waves (S-waves) are the most important wave types. Current developments also focus on the use of surface waves. As explained above, the ultrasonic wave velocities are determined from the length of the travel pathway and the travel time. The velocity of the P-wave is always higher than that of the S-wave. As they travel through the rock sample the different wave types also experience different attenuation. During the experimental measurements the S-waves are more difficult to handle, so that for most applications P-waves are used. However, the determination of the S-wave is essential for the calculation of the modulus of elasticity (E) or Young's Modulus as shown in the following equations, either directly or through Poisson's ratio,

$$E = \rho \frac{(3 V_p^2 - 2 V_s^2)}{(V_p^2 - \frac{1}{3} V_s^2)} V_s^2 \quad \text{or} \quad E = \rho \frac{(1 + \mu)(1 - 2\mu)}{(1 - \mu)} V_p^2 \quad (3.20)$$

Fig. 3.81 Relationship between the ultrasound velocities and the frequency as well as the wave length, which must be considered when using this technique for damage characterization (modified after Köhler 2009)



where V_p is the P-wave velocity, V_s is the S-wave velocity, ρ the density, and μ the Poisson's ratio. However, to calculate the Poisson's ratio one has to know the transversal wave velocity. Poisson's ratio can be calculated by

$$\mu = \frac{V_p^2 - 2 V_s^2}{2 (V_p^2 - V_s^2)} \quad (3.21)$$

with V_p the compressional wave velocity (P-wave) and V_s the transversal wave velocity (S-wave). Very often, Poisson's ratio is estimated due to the fact that most rocks show a value between 0.20 and 0.30. Based on extensive laboratory velocity studies on rocks, it could be suggested that especially quartz-rich rocks show lower Poisson's ratios because most of the important rock-forming minerals exhibit values between 0.24 and 0.30 compared to quartz (0.07) (see discussion in Siegesmund 1996). The direct measurement of the modulus of elasticity or Young's modulus (E) is based as already explained on the relationship between stress and strain, i.e. expressed as the ratio of stress to rate or strain (statically measured Young's modulus, see Sect. 3.6.1). However, the dynamic elastic modulus can be measured by a combination of compressional (V_p) and shear wave velocities (V_s) as well as the densities. However, Rentsch and Krompholz (1961) had suggested using rod waves, which require a fixed geometry of the sample. By using this approach the dynamic Young's modulus can be directly obtained. The experimental approach based on Rentsch and Krompholz (1961) has been realized by the Geotron Company (Germany, www.geotron.de) and correlates with one-dimensional state of stress. However, as already discussed the dynamic and static obtained modulus of elasticity are always different to each other. Schön (1983) could show that the dynamic Young's modulus is usually higher than the static one; however, Strohmeyer (2003) made for metamorphic and magmatic rocks a different observation.

3.7.1 Ultrasound Velocity and Lithology

The elastic wave propagation through natural stones is mainly controlled by the mineralogical composition, rock fabric, confining pressure, temperature, fluid pressure, the pore space and its distribution and interconnectivity as well as the pore

fluids and its composition and the degree of saturation (e.g. Birch 1961, Klima and Kluhanek 1968, Nur and Simmons 1969, Christensen 1979, Gebrande 1982, Siegesmund 1996). However, for practical applications and described here in simple terms, the ultrasonic velocities of a rock sample mainly depend on its water content (degree of saturation), on its density (composition) and on its state of preservation. Therefore, the P-wave velocities are often used as a diagnostic tool for the quality assessment of natural stones. Most of these relationships are empirical, like the P-wave-density relationship, which was already described in the late 1940s by Wollard (1959) and Nafe und Drake (1963), for example. Since their relations are based mainly on data from sedimentary rocks, Birch (1961) introduced a later relationship based on magmatic and metamorphic rock data (see Fig. 3.3 or 3.4).

The mean P-wave velocity of typical rock-forming minerals varies in a very broad range (e.g. Babuska and Cara 1991, Siegesmund 1996). Mica has acoustic velocities of about 5.5 km/s for V_p and 3 km/s for V_s , whereas garnet has values of 8.5 km/s and 5.0 km/s for V_p and V_s , respectively. In general, polyphase rocks exhibit a variation in the mineral composition, so that a unique separation of different rock types using their average P-wave velocities is not possible. For example, an average P-wave velocity of 5.5 km/s can be found for granite (range 4.18–5.85 km/s), as well as for quartzite (range 2.46–5.56 km/s) or serpentinite (range 4.25–6.23 km/s, after Schön 1996).

In Fig. 3.80 data sets comprising different rock types are shown comprising three groups, plutonic rocks, sandstones and carbonate rocks, with altogether 245 varieties. From the data evaluation these three groups show clear differences with respect to the absolute acoustic velocities. For the sandstones Mosch and Siegesmund (2007) documented that the majority of the values are in the range between 1.86 km/s and 3.62 km/s, with only a few outliers. The plutonic group showed much higher values, in the range of 4.19–6.06 km/s, whereas the carbonate group exhibits a much larger range of values, from 2.06 km/s to 6.47 km/s. This separation is seen in the scatter diagrams in Fig. 3.82 although no clear relationship can be established. In fact, a similar trend of the P-wave velocities for rocks with a bulk density up to 2.6 g/cm^3 and a porosity of about 5% can be seen. But comparing the carbonate rocks with the sandstone group, it is obvious that for the same porosity the sandstones have a lower P-wave velocity than the carbonates. This is due to the mineral specific acoustic velocities of the main minerals quartz ($V_p = 6.05 \text{ km/s}$) and calcite ($V_p = 6.54 \text{ km/s}$) (Gebrande 1982) and the related rock specific acoustic velocities (see Chap. 6).

Han et al. (1986) has shown that clay minerals exhibit relatively lower P-wave velocities (1.1–2.8 km/s) and that because of this sandstones with higher clay contents have lower V_p -values than those with siliceous or carbonate cements. The main reason is the compressibility of the clay minerals, since this contributes significantly to the attenuation of the seismic waves travelling through the rock (Schön 1996). Furthermore, higher clay contents in the rock can also lead to variations in the mechanical and elastic properties of the rock due to the change of the mineral specific properties of the clay minerals as the result of changes in the water content. This mineral specific dependency of the V_p -values can also be found in the

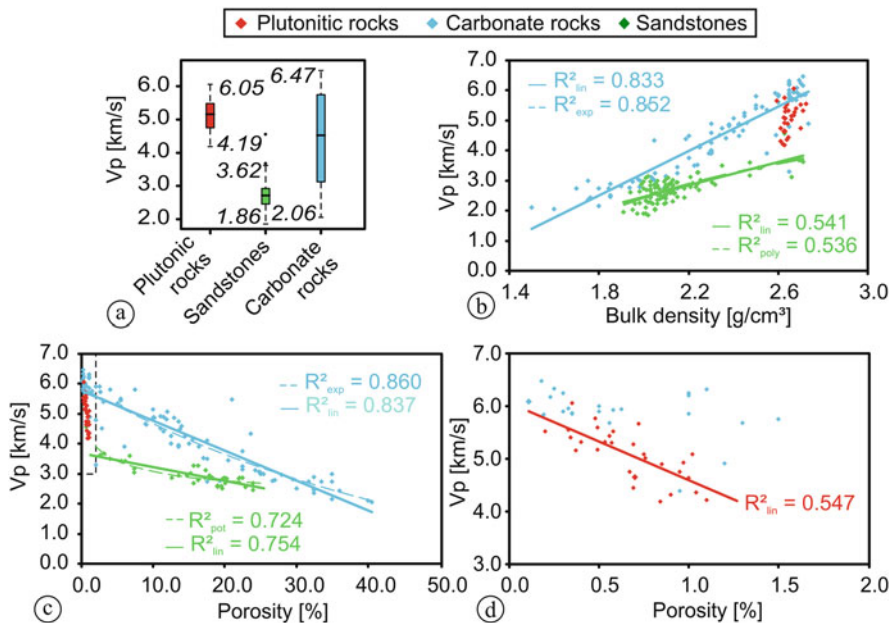


Fig. 3.82 (a) Ultrasonic wave velocities (V_p) of selected samples comprising sedimentary and magmatic rocks. Included are the statistical characteristics of upper and lower extreme. (b–c) Scatter plots for the correlations between the basic parameters bulk density and porosity and the ultrasonic wave velocity as well as the determination coefficients R^2 for each linear (R^2_{lin}) and polynomial (quadratic, R^2_{poly}) dependencies. (d) Detail of c for rocks with low porosity (porosities <2%)

low porosity rocks of the carbonate and plutonic group, since the feldspar minerals (in plutonic rocks) have in general lower acoustic velocities than the calcite mineral (in carbonate rocks).

Higher velocity values are only expected for rocks like peridotites with a higher olivine content (Olivine $V_p = 8.42$ km/s, Babushka and Cara 1991, Weiss et al. 1999), or some metamorphic rocks, like eclogites, metapelites, or high metamorphic gneisses (e.g. Siegesmund et al. 1996), but these rocks are not part of the data set presented in Fig. 3.82. Lower SiO_2 contents in rocks usually result in higher P-wave velocities (Fig. 3.83; Rudnick and Fountain 1995; Schön 1996). Velocities of metamorphic rocks may cover the broad range.

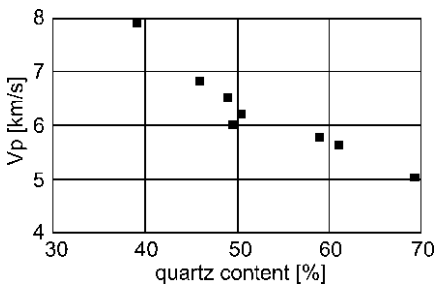


Fig. 3.83 P-wave velocities versus quartz content in magmatic rocks (after Schön 1996)

Velocity–density relations for plutonic, volcanic, and quasi-isotropic metamorphic rocks were reported by Gebrande (1982) based on a quite larger data set. Although the single values may show a large scatter, the obtained mean values of the velocities and densities correlate clearly. The following equations are the result:

$$\begin{aligned} \text{Plutonic rocks} \quad \overline{V_p} &= (-6.73 + 4.36 \bar{\rho} \pm 0.03) \text{ km/s}, \\ \text{Volcanic rocks} \quad \overline{V_p} &= (-2.73 + 2.81 \bar{\rho} \pm 0.18) \text{ km/s}, \\ \text{Metamorphic rocks} \quad \overline{V_p} &= (-6.93 + 4.41 \bar{\rho} \pm 0.37) \text{ km/s}. \end{aligned}$$

The direct correlation between average V_p and average density is based on the assumption that for most of the rocks a variation in the mineralogical composition has an effect on both parameters in the same way (Simmons 1964).

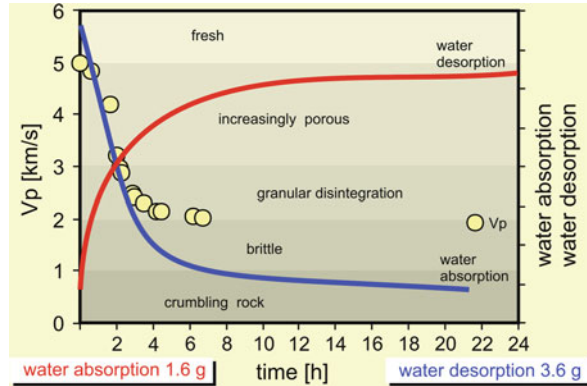
3.7.2 Ultrasound Velocity – Effect of Porosity and Fluids

The acoustic velocities of porous rocks strongly depend on the porosity and the fluid content inside. Rocks in the lower porosity range ($\Phi < 1.5\%$) show a clear linear trend between the porosity and the compressional wave velocities (see Fig. 3.82). The highly compacted carbonate rocks nearly follow the trend of the plutonic rocks. For the carbonate variety Marble Paloma (France, $\Phi = 0.18\%$), a V_p -value of 6.47 km/s is measured indicating that this rock almost consists of calcite ($V_p = 6.54$ km/s) and is more or less comparable to a marble (the average P-wave velocities calculated from 57 different marbles is given by Gebrande 1982 with 5.73 km/s or varying between 5.04 km/s and 6.42 km/s).

The physical influence of fluids on the ultrasonic velocities of a porous rock can be described by the sum of effects related to the elastic properties of the solid matrix in interaction with the compressibility of the fluid phase in the pore space of the rock. In laboratory experiments Simmons and Nur (1969) observed that an increase in the water saturation corresponds to an increase in the compressional wave velocities, whereas the shear wave velocities remain constant or slightly decrease. This phenomenon is related to the fact that the shear modulus of a rock is less sensitive to changes in the water saturation of the pore space than the effective bulk modulus.

The degree of water uptake as well as the degree of the water saturation is mainly related to the pore space itself, with the porosity, the type of pores, the pore geometry and the interconnectivity of the pores. During the initial stage of deterioration only very small and flat pores are opened, which do not allow capillary water intake. With increasing deterioration the individual pore size increases, subsequently also the pore connectivity, and finally the overall pore space. The opening of these flat pores along the grain boundaries in marble samples itself results in a drastic reduction of the ultrasonic velocities as shown in Fig. 3.84. A decrease in the water saturation through a drying procedure of the marble also results in a drastic velocity reduction. The drying of a Carrara Marble in Fig. 3.84 shows that a decrease of the water content in the pore space corresponds to a significant decrease in the acoustic velocities as indicated by the blue line. After four hours of drying the marble sample exhibits

Fig. 3.84 Drying behavior of Carrara Marble, which has a porosity of 1%. The blue curve represents a completely water-saturated sample with corresponding ultrasonic velocities (circles). The dynamics of capillary water absorption is shown for the same marble by the red curve (after Weiss et al. 2002a, Siegesmund et al. 2004b)



ultrasonic P-wave velocities of only about 2 km/s compared to an initial value of 5 km/s.

There are different theoretical approaches describing the elastic behavior of cracked or porous materials. The velocity reduction as a function of crack geometry can be calculated by the well known theoretical prediction (self-consistent model) of O'Connell and Budiansky (1974). The basic principle is that a given porosity is formed by certain types of ellipsoidal cracks. The crack geometry is only defined by the aspect ratio (i.e. the ratio between small and large axis of the ellipsoid) of the cracks. Spherical cracks have an aspect ratio of 1 and flat cracks of less than 1. The model calculations given by Weiss et al. (2002b) revealed that V_p strongly decreases as a function of porosity as observed experimentally (Weiss et al. 2001). The model calculation argues that the observed decrease in V_p can only be caused by extremely flat cracks with an aspect ratio of about 0.005 (Fig. 3.85). Compared to the same samples under water-saturated conditions, it is obvious that the velocity reduction is less pronounced (Fig. 3.86). Water saturation has an important influence on the magnitude and directional dependence of ultrasonic wave velocities. Popp (1994) also carried out acoustic velocity measurements of samples saturated with fluids. He has shown that a water saturation of the pore space results in a significant increase in the P-wave velocities with a parallel decrease in the crack related anisotropy and more or less constant shear wave velocities. Hence, it is essential to gather sufficient information on the state of water saturation of an object under investigation.

3.7.3 Ultrasonic Velocity and Anisotropy

The analysis of the ultrasonic velocities for a tombstone revealed that the velocities can vary significantly depending on the location of measurement. Comparing this data (tombstone of Wegmüller) with a statistical evaluation of Gebrande (1982), which used 57 fresh quarried marbles and having a mean P-wave velocity of

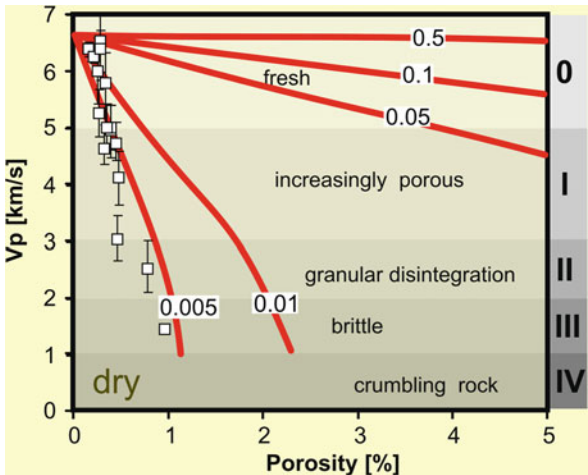
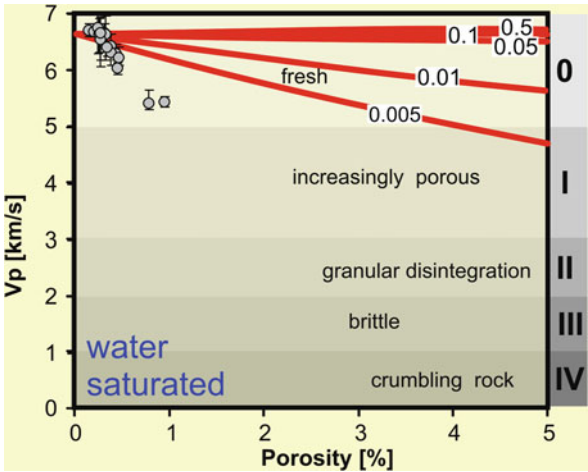


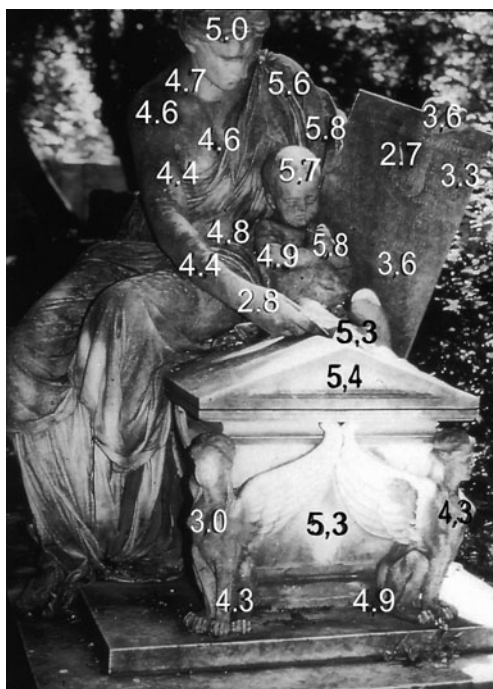
Fig. 3.85 Experimental versus modeled ultrasonic wave velocities (V_p). Velocity reduction due to microcracks with a certain aspect ratio (here: 0.005, 0.01, 0.05, 0.1 and 0.5) for dry sample conditions. The velocity reduction as a function of the porosity is according to the model of O’Connel and Budiansky (1974). The experimental data is clearly shown for the modeled velocity porosity function, which is controlled by the aspect ratio of the pores. In addition, the V_p -structural damage classification (fresh, increasingly porous, granular disintegration, brittle and crumbling) for marbles (after Köhler 1991) is given (after Weiss et al. 2002a, Siegesmund et al. 2004b)

Fig. 3.86 Experimental versus modeled velocities for water-saturated sample conditions. The same samples were used as given in Fig. 3.85. The effect of deterioration that is clearly documented on the V_p is less pronounced, if the samples are water-saturated (after Weiss et al. 2002a and Siegesmund et al. 2004b)



5.73 km/s, significant variations can be seen (see Fig. 3.87). Especially the parts of the tomb that have fine and detailed artwork exhibit lower velocity values, down to 2.7 km/s, and these parts also have lower tensile strengths. Other parts can still show P-wave velocities of around 5.7–5.8 km/s, which are comparable to the average velocity of the fresh quarried rock samples.

Fig. 3.87 Tombstone of the sculpture Michael von Wagnmüller (after Grimm 1999)



In order to understand a rock's elastic behavior, it is essential to obtain as much information as possible on the rock's structure, since there is a complex interaction between the rock fabric and petrophysical properties as compiled by Siegesmund (1996). The most important fabric elements are the crystallographic preferred orientation of the rock-forming minerals, the microcrack populations and the shape fabric (grain size, grain aspect ratios, grain shape preferred orientation, see also Sect. 2.8 and 6.4.2.5). Combined, these fabrics are responsible for the physical anisotropy of rocks, i.e. for the directional dependence of the elastic wave velocities. This means, for example, that the P-wave velocity distribution shows different values when measured in different directions of the same rock sample; for the S-waves it is similar. The spatial dependence of elastic wave velocities (V_p , V_s) considering anisotropy has mostly been studied in three perpendicular directions, which require the study of different plugs of one stone block if one uses cylindrical samples or velocity measurements on sample cubes. However, the study of anisotropy on spherical samples enables the measurements of compressional wave velocities in more or less any direction with the same accuracy, except the area near the vertical axes of rotation (e.g. Pros and Babuška 1967; Thill et al. 1969).

The complete V_p -distribution of a strongly foliated gneiss at 0.1 MPa confining pressure and dry conditions is shown in Fig. 3.88a (Siegesmund et al. 1993, Siegesmund 1996 or Rasolofosaon et al. 2001), with a minimum velocity of 3.0 km/s

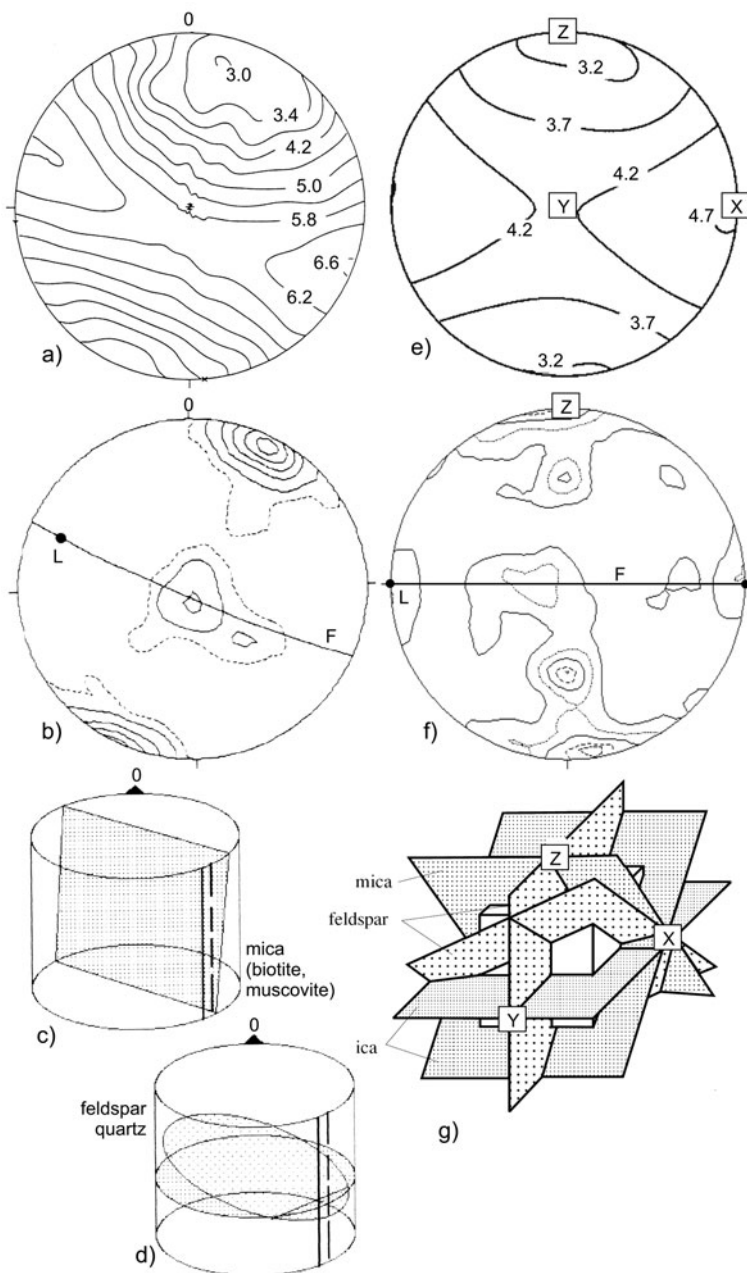


Fig. 3.88 Anisotropy of the P-wave velocities (in km/s) of strongly foliated gneiss (a–d) after Siegesmund 1996, Siegesmund et al. 1993) and granodiorite (e–g) after Schild et al. 2001): (a) complete V_p -distribution at 0.1 MPa confining pressure and (e) atmospheric pressure, Schmidt Net, lower hemisphere; (b, f) pole figures of bulk microcrack population in isolines of multiples of random distribution, Schmidt Net, lower hemisphere, F-foliation, L-lineation; (c, d, g) Schematic illustrations of the microcrack orientations

and a maximum of 6.6 km/s, giving a real anisotropy value (A) of 55% following the equation with $A = 100 \cdot (V_{p_max} - V_{p_min}) / V_{p_max}$ (%). The P-wave velocity distribution shows a distinct pattern, which reflects the orientation of the foliation as shown in Fig. 3.88b and by this the orientation of the mica minerals, biotite and muscovite, with their mineralogical (001) planes parallel to the foliation plane (see Fig. 3.88c). The strong anisotropy is the result of a combination from the single crystal anisotropy of the mica minerals and the orientation of open cleavage cracks parallel to the (001) planes of the mica minerals. Mica minerals have the highest velocities perpendicular to the [001] axis or parallel to the (001) plane, and the lowest ones parallel to the [001] axis. Because of this, a rock sample with a strong foliation formed by mica minerals would have a perfect transverse isotropic symmetry of the V_p -distribution. Furthermore, perpendicular to the (001) open cleavage cracks the P-wave velocity have the lowest values, parallel to them the highest ones. However, a perfect transverse isotropic symmetry of the V_p -distribution is not achieved due to the existence of nearly horizontal open cracks in quartz and feldspar minerals (see Fig. 3.88b, d), which decrease the P-wave velocities in the vertical direction or the centre of the V_p -distribution. A rather detailed investigation of almost all aspects of the rock fabric of this sample and additional P-wave velocity distributions with increasing confining pressure can be found in Siegesmund (1996).

In Fig. 3.88e, f the complete V_p -distribution at atmospheric pressure and the bulk microcrack orientation of a metamorphic overprinted granodiorite are presented (Schild et al. 2001). The main minerals of the foliated rock are K-feldspar, with grain sizes between 100 μm and 2 cm, plagioclase, quartz, and biotite, however the foliation is not as pronounced as in the previous sample. Features of alteration and brittle deformation can be found. The highest P-wave velocities of more than 4.7 km/s are in the foliation and parallel to the lineation, the lowest one with less than 3.2 km/s are perpendicular to the foliation as shown in Fig. 3.88e, resulting in an anisotropy of 21.8%. Comparable to the gneiss sample, the main cause for the anisotropy is the open cleavage cracks parallel to the (001) planes in the mica minerals, which dominate the bulk open crack distribution (Fig. 3.88f). The mica minerals are oriented mainly parallel to the foliation in the xy-plane, so that their poles plot perpendicular to that and parallel to the z-direction. However, the foliation is not as strong developed as in the gneiss, but the lineation is more pronounced. This means that the mica minerals rotate parallel to the lineation resulting in a girdle-like orientation of the open cleavage cracks. The schematic diagram in Fig. 3.88 g illustrates the orientations of these different crack populations.

A sandstone sample (porosity 1.8%) investigated by Dürrast et al. (2002) is shown in Fig. 3.89. The complete V_p -distribution at 0.1 MPa confining pressure has its highest values with 4.64 km/s almost horizontal and the lowest one with 3.98 km/s almost perpendicular to it, where this reflects the orientation of the nearly horizontal sedimentary layering (ss); the anisotropy is 14.2%. The sedimentary layering can be identified by small layers of dark clay minerals in the sandstone. Two macroscopically visible cracks (r1, r2), partially open and partially mineralized, with semi-vertical orientation have some but not significant effect on the P-wave velocity distribution (Fig. 3.89b, c).

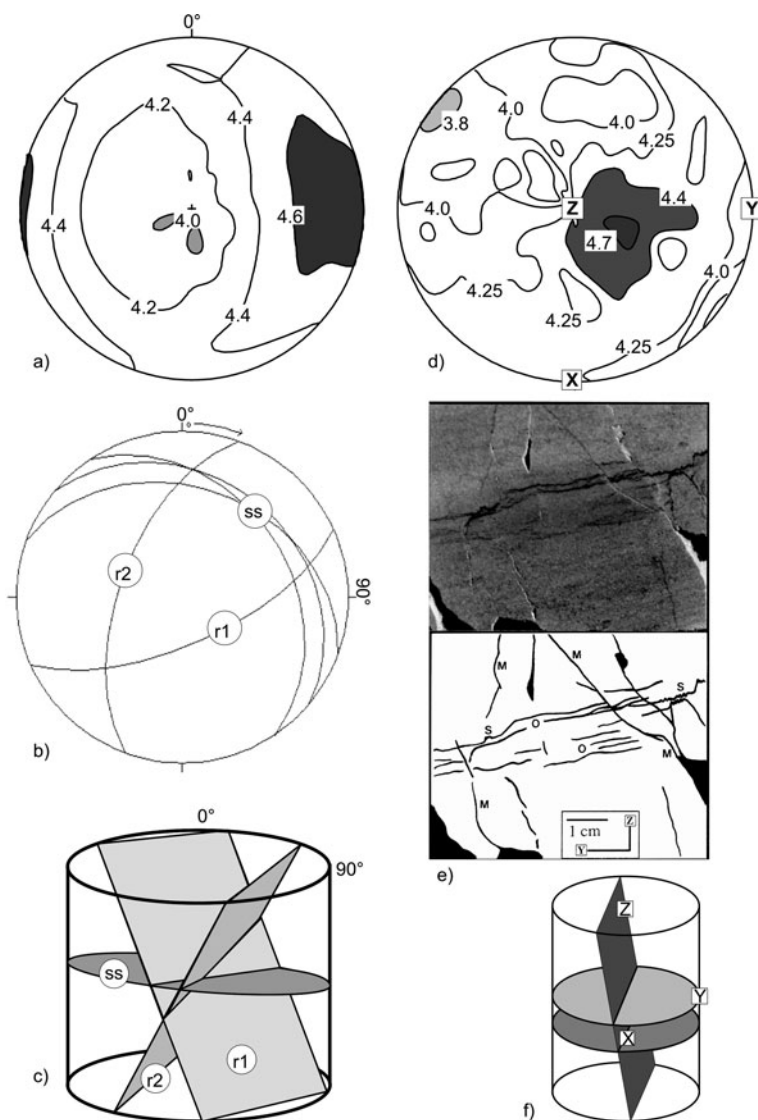


Fig. 3.89 Anisotropy of the P-wave velocities (in km/s) of a sandstone (a–c) after Dürrast et al. 2002) and a dolomitic limestone (d–f) after Dürrast and Siegesmund 1999): (a, d) complete V_p -distribution at 0.1 MPa confining pressure, Schmidt Net, lower hemisphere; (b, f) great circle figure of a sedimentary layer (ss) and crack (r1, r2) orientation, Schmidt Net, lower hemisphere; (e) Photo and drawing of the main microstructure, M - mineralized cracks, S - horizontal stylolites, O - layers of organic material. (c, f) Schematic illustrations of the microstructure orientations

Results from ultrasonic laboratory measurements and rock fabric analysis of a dolomitic (94.6%) carbonate rock sample with a porosity of 9.6% are shown in Fig. 3.89d–f (Dürrast and Siegesmund 1999). The complete Vp-distribution at 0.1 MPa confining pressure has the highest velocities with more than 4.7 km/s semi-vertical, and almost parallel to the vertical z-axis. The lowest values with less than 3.8 km/s are in the horizontal plane subparallel to the y-direction (Fig. 3.89d). The anisotropy of nearly 25% is the result of a combination from horizontal oriented microstructures, stylolithes (S) and small layers of accumulated organic material (O), and semi-vertical cracks, partially open and partially mineralized (M), as shown in Fig. 3.89e, f. However, here the influence of the cracks on the Vp-distribution is much more significant, since the horizontal features at this pressure level only have a minor influence. The highest P-wave velocities are therefore more or less parallel to the fracture planes and the lowest perpendicular.

All examples shown here demonstrate clearly that for all rock types open microcracks - in cases where they are present - have significant effects on the anisotropy of the P-wave velocity distribution at (near) atmospheric conditions. For metamorphic rocks the preferred orientations of minerals combined with anisotropic minerals have additional effects, whereas for sedimentary rocks sedimentary features play an important secondary role.

Water saturation of these samples would increase the overall P-wave velocities due to higher density and higher stiffness of the rock samples, and it decreases the anisotropy. But in most cases the character of the Vp-distribution does not change as shown in Fig. 3.90. The granodiorite from Schild et al. (2001) discussed above was also measured under fully water-saturated conditions and atmospheric pressure. The Vp-values increase in all directions as the calculated ΔV_p diagram ($=V_{p_saturated} - V_{p_dry}$) indicates (Fig. 3.90c), but not in the same amount. The highest increase with more than 2.0 km/s is perpendicular to the foliation due to the filling of the open mica cleavage cracks. However, the overall pattern of the Vp-distribution of the water-saturated samples is still similar to the sample measured

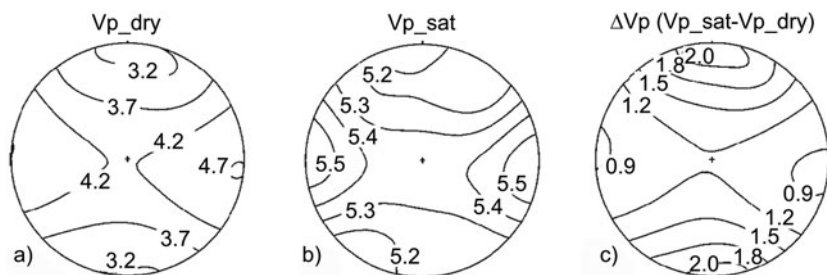


Fig. 3.90 Anisotropy of the P-wave velocities (in km/s) of a granodiorite under dry (a) and water-saturated (b) conditions at atmospheric pressure (after Schild et al. 2001). (c) Calculated difference in the Vp-distribution between saturated and dry conditions. All projections are Schmidt Net, lower hemisphere

under dry conditions, but with an anisotropy of about 7% compared to 21.8% under dry conditions (see also Dürrast et al. 2002).

3.7.4 Velocity and Degree of Weathering

Based on a number of ultrasonic measurements Köhler (1991) introduced a decay classification scheme for Carrara marbles that mainly depends on the relationship between the acoustic P-wave velocities and the porosity (see Fig. 3.85 and 3.86, see also the discussion in Dürrast et al. 1999 or Ruedrich et al. 2001). The initial value of $V_p = 6.6$ km/s represents a pore free “quarry-fresh” or “healthy” calcite marble sample. With decreasing V_p the degree of deterioration becomes worse and is alarming. At P-wave velocities of 3.0 km/s an outside exposition is not recommended any more and a static consolidation should be considered. These low compressional wave velocities can be explained qualitatively by a decrease in the cohesion and a corresponding decrease in the elastic coupling between the calcite crystals of the marble.

A systematic analysis of various marble samples of different origin and decay degree (V_p range from 6.5 km/s to 1.5 km/s) has shown that even at very low porosities of around 1% the ultrasonic velocities can be reduced drastically (Fig. 3.85). The same sample, a highly deteriorated marble, in contrast exhibits a V_p -value of around 5.0 km/s under water-saturated conditions suggesting wrongly that this marble is not deteriorated at all (Fig. 3.86). This example clearly shows the significant influence of the pore space and the degree of water saturation on the acoustic velocities as was already discussed in Chap. 6.

The empirically derived V_p -damage classification developed for Carrara Marble was also incorporated in Table 3.10. In this classification a damage class of zero (Table 3.10) corresponds to a fresh marble with velocities higher than 5.0 km/s (Köhler 1991). A fragile marble will show V_p -values between 1.5 km/s and 2.0 km/s (damage class III). Fitzner and Heinrichs (1992) have reported a V_p -damage classification for the so-called Schaumkalk, the most important limestone used for the construction of the Naumburg Cathedral in Germany: fresh Schaumkalk gives V_p -values of 3.9 km/s to 4.1 km/s, while the strongly weathered ones give values below 1.0 km/s.

Comparable values were reported from Illiev (1967) for graniotoids ranging between >5.0 km/s (fresh) and <2.0 km/s for strongly weathered ones (Table 3.11). This classification is more or less in agreement with observations from Ceryan et al. (2008) on the Harsit Granitoids (Turkey), where the V_p -values cover the range from 5.7 km/s (fresh) to 0.76 km/s (completely weathered). The effect of weathering on the compressional wave velocities is also given for volcanic rocks (Arikan et al. 2007) or for travertine (Akin 2010). The basic observation is that as a consequence of weathering the state of porosity changes together with the mineralogical and chemical composition.

Table 3.10 Vp-structural damage classifications for Carrara Marble (Köhler 1991), limestone (Schaumkalk; Fitzner and Heinrichs 1992), and travertine (Akin 2010)

Marble (Köhler 1991)			Limestone (Fitzner and Heinrichs 1992)		Yellow travertine (Akin 2010)	
Rock state	Vp [km/s]	Damage class	Damage of the stone	Vp [km/s]	Weathering degree	Vp [km/s]
Fresh	$5.0 < Vp$	0	None (fresh)	$3.9 < Vp \leq 4.1$	None (fresh)	$3.66 \leq Vp$
Increasingly porous	$3.0 < Vp \leq 5.0$	I	Little	$3.0 < Vp \leq 3.9$	Slightly	$2.60 < Vp \leq 3.66$
Granular disintegration	$2.0 < Vp \leq 3.0$	II	Moderately	$2.0 < Vp \leq 3.0$	Moderately	$Vp < 2.60$
Brittle	$1.5 < Vp \leq 2.0$	III	Heavy	$1.0 < Vp \leq 2.0$		
Crumbling rock	$Vp \leq 1.5$	IV	Very heavy	$Vp \leq 1.0$		

Table 3.11 Vp-structural damage classifications for granite (Iliev 1967), granitoids from Turkey (Ceryan et al. 2008), and acid volcanic rocks from Turkey (Arikan et al. 2007)

Granite (Iliev 1967)		Granitoids (Ceryan et al. 2008)		Acidic volcanic rocks (Arikan et al. 2007)	
Weathering degree	Vp [km/s]	Weathering degree	Vp [km/s] (dry rocks)	Weathering degree	Vp [km/s]
None (fresh)	$5.0 < V_p$	None (fresh)	4.111 ± 0.198	None (fresh)	3.35–5.20
Slightly	$4.0 < V_p \leq 5.0$	Slightly	3.553 ± 0.396	Slightly	2.49–4.66
Moderately	$3.0 < V_p \leq 4.0$	Moderately	2.769 ± 0.553	Moderately	2.00–3.44
Highly	$2.0 < V_p \leq 3.0$	Highly	2.158 ± 0.486	Highly	1.39–2.97
Completely	$V_p \leq 2.0$	Completely	0.753 ± 0.184		

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