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ORIGINAL PAPER

Estimation of bearing capacity of basalts at the Atasu dam site, Turkey

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Abstract This paper describes the results of the engineering geological investigations and bearing capacity studies carried out at the proposed site of the rock fill Atasu Dam, to be constructed on basalts and pyroclastics. Rock mass strength and modulus of elasticity of the rock mass were determined using the Hoek–Brown empirical strength criterion. Rock mass classifications for the dam rock foundation were undertaken following the RMR, *Q* and GSI systems and the stress distributions using the finite element technique. To estimate the bearing capacity of the basalts, different empirical equations were used and compared.

Keywords Basalt · Bearing capacity · Hoek–Brown empirical failure criteria

Résumé L'article décrit les résultats des investigations de géologie de l'ingénieur et des études de capacité portante réalisées sur le site du barrage d'Atasu, barrage en enrochements fondé sur des basaltes et des roches pyroclastiques. Les modules d'élasticité et la résistance de la masse rocheuse ont été évalués à partir des critères de Hoek et Brown. L'utilisation des indices RMR, *Q* et GSI a permis de préciser la classe du massif rocheux, la technique des éléments finis permettant de définir la distribution des contraintes induites dans le terrain. Afin d'évaluer la capacité portante des basaltes, différentes équations empiriques ont été utilisées et comparées.

Mots clés Basalte · Capacité portante · Critères de Hoek et Brown

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Introduction

Properties of rock masses are important design parameters for dams. Rock masses are generally heterogeneous, having cracks, fissures, joints, faults and/or bedding planes with varying degrees of strengths along these natural planes of weakness (Merifield et al. 2006). As a consequence, the bearing capacity of dam foundations should be established as accurately as possible for a safe design.

The determination of the bearing capacity of rock masses has traditionally been based on previous experience, using empirical criteria or applying local or national codes. For the determination of realistic values, however, comparison and the use of different models are necessary.

The work reported in this paper was undertaken to determine the bearing capacity of basalts at a dam site in Turkey. In order to investigate the stress distribution at the dam site, stress analyses was performed using commercially available software, ANSYS (1997). Equations suggested by different researchers were used to evaluate the bearing capacity of the bedrock.

Geology of the study area

The Atasu dam, which is under construction on the Galyan river, 4 km SE of Esiroglu town (Fig. 1), was planned to have a height of 116 m and a crest length of 372 m.

Volcanic/sedimentary rocks are dominant at the site ranging from the Upper Cretaceous Caglayan Formation to the Quaternary talus and alluvium deposits (Fig. 2). The Caglayan Formation, as described by Guven (1993), is well exposed around the Caglayan between Macka and Trabzon.

The formation exposed along the Galyan valley (Fig. 2) consists of marl, sandy limestone, red biomicrite, basalt, spilitic basalt and pyroclastics with grey limestone interlayers. The fresh surfaces of the basalts are black and dark grey and alter to yellowish and brownish with weathering. In places they are columnar or have a pillow lava character (Alemdag 2004).

The pyroclastic rocks generally consist of agglomerate, tuffite and breccia. The grain size of the coarser material varies from 100 to 300 mm. The tuffites are generally altered with bedding thicknesses of 0.05 to 1.5 m, dipping 25/N225°. The grey-colored pyroclastics are interbedded with the dark volcanic rocks.

The Quaternary talus is widely distributed on the valley slopes around the Orta Mahalle area (Fig. 2). From the drillings undertaken at the dam site by the State Water Works (DSI 1991), the thickness of the talus varies between 3 and 7 m. These deposits are coarse blocky, pebbly, sandy and clayey in nature, some basaltic blocks being up to a metre across.

Alluvium is present in the Galyan river bed (Fig. 2). Based on the drilling data, it consists of boulders, pebbles, sand and silt, having a combined thickness of 5–8 m. The pebbles and blocks are derived from the basalt, limestone and diorite and may be up to a metre in size.

Engineering geological investigations

The engineering geological investigations included core drilling, discontinuity surveying and laboratory testing.

To determine the vertical and horizontal properties of the units at the dam site, 17 boreholes were drilled in 1991. The rock quality designation (RQD) values of the basalts, obtained following Deere (1964), are presented in Table 1.

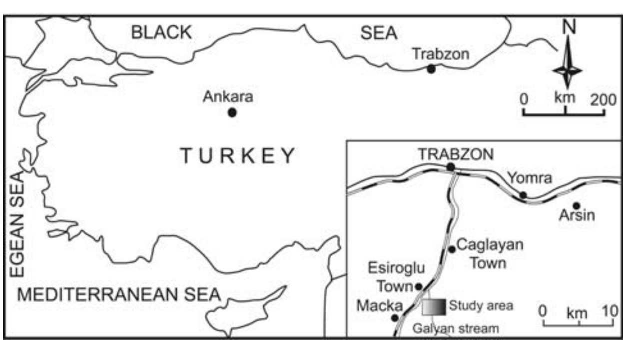

Fig. 1 Location map of the study area

The main orientation of the tectonically induced discontinuities, their spacing, persistence, roughness and filling were determined using the ISRM (1981) criteria and scan-line survey methods. A total of 231 joint measurements were taken from the right and left slopes at the dam site and evaluated using DIPS (Diederichs and Hoek 1989) software (Fig. 3). From the stereographic projection, the major orientations are observed to be 84°/135° (joint set 1) and 81°/037° (joint set 2).

The joint roughness was determined following Barton and Choubey (1977). The degree of weathering of joint faces was determined from Schmidt hammer tests using the weathering index equation proposed by Gokceoglu (1997):

$$W_c = \frac{R_f}{R_w} \quad (1)$$

where

R_f = Schmidt hammer rebound value of unaltered surface, and
R_w = Schmidt hammer rebound value of the weathered joint surface

The histograms obtained from observations at the dam site are shown in Fig. 4. According to ISRM (1981), the joint sets have a very close spacing, low persistence and undulating roughness. They are moderately wide and slightly weathered and contain calcite and clay infilling of up to 5 mm.

The results of the laboratory testing are given in Table 2. Uniaxial compressive strength (σ_c), unit weight (γ), P-wave velocity (C_p) and S-wave velocity (C_s) were established using ISRM (1981) methods. Shear strength parameters (ϕ and C) were determined from triaxial compressive strength tests and the dynamic elasticity modulus (E_d) from Eq. 2 based on the ASTM (1980):

$$E_d = \frac{(1-2v)(1+v)}{(1-v)}\rho C_p^2 \quad (2)$$

where

E_d = dynamic elasticity modulus (MPa)
v = Poisson's ratio, and
ρ = density (g/cm^3).

The Poisson's ratio (v) was calculated from the following equation (ASTM 1980):

$$v = \frac{\left(C_p^2/C_s^2\right) - 2}{2\left[\left(C_p^2/C_s^2\right) - 1\right]}. \quad (3)$$

Rock mass classification systems

Rock mass classification systems are important for describing quantitatively the rock mass quality. The most

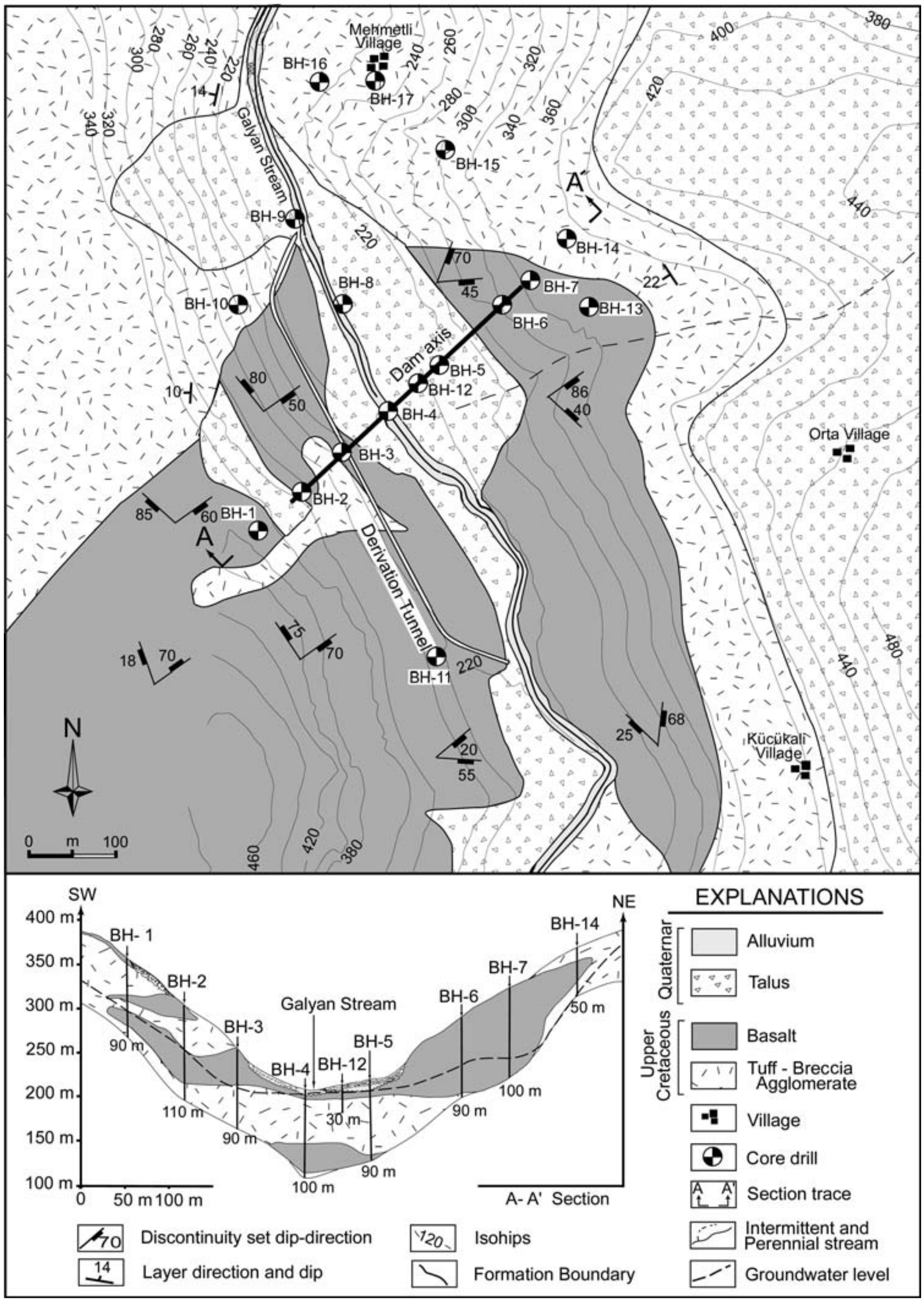

Fig. 2 Geological map and cross-section of the Atasu dam site

Table 1 Percentage distribution of RQD values in the study area

RQD	Rock quality	Distribution (%)
0–25	Very poor	3
25–50	Poor	10
50–75	Fair	30
75–90	Good	40
90–100	Excellent	17

widely used rock mass classification systems—RMR, *Q* and GSI—were used in this research.

Bieniawski (1974) initially developed a rock mass rating (RMR) system based on experiences in tunnel projects in South Africa. Since then, this classification system has undergone significant changes, with ratings added for ground water, joint condition, and joint spacing. The RMR classification established following Bieniawski (1989) was used; the results are summarized in Table 3.

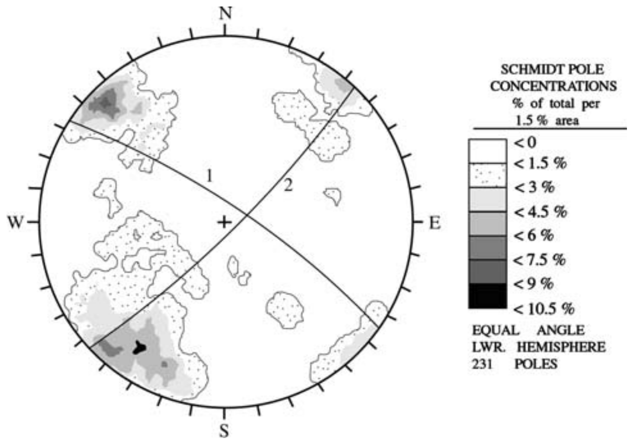

Fig. 3 Stereographic projections of joint sets in the basalts

Barton et al. (1974) developed the Q rock mass classification system, which is also known as the NGI (Norwegian Technical Institute) rock mass classification system. The important parameters are RQD, joint sets (J_n), discontinuity roughness (J_r), joint alteration (J_a), pore water pressure (J_w) and stress reduction (SRF).

$$Q = \frac{\mathrm{RQD}}{J_n}\frac{J_r}{J_a}\frac{J_w}{\mathrm{SRF}} \quad (4)$$

Subsequently, a stress-free form of this equation (Q_N) was suggested by Goel et al. (1995), see Eq. 5:

$$Q_N = \left(\frac{\mathrm{RQD}}{J_n}\right)\left(\frac{J_r}{J_a}\right)J_w \quad (5)$$

In 2002, Barton (2002) reviewed the system and made some changes in the support recommendations. He defined a new parameter, Q_c, to improve correlation between the engineering parameters:

$$Q_c = Q\frac{\sigma_c}{100}. \quad (6)$$

The results of the Q classification system are summarized in Table 4.

The geological strength index (GSI) was developed by Hoek et al. (1995) based on the appearance and structure of the rock mass. Marinos and Hoek (2001) introduced more geological properties into the Hoek–Brown failure criterion and proposed a new GSI chart for heterogeneous weak rock masses. Alternatively, the 1989 version of RMR and Q classification systems can be used to determine the GSI proposed by Hoek et al. (1995):

$$\mathrm{GSI} = \mathrm{RMR}_{89} - 5 \quad (7)$$

$$\mathrm{GSI} = 9\log_e Q' + 44 \quad (8)$$

where RMR_{89} is the latest version of the RMR classification system and Q' is a modified Q given by

$$Q' = \frac{\mathrm{RQD}}{J_n}\frac{J_r}{J_a}. \quad (9)$$

The RMR value of the basalts was 60, $Q = 6.67$, $Q_c = 6$, $Q_{N} = 6.67$. Using Eqs. 7 and 8, the average GSI value is 58.

The Hoek and Brown (1997) failure criterion was used here to determine the rock mass properties of the basalt.

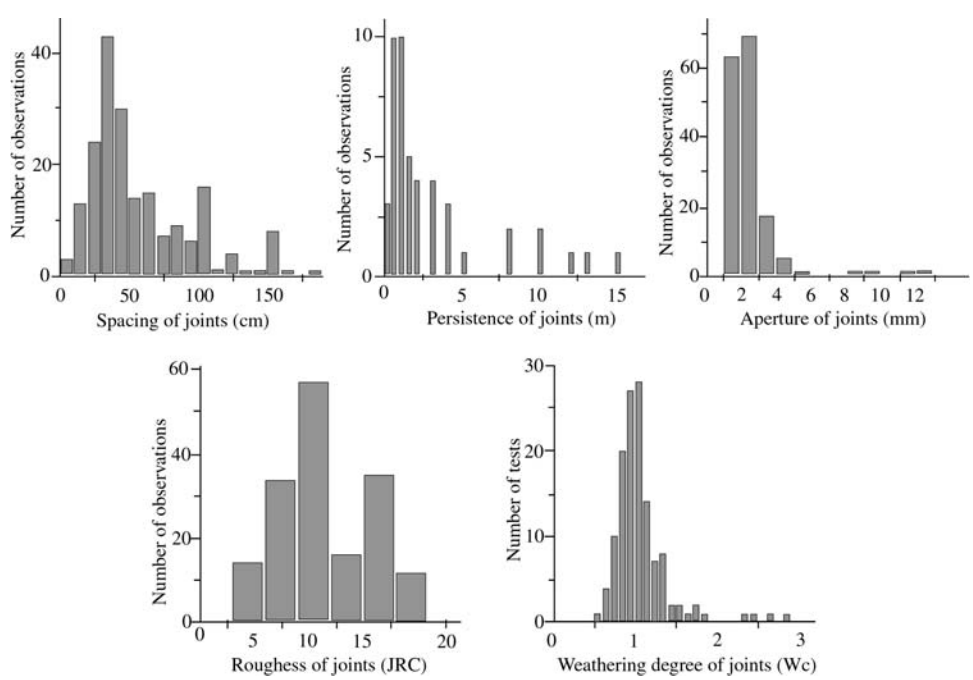

Fig. 4 Histograms for engineering properties of joints

Table 2 Geomechanical properties of the basalts

Properties	Basalt			
	Mean	Min	Max	Stand error
Uniaxial compressive strength,(σ_c (MPa)	90.00	16.60	162.0	4.913
Unit weight (γ, kN/m^3)	28.49	11.8	28.73	0.214
P-wave velocity (C_p, m/s)	3,600	2,451	4,962	67.12
S-wave velocity (C_s, m/s)	1,836	1,250	2,531	34.23
Elasticity modulus (E_d, GPa)	25.50	3.77	57.02	2.611
Poisson's ratio (v)	0.324	0.328	0.298	0.019
Cohesion (c, MPa)	13			
Friction angle ($\phi°$)	34			

Hoek et al. (2002) suggested the following equations for calculating rock mass constants (i.e. m_b, s and a):

$$m_b = m_i \exp\left(\frac{\mathrm{GSI} - 100}{28 - 14D}\right) \tag{10}$$

$$s = \exp\left(\frac{\mathrm{GSI} - 100}{9 - 3D}\right) \tag{11}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(\mathrm{e}^{-\mathrm{GSI}/15} - \mathrm{e}^{-20/3}\right) \tag{12}$$

where D is a factor that depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. In this study the value of D was considered as zero. The calculated GSI and the Hoek–Brown constants are presented in Table 5.

Stress analysis of the dam site

The finite element technique is one of the most reliable methods for determining stresses within and under a rock-fill dam. This method can incorporate dam sections with various material characteristics, irregular foundation layers and different elastic properties. In this study, the dynamic elasticity modulus, Poisson's ratio, unit weight, cohesion and friction angle of the diorite and basalt were used with ANSYS (1997) software.

The height of the dam from the river bed is 116 m and in the model the mesh was extended upstream, downstream and for a depth equal to the height of the dam (Fig. 5a). Solutions were obtained for static loading only and the effect of pore water pressure was not taken into account. The resulting stress distributions are shown in Fig. 5b, which indicates the largest effective stress at the level of the excavation is 1.90 MPa.

Bearing capacity

The bearing capacity was computed from different equations that utilize the Hoek–Brown empirical failure criterion. For example, Kulhawy and Carter (1992) suggested the following equation for calculating the ultimate bearing capacity of a rock mass:

Table 3 RMR_{89} rating for basalts

Classification parameters	Value of parameters	Rating
Uniaxial compressive strength (MPa)	90	9
RQD (%)	80	16
Discontinuity spacing (cm)	35	9
Discontinuity condition		
Persistence (m)	1–3	4
Aperture (mm)	2.50–3.00	1
Roughness	Rough-planar	5
Filling	Calcite	4
Weathering	Slightly weathered	5
Groundwater	Damp	7
Basic RMR value		60
Rock mass quality		Fair rock

Table 4 Q rating for basalts

Classification parameters	Value of parameters	Rating
RQD (%)	80%	80
Joint set number (J_n)	Two joint sets plus random joints	6
Joint roughness number (J_r)	Rough or irregular, planar	1.5
Joint alteration number (j_a)	Slightly altered joint walls	3
Joint water reduction factor (j_w)	Medium inflow or pressure	1
Stress reduction factor (SRF)	Low stress, near surface, open joints	1
Q		6.67
Rock mass quality		Fair rock

Table 5 Rock mass properties of basalts

Parameters	Values
Intact uniaxial compressive strength (σ_c, MPa)	90
Geological strength index (GSI)	58
Hoek–Brown constant of intact rock (m_i)	25
Disturbance factor (D)	0
Hoek–Brown constant of rock mass (m_b)	5.578
Hoek–Brown constant of rock mass (s)	0.0094
Hoek–Brown constant of rock mass (a)	0.503
Strength of rock mass (σ_{cmass}, MPa)	28.76
Modulus of deformation of rock mass (E_m, GPa)	17.57

$$q_u = \sigma_{ci}\left(s^a + \left(m_b s^a + s\right)^a\right) \quad (13)$$

where

σ_{ci} = uniaxial compressive strength of the intact rock (MPa),
s, a = Hoek–Brown constant of the rock mass, and
m_b = Hoek–Brown constant of the intact rock.

Wyllie (1992), on the other hand, suggested the following equation for calculating the ultimate bearing capacity of the rock mass:

$$q_u = \frac{C_{f1} s^{0.5} \sigma_{ci}\left[1 + \left(m_b s^{-0.5} + 1\right)^{0.5}\right]}{F} \quad (14)$$

where

σ_{ci} = uniaxial compressive strength of the intact rock (MPa),
s = Hoek–Brown constant of the rock mass,
m_b = Hoek–Brown constant of the intact rock,

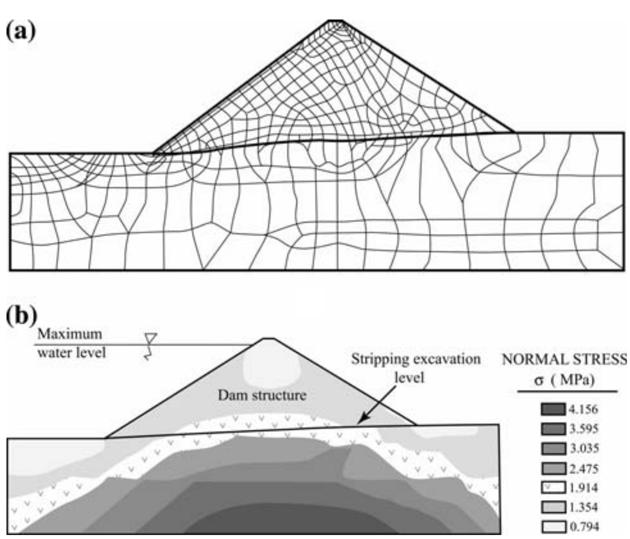

Fig. 5 Stress distributions at the dam site

C_{f1} = factor of correction, and
F = factor of safety.

Serrano and Olalla (1994) proposed the following equation for the evaluation of the ultimate bearing capacity of the rock mass:

$$q_u = \beta_n\left(N_\beta - \zeta_n\right) \quad (15)$$

where

N_β = bearing capacity factor—in this study determined from m_i and the GSI approach,
β_n = strength modulus of the rock mass, and
ζ_n = rock mass toughness

β_n and ζ_n can be calculated using the following equations:

$$\beta_n = \sigma_{ci} A_n \quad (16)$$

and,

$$\zeta_n = \frac{s}{m_b A_n} \quad (17)$$

where

A_n = function of the normalized external load calculated from the following equation:

$$A_n = \left(\frac{m_b(1-a)}{2^{1/a}}\right)^{a/1-a} \quad (18)$$

Merifield et al. (2006) suggested a different equation for ultimate bearing capacity assessment:

$$q_u = \sigma_{ci} N_\sigma \quad (19)$$

where

σ_{ci} = uniaxial compressive strength of the intact rock (MPa), and
N_σ = bearing capacity factor—in this study determined from to m_i and the GSI approach (Merifield et al. 2006).

The ultimate bearing capacities calculated using each of these equations are presented in Table 6.

Conclusions

The rock-fill Atasu Dam will be constructed on basalts with a fair rock mass quality. The ultimate bearing capacity values of the basalts obtained by empirical methods are in the range of 25–90 MPa. According to the equations proposed by Kulhawy and Carter (1992) and Wyllie (1992), the ultimate bearing capacity of the rock mass is almost the same with an average value of 25 MPa. From the equations suggested by Serrano and Olalla (1994) and Merifield et al.

Table 6 Calculated ultimate bearing capacity of the rock mass

Empirical equations	Ultimate bearing capacity (MPa) (3 for factor of safety)
Kulhawy and Carter (1992)	24.93
Wyllie (1992)	25.16
Serrano and Olalla (1994)	87.63
Merifield et al. (2006)	90.00

(2006), the ultimate bearing capacity values average 88.8 MPa. This difference in the bearing capacity results can be justified on the basis of the different parameters involved in the equations. The use of GSI in the equations suggested by Serrano and Olalla (1994) and Merifield et al. (2006) provides an over-estimate of the bearing capacity.

To estimate the stress distribution at the dam site, a commercially available software ANSYS (1997) was used. The largest effective stress at excavation level was determined as 1.9 MPa, which indicates that the Atasu rock-fill dam can be safely constructed at the proposed site.

It is concluded that the use of rock mass strength parameters provides an acceptable estimate of bearing capacity for jointed rock masses. For final design purposes, however, additional geotechnical investigations will be required for the supplementary structures such as the location and the design of the spillway.

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