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TECHNICAL NOTE

Assessment of Inherent Anisotropy and Confining Pressure Influences on Mechanical Behavior of Anisotropic Foliated Rocks Under Triaxial Compression

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1 Introduction

The compressive strength and deformability are the most important mechanical properties of rocks in engineering applications. The critical factors influencing these parameters include constituent mineral composition, porosity, water content, temperature, anisotropy, and confining pressure (Li et al. 2012). Some rocks show well-defined fabric elements in the form of bedding, stratification, layering, foliation, fissuring or jointing. In general, these anisotropic rocks have physical, mechanical and hydraulic properties that are varied in different directions. Metamorphic rocks usually display the highest degree of anisotropy (Ramamurthy et al. 1993). The strength anisotropy of various rock types has been studied in compression tests by different researchers such as Donath (1964), McLamore and Gray (1967), Hoek (1968), Attewell and Sandford (1974), and Brown et al. (1977) on shales and slates, Ramamurthy et al. (1988) on phyllites, Akai et al. (1970), McCabe and Koerner (1975), Behrestaghi et al. (1996), Nasseri et al. (1997, 2003), Singh et al. (2001) and Zhang et al. (2011) on gneisses and schists. A review of the mentioned work shows that the maximum failure strength occurred when anisotropy angle (β) equals to 0°or 90° and it reaches to the minimum point when anisotropy angle (β) is about 30°.

Saroglou et al. (2004a, b) tested Athens schist and gneiss under triaxial compression and they found that when anisotropy angle increases from 0° to 90°, the maximum principal stress (σ_1) declines and it reaches to the minimum value at $\beta \approx 30°$, and after that is rises again. With increasing confining pressure, the maximum values of σ_1 increase, and the mode of the curves between σ_1 and β are also U shape. According to Akai et al. (1970), the strength reduction of schists at $\beta = 30°$ and 20 MPa confining pressure is about 50 %. According to Goshtasbi et al. (2006), slates of Sanandaj-Sirjan zone in Iran had a U-shaped anisotropy and their highest and lowest triaxial compressive strengths occurred at $\beta = 90°$ and $\beta = 30°$, respectively. In these rocks, the maximum values of internal friction and cohesion occured at $\beta = 90°$ and the minimum values occured at $\beta = 30°$. Li et al. (2012) studies on meta-sedimentary rocks show that triaxial compressive strength of rocks in perpendicular direction of the bedding planes was higher than the parallel direction under certain confining pressures. According to Ramamurthy et al. (1993) and Nasseri et al. (2003), the values of compressive strength and elasticity modulus in anisotropic rocks have a nonlinear relationship with anisotropy angle and confining pressure.

Behrestaghi et al. (1996) found that the maximum strength happened at $\beta = 90°$ in schists, throughout the range of confining pressure. They found that the minimum strength took place at $\beta = 30°$, nevertheless at confining pressures higher than 15 MPa, the minimum point of the curve shifted to $\beta = 45°$. A similar observation has been reported by McLamore and Gray (1967) for slate and Singh

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et al. (1989) for phyllites beyond 276 and 70 MPa confining pressures, respectively.

Behrestaghi et al. (1996) have pointed out that similar to the triaxial compressive strength, deformation modulus is a function of β at all values of the confining pressures. Also, they found that with increasing confining pressure, the cohesive strength of the schists is increased but the value of friction angle decreases. For these rocks, the maximum and minimum values of cohesive strength have been observed at $\beta = 90°$ and $\beta = 30°–45°$, at all confining pressures, respectively. McCabe and Koerner (1975) studied on a mica schist sample in compression condition and found that the shear strength parameters and consequently compressive strength varied with the change of foliation angle. The maximum values of shear strength and friction angle have been recorded at $\beta < 30°$ and $\beta > 70°$, whereas the minimum values were recorded at $\beta = 50°–59°$.

In this research, the mechanical behavior of anisotropic metamorphic rocks of Hamedan Province, west of Iran, was measured under confined condition at seven anisotropy angles, beta (i.e. $\beta = 0°$, 15°, 30°, 45°, 60°, 75° and 90°), between anisotropy planes and the major loading directions.

2 Methods and Materials

Suitable sampling locations of rock types were selected from the 1:250,000 scaled Hamedan geological map (GSI 1977). During field investigations, samples were obtained from quarries, road and railway cuttings and excavated foundations on different parts of the Hamedan Province. The sampling locations include Avarzaman (AVZ), Heydareh (HDR), Malayer (MLR), Varkaneh (VRK) and Zagheh (ZGH). All selected samples were fresh rocks obtained from depths of about 5 m. All collected rock samples were transferred to the geotechnical laboratory. In the laboratory, cylindrical cores were obtained by a coring machine. Cut end-faces of the obtained cores were smoothened and made perpendicular to the core axes on a polishing and lapping machine. Finally, 140 specimens were prepared for performing triaxial compressive strength test at seven different anisotropy angles and various confining pressures. Diameters of prepared specimens were about 54 mm. Length to diameter ratios of the specimens were kept in accordance to the ISRM (2007) standard.

3 Results and Discussion

3.1 Petrographical and Mineralogical Studies

Thin sections were prepared in two directions: parallel and perpendicular to the rock anisotropy planes, and they were studied using a polarizing microscope as suggested by ISRM (2007). The studied rocks were commonly composed of quartz, biotite, feldspar, muscovite, garnet, sillimanite, staurolite, andalusite, graphite and other tiny cryptocrystalline matrix materials. The textures of the studied rocks were different in the parallel and perpendicular directions. The anisotropy planes were clearly seen when the thin sections were prepared perpendicular to the rock anisotropy.

Textures of the rocks were found to be porphyroblastic and lepidoblastic. In the porphyroblastic fabric, the rock

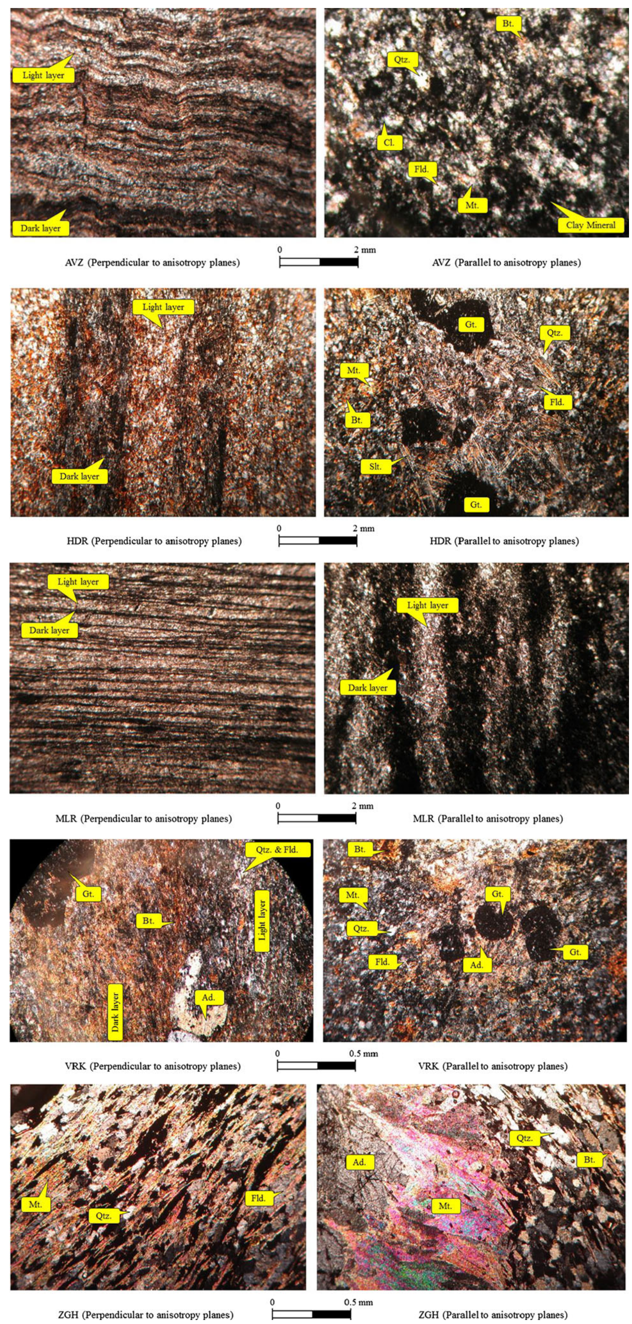

Fig. 1 Photomicrographs of the five tested rock types

texture could be divided into two components: porphyroblasts and matrix. Andalusite, staurolite and garnet porphyroblasts were found to be dominant types of minerals with sizes of 0.2–1 mm (Fig. 1). The matrix was characterized by quartz, feldspar, biotite and muscovite (50–100 μm). The results of petrographical and mineralogical studies are presented in Table 1. Figure 1 shows microscopic images of tested rock samples.

3.2 Physical Properties

Physical properties of the rock samples, such as dry and saturated unit weights (γ_d and γ_{sat}), specific gravity (G_s), porosity (n) and water absorption (W_a) were determined as suggested by ISRM (2007). For this purpose, five sets of tests were performed on the core specimens prepared in different directions of the anisotropy planes. On each type of studied rocks, 35 tests and therefore, a total of 175 tests were performed for calculating physical properties. Standard deviations (SD) are provided to get an idea of the natural variability of the tested samples (Table 2). For studied rocks, the dry unit weight ranges between 2.55 and 2.81 g/cm^3, the saturated unit weight ranges between 2.64 and 2.82 g/cm^3, the specific gravity ranges between 2.77 and 2.84, the porosity ranges between 1.13 and 8.99 %, and the water absorption ranges between 0.40 and 3.55 %.

Mineral content and porosity are the most affecting parameters on the density of tested rock. For example, the presence of dense metamorphic minerals such as garnet and andalusite in the samples of HDR and VRK increases the density and unit weight. Also, the low water absorption in these samples is due to the low values of porosity. On the other hand, the presence of minerals with low density such as muscovite and the high value of porosity decrease the unit weights as can be seen in the sample of ZGH.

3.3 Triaxial Compressive Strength

Triaxial compressive strength tests were performed at seven anisotropy angles (β) and different confining pressures (σ_3) as suggested by ISRM (2007) and ASTM (1996). The maximum principle stress (σ_1) and axial strain of specimens were recorded during the tests. The variations of σ_1 with β for tested rocks are plotted in Fig. 2. In these rocks, with increasing σ_3, the values of σ_1 are increased. Variation rates of σ_1 at $\beta = 0°$ are greatest in all anisotropy angles. These results are comparable to the results presented by Saroglou et al. (2004a, b) Akai et al. (1970), Goshtasbi et al. (2006) Behrestaghi et al. (1996), Ramamurthy et al. (1993) and Nasseri et al. (2003).

In unconfined condition ($\sigma_3 = 0$), the minimum values of compressive strength of studied rocks were obtained at $\beta = 15°$–$30°$. In confined condition ($\sigma_3 > 0$), the minimum values of compressive strength were obtained at higher anisotropy angles. In other words, by increasing confining pressure, the anisotropy angle corresponding to minimum strength is increased. For example, in the sample of AVZ, the minimum values of unconfined compressive strength were obtained at $\beta = 15°$, whereas the minimum value of confined compressive strength at

Table 1 Mineral composition of tested samples

Rock mark	Rock type	Mineral content (%)									
		Qtz.	Fld.	Bt.	Mt.	Gt.	Ad.	Slt.	St.	Chl.	Other minerals
AVZ	Phyllite	20	10	40	7	–	3	–	–	12	A. (3 %), T. (1 %), Z. (1 %), P. (3 %)
HDR	Sillimanite garnet hornfels	16	9	38	5	13	4	13	1	–	T. (1 %)
MLR	Slate	20	10	32	7	–	–	–	–	3	G. (12 %), CM (10 %), H (7 %)
VRK	Andalusite garnet hornfels	20	10	41	4	10	6	–	5	–	T. (1 %), Z. (1 %)
ZGH	Staurolite andalusite schist	20	5	2	20	–	18	14	4	–	T. (2 %), H. (15 %)

Qtz. quartz, *Fld.* feldspar, *Bt.* biotite, *Mt.* Muscovite, *Gt.* garnet, *Ad.* andalusite, *Slt.* sillimanite, *St.* staurolite, *Chl.* chlorite, *G.* graphite, *A.* Apatite, *T.* tourmaline, *Z.* zircon, *P.* prehnite, *H.* hematite, *CM* clay minerals

Table 2 Physical properties of tested samples

Rock mark	γ_d (g/cm^3)		γ_{sat} (g/cm^3)		G_s		n (%)		W_a (%)	
	Value	SD	Value	SD	Value	SD	Value	SD	Value	SD
AVZ	2.73	1.58	2.74	1.59	2.77	1.61	1.74	1.88	0.64	0.41
HDR	2.81	0.09	2.82	0.08	2.84	0.07	1.13	1.77	0.40	0.28
MLR	2.71	1.58	2.75	1.59	2.82	1.61	3.76	1.97	1.33	0.78
VRK	2.79	1.58	2.80	1.59	2.82	1.61	1.16	2.16	0.41	0.43
ZGH	2.55	0.20	2.64	0.12	2.80	0.03	8.99	2.82	3.55	2.17

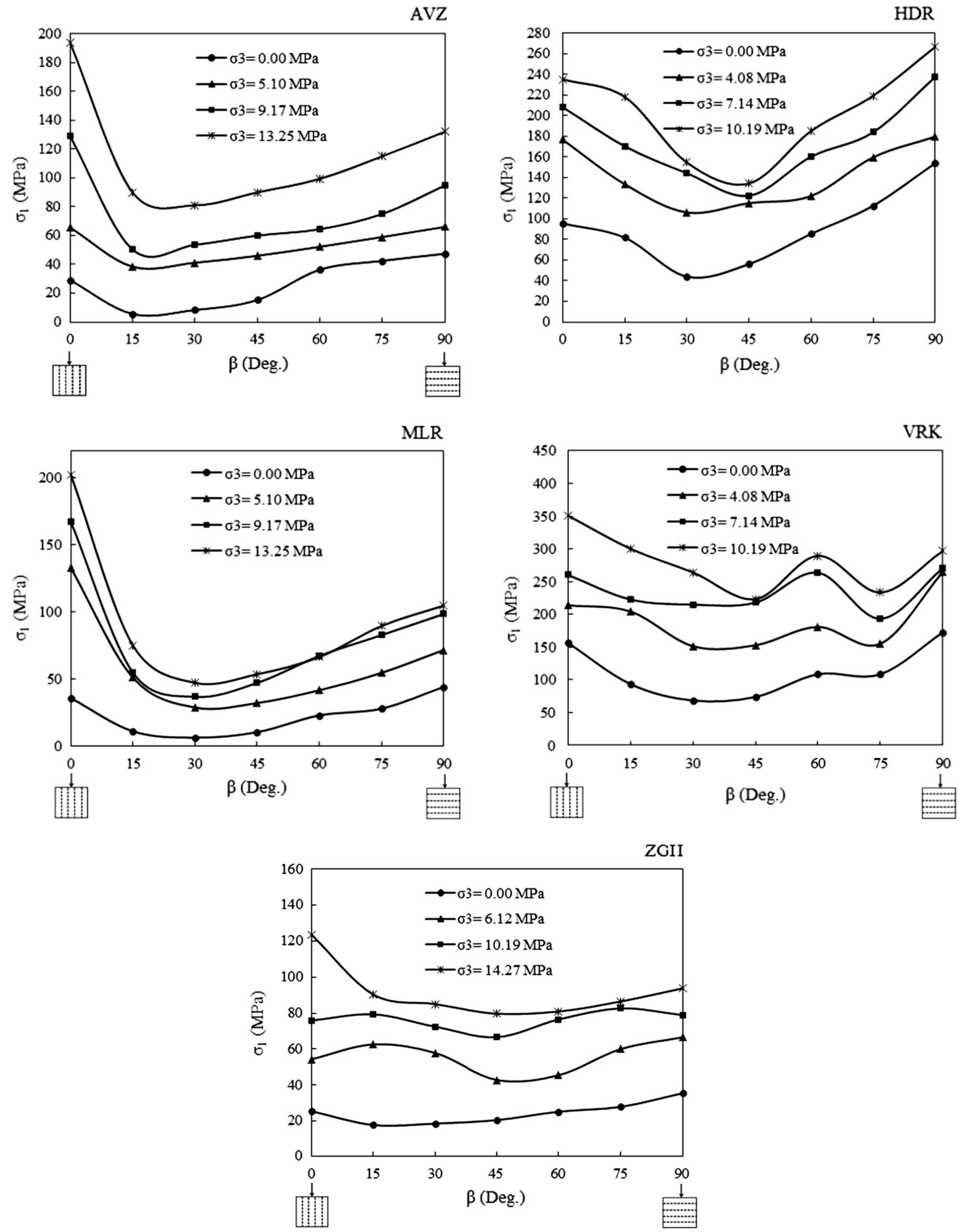

Fig. 2 Correlations between triaxial compressive strength (σ_1), confining pressure (σ_3) and anisotropy angles (β) for tested samples

$\sigma_3 = 13.25$ MPa was obtained at $\beta = 30°$. Other samples had the same status of AVZ except MLR. Therefore, in most studied rocks, with increasing confining pressure, the point of least strength shifts to higher angles of anisotropy.

Furthermore, the maximum unconfined compressive strength of the tested samples has been obtained at $\beta = 90°$, whereas the maximum confined compressive strength can be obtained at $\beta = 0°$. Therefore, with increasing confining pressure, the maximum strength is

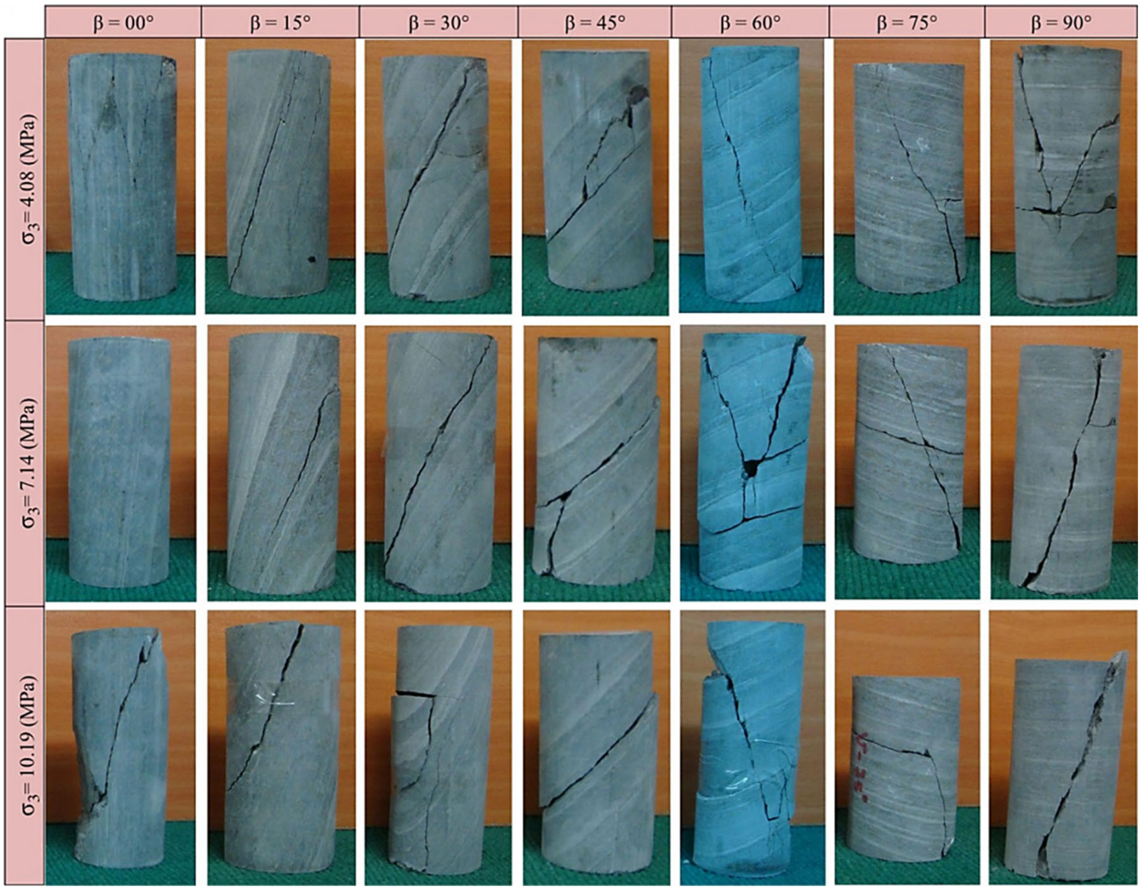

Fig. 3 Failure pattern of the sample of VRK after triaxial compressive strength test

occurred at $\beta = 0°$. This means that the additive rate of σ_1 at $\beta = 0°$ is higher than $\beta = 90°$. Therefore, in high confining pressures, maximum values of σ_1 are always obtained at $\beta = 0°$. Because at $\beta = 0°$, if the confining pressure is high enough, it can prevent buckling of the specimen. This will increase the confined compressive strength. This is not true for the sample of HDR at applied confining pressures. But, the additive rate of σ_1 at $\beta = 0°$ is also higher than $\beta = 90°$ for this sample. Therefore, at higher confining pressure, probably, maximum values of σ_1 will be obtained at $\beta = 0°$. Undulating curves of the samples of VRK and ZGH are due to the specimen and laboratory conditions. In other words, the increase in strength of the sample of VRK at $\beta = 60°$ when compared to $\beta = 45°$ and $\beta = 75°$ is related to specimen and laboratory conditions. It should be noted that the strength of the sample of VRK at $\beta = 60°$ should not be considered for the analysis. This is true for the samples of ZGH at $\beta = 0°$ ($\sigma_3 = 6.12$ MPa and $\sigma_3 = 10.19$ MPa) and $\beta = 90°$ ($\sigma_3 = 10.19$ MPa).

The values of triaxial compressive strength of the tested rocks are also affected by their physical properties. For example, the values of triaxial compressive strength of the samples of HDR and VRK are higher than the other tested samples at all anisotropy angles and confining pressures. This is due to the high density and low porosity of the samples. On the other hand, the values of triaxial compressive strength of the sample of ZGH are lower than the other tested samples because of its low density and high porosity.

Figure 3 shows the failure patterns of the sample of VRK after performing the triaxial compressive strength tests at different σ_3 and β as a representative sample. In this research, failure patterns of all tested samples are similar. After performing the tests, failure planes in specimens are matched to anisotropy planes at $\beta = 0°$, 15°, 30° and 45°. At $\beta = 60°$, 75° and 90°, failure planes are not match to anisotropy planes, but they tend to propagate parallel to anisotropy planes. This failure pattern led to a crashed rock specimen. According to Ramamurthy et al. (1993), when the ratio of σ_c/σ_3 is lower than one, shear failure mechanism alone controls the behavior of anisotropic rocks. In this research, in most tested specimens the ratio of σ_c/σ_3 is higher than one. Thus, at the anisotropy angles less than 45°, the shear failure pattern of tested rock

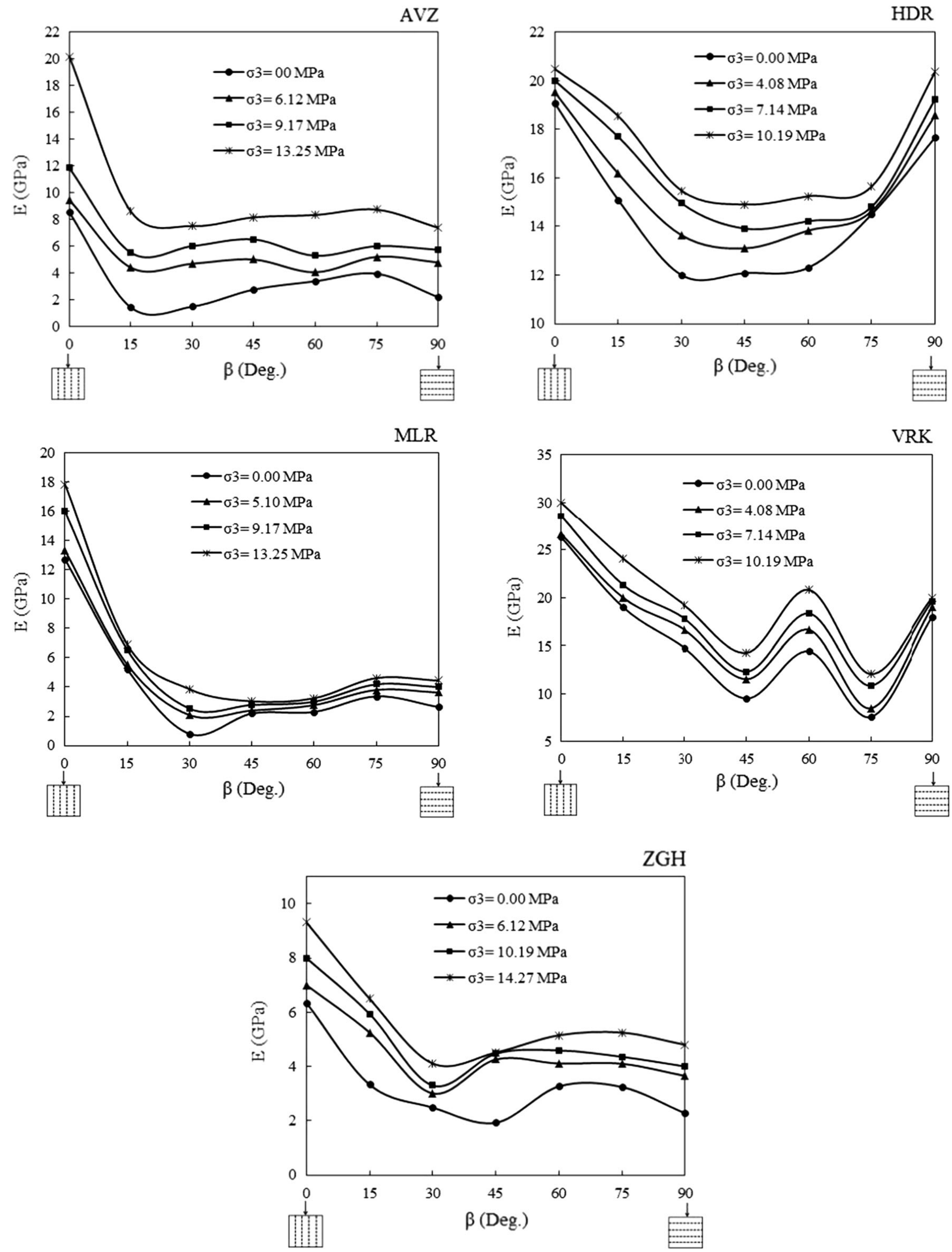

Fig. 4 Correlations between elasticity modulus (E), confining pressure (σ_3) and anisotropy angles (β) for tested samples

specimens is affected by anisotropy planes, whereas at the anisotropy angles more than 60°, the failure pattern of tested rock specimens is usually controlled by shear failure mechanism in intact rock.

3.4 Elasticity Modulus

The calculated elasticity modulus (E) is secant modulus calculated between two points on the linear part of the

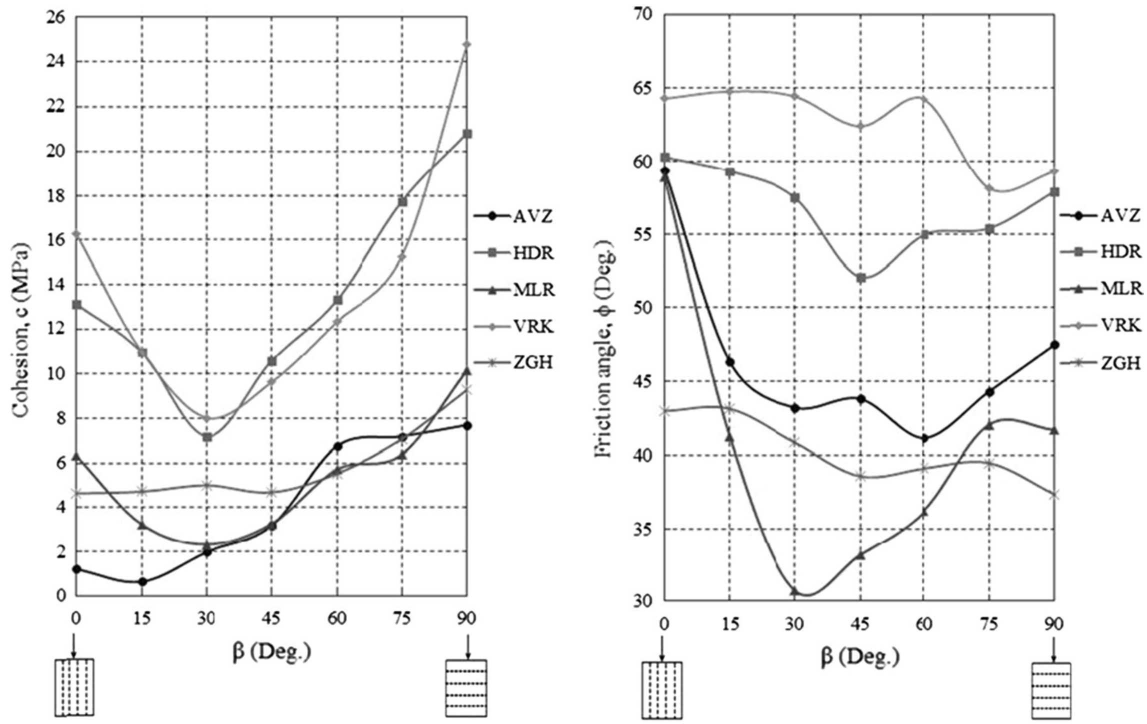

Fig. 5 Correlation curves between shear strength parameters (c and ϕ) and anisotropy angles (β) for tested samples

stress–strain curves at different β and σ_3. Correlation curves between E and β, as presented in Fig. 4, are semi-U shape. The maximum values of E for all tested samples in both unconfined and confined conditions are achieved at $\beta = 0°$. The minimum values of elasticity modulus are obtained at different anisotropy angles. These results are comparable to the results presented by Behrestaghi et al. (1996), Ramamurthy et al. (1998) and Nasseri et al. (2003). In the studied rocks, with increasing confining pressure, the values of elasticity modulus are increased. Furthermore, with increasing confining pressure, in most studied rocks such as the samples of AVZ, HDR and MLR, the point corresponding to the minimum value of elasticity modulus shifts to higher angles of anisotropy. The changing range of elasticity modulus for the samples of HDR and VRK is maximum at $\beta = 30°$ and $\beta = 45°$, respectively, whereas this parameter is maximum at $\beta = 0°$ for the samples of AVZ, MLR and ZGH. This is due to the high anisotropy degrees of the latter samples. In other words, with increasing confining pressure, highly foliated rocks show highest changes in the values of their elasticity modulus at $\beta = 0°$.

Similar to triaxial compressive strength, the values of elasticity modulus of the samples of HDR and VRK are higher than others due to their mineral compositions and physical properties. The increase in elasticity modulus of the sample of VRK at $\beta = 60°$ when compared to $\beta = 45°$ and $\beta = 75°$ is related to specimen and laboratory conditions. It should be noted that the strength of the sample of VRK at $\beta = 60°$ should not be considered for the analysis.

3.5 Shear Strength Parameters

Shear strength parameters contain cohesion (i) and friction angle (ϕ). In this research, the parameters were calculated using the software of the Mohr Circle Application, Version 1.05 (PAA 2010). For this purpose, four circles were drawn at different anisotropy angles (β) and confining pressures (σ_3) for each sample. Correlation curves between shear strength parameters (c and ϕ) and anisotropy angles (β) are presented in Fig. 5. For tested rocks, the maximum values of cohesion and friction angles were obtained at $\beta = 90°$ and $\beta = 0°$, respectively, whereas the minimum values of cohesion and friction angle were obtained at $\beta = 0°$-$30°$ and $\beta = 30°$-$90°$, respectively. In other words, in comparison with friction angle, the minimum values of cohesion have been obtained at lower anisotropy angles. This is due to the anisotropic nature of the rocks and the different behaviors of the competent and incompetent planes in the rocks under different confining pressures. In addition, in the studied anisotropic rocks, minerals in the rock matrix are oriented parallel to anisotropy planes. This affects the values of shear strength parameters on the failure plane at different anisotropy angles. Therefore, the values of shear strength parameters of the rocks are as a function of the

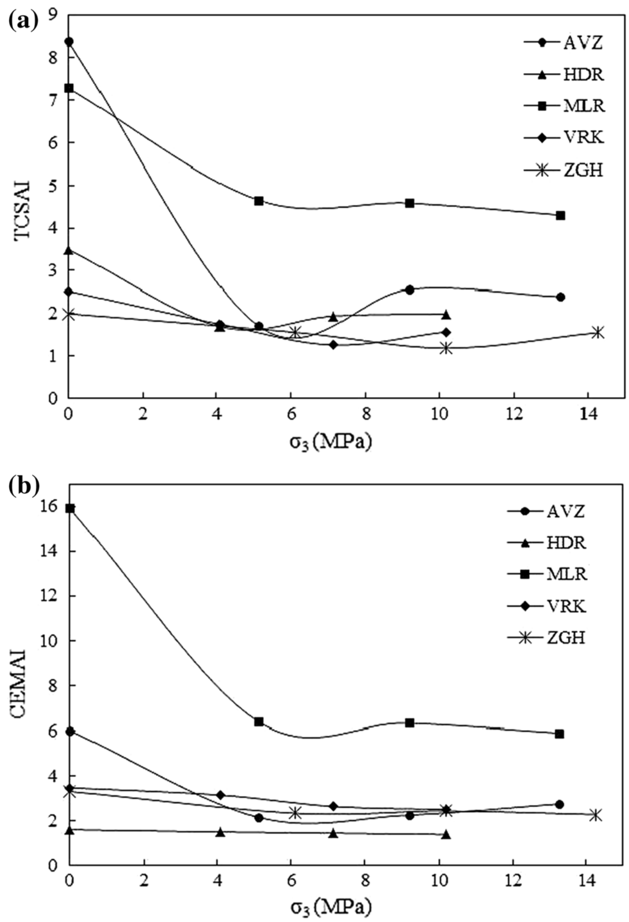

Fig. 6 TCSAI and CEMAI changes with increasing confining pressure in tested rocks

rock anisotropy and mineral orientation of the rock matrix. These results are comparable to the results presented by McCabe and Koerner (1975), Behrestaghi et al. (1996) and Goshtasbi et al. (2006).

3.6 Influence of Confining Pressure on Inherent Anisotropy

The influence of anisotropy is superlative in the unconfined state. The importance of anisotropy decreases with the increasing confining pressure. In this research, two new methods are introduced to determine the influence of confining pressure on inherent anisotropy of rocks. These methods are based on the calculation of Triaxial Compressive Strength Anisotropy Index (TCSAI) and Confined Elasticity Modulus Anisotropy Index (CEMAI). TCSAI is obtained from the ratio of maximum values of triaxial compressive strength to the minimum values at different β and certain σ_3. The high values of the index mean high degrees of rock anisotropy and vice versa. For all tested samples, with increasing σ_3, the values of TCSAI are decreased (especially at $\sigma_3 < 5$ MPa) (Fig. 6a). This means that the effect of rock anisotropy is decreased in all tested rocks.

Also, the effect of confining pressure on the inherent anisotropy of rocks is considerable when CEMAI is calculated. This index is the ratio of the maximum values of elasticity modulus to the minimum values at different β and certain σ_3. The values of CEMAI are decreased with increasing σ_3 (especially at $\sigma_3 < 6$ MPa) (Fig. 6b).

4 Conclusion

The results show that, in addition to anisotropy, the values of triaxial compressive strength (σ_1) and elasticity modulus (E) of the tested rocks are affected by their mineral contents and physical properties. For example, the values of triaxial compressive strength and elasticity modulus of the samples of HDR and VRK are higher than other tested samples at all anisotropy angles and confining pressures.

The shape of the curves between triaxial compressive strength (σ_1) and anisotropy angle (β) is generally U shape. In all tested samples, with increasing confining pressure (σ_3), the values of σ_1 are increased. The minimum values of σ_1 are obtained at different β, so that with increasing σ_3, the anisotropy angle corresponding to the minimum values of σ_1 is increased (except for MLR sample). On the other hand, the anisotropy angle corresponding to the maximum values of σ_1 is variable, so that with increasing σ_3, maximum values of σ_1 are obtained at $\beta = 0°$. In other words, increasing rate of σ_1 at $\beta = 0°$ is higher than other anisotropy angles.

Failure planes in tested specimens under confined condition are matched to anisotropy planes at all anisotropy angles from 0° to 45°. At anisotropy angles from 60° to 90°, failure planes are not matched to the anisotropy planes but they tend to propagate parallel to the anisotropy planes.

Correlations curves between elasticity modulus (E) and anisotropy angle (β) for the tested rocks are semi-U shape. The maximum values of E for all tested samples were obtained at $\beta = 0°$, whereas the minimum values were obtained at different anisotropy angles. With increasing σ_3, the values of E are increased in all tested rocks and the anisotropy angle corresponding to the minimum values of E is increased (except for VRK and ZGH samples).

For tested rocks, the maximum values of cohesion and friction angle were obtained at $\beta = 90°$ and $\beta = 0°$, and the minimum values of these parameters were obtained at $\beta = 0°–30°$ and $\beta = 30°–90°$, respectively. In this research, two new methods are introduced to determine the effect of confining pressure on inherent anisotropy. The results obtained from these methods show that with increasing confining pressure, the effect of inherent anisotropy on the strength behavior of tested rocks is decreased.

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References

Akai K, Yamamoto K, Arioka M (1970) Experimental research on the structural anisotropy of crystalline schists. In: Proceedings of 2nd international congress for rock mechanics 2, pp 181–186

ASTM (1996) Standard test method for triaxial compressive strength of undrained rock core specimens without pore pressure measurements. ASTM Standards on Disc 04.08; Designation D2664–95

Attewell PB, Sandford MR (1974) Intrinsic shear strength of a brittle anisotropic rock—I; experimental and mechanical interpretation. Int J Rock Mech Min Sci 11:423–430

Behrestaghi MHN, Rao KS, Ramamurthy T (1996) Engineering geological and geotechnical responses of schistose rocks from dam project areas in India. Eng Geol 44(1–4):183–201

Brown ET, Richard LR, Barr MV (1977) Shear strength characteristics of Delabole slates. In: Proceedings of conference for rock engineering. Tyne, Newcastle, pp 31–51

Donath FA (1964) Strength variation and deformational behavior of anisotropic rocks. State of stress in the Earth's crust. Elsevier, New York

Goshtasbi K, Ahmadi M, Seyedi J (2006) Anisotropic strength behavior of slates in the Sirjan-Sanandaj zone. J S Afr Inst Min Metall 106:71–76

GSI (Geological Society of Iran) (1977) Geological quadrangle map of Iran. No. D6, Scale 1:250000, Printed by Offset Press Inc. Tehran

Hoek E (1968) Brittle failure of rock. Rock mechanics in engineering practice. Wiley, London

ISRM (2007) The blue book: the complete ISRM suggested methods for rock characterization, testing and monitoring: 1974–2006. In: Ulusay R, Hudson JA (eds) Compilation arranged by the ISRM Turkish National Group, Ankara, Turkey. Kazan Offset Press, Ankara

Li D, Wong LNY, Liu G, Zhang X (2012) Influence of water content and anisotropy on the strength and deformability of low porosity meta-sedimentary rocks under triaxial compression. Eng Geol 126:46–66

McCabe WM, Koerner RM (1975) High pressure shear strength investigation of an anisotropic mica schist rock. Int J Rock Mech Min Sci Geomech Abstr 12:219–228

McLamore R, Gray KE (1967) The mechanical behavior of anisotropic sedimentary rocks. Trans Am Soc Mech Eng B 89:62–76

Nasseri MHB, Rao KS, Ramamurthy T (1997) Failure mechanism in schistose rocks. Int J Rock Mech Min Sci 34:3–4

Nasseri MHB, Rao KS, Ramamurthy T (2003) Anisotropic strength and deformational behavior of Himalayan schists. Int J Rock Mech Min Sci 40:3–23

PAA (Pars Abzar Azmoon) (2010) Mohr Circle Application, Version 1.05. Pars Abzar Azmoon Ltd. http://www.DataLogger.ir. Accessed 30 Mar 2013

Ramamurthy T, Rao GV, Singh J (1988) A strength criterion for anisotropic rocks. In: Proceedings of international symposium for rock great depth, pp 37–44

Ramamurthy T, Rao GV, Singh J (1993) Engineering behavior of phyllites. Eng Geol 33:209–225

Ramamurthy T, Rao GV, Nasseri MHB (1998) Anisotropic strength behavior of Himalayan schists. In: Proceedings of Indian geotechnical conference, Golden Jubilee, New Delhi 1:335–338

Saroglou H, Marinos P, Tsiambaos G (2004a) Applicability of the Hoek-Brown failure criterion and the effect of the anisotropy on intact rock samples from Athens Schist. The J S Afr Inst Min Metall 104(4):209–216

Saroglou H, Marinos P, Tsiambaos G (2004b) The anisotropic nature of selected metamorphic rocks from Greece. J S Afr Inst Min Metall 104(4):217–222

Singh J, Ramamurthy T, Rao GV (1989) Strength anisotropies in rocks. Indian Geotech J 19:147–166

Singh VK, Singh D, Singh TN (2001) Prediction of strength properties of some schistose rocks from petrographic properties using artificial neural networks. Int J Rock Mech Min Sci 38(2):269–284

Zhang XP, Wong LNY, Wang SJ, Han GY (2011) Engineering properties of quartz mica schist. Eng Geol 121:135–149