QUIZ GT401
Rock Mechanics

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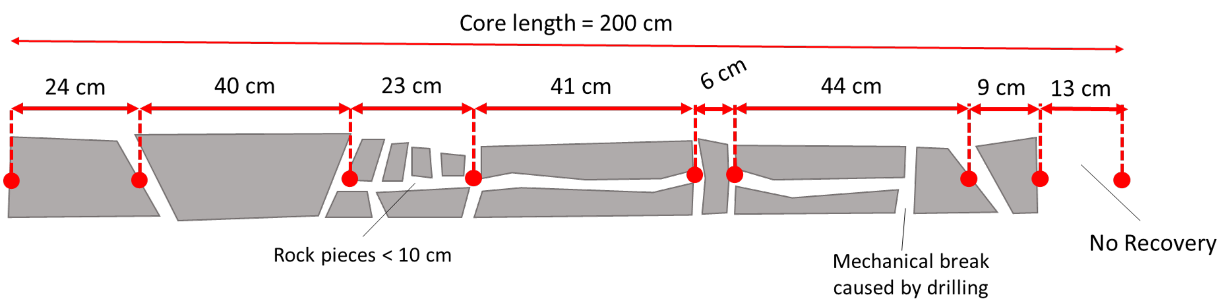
▸ PROBLEMS

PROBLEM 1

A 200-cm borehole log of a rock is illustrated below. Find the RQD for this rock sample.



A) *RQD* = 36.6%

B) *RQD* = 51.8%

C) *RQD* = 74.5%

D) *RQD* = 90.8%

PROBLEM 2

Regarding rock properties, rock classification systems and other aspects of rock mechanics, true or false?

1.( ) There are several ways to measure the degree of jointing. As the joint spacings generally vary greatly, there will be differences in the degree of jointing in an outcrop or a location. Therefore, the characterization of block size should be given as an interval rather than a single value. The RQD and volumetric joint count J_V are less suitable for this, as they, by definition, express an average of the joints where measured. Indeed, it has been shown that there is a poor correlation between the RQD and other types of block size measurements.

2.( ) Palmström and the ISRM have suggested that the rock quality designation can be estimated from the volumetric joint count J_V by means of an equation of the form

$$RQD = 115 - 3.3J_V$$

Later, however, the same author concluded that an expression of the form

$$RQD = 110 - 2.5J_V$$

probably gives more appropriate average correlations than the former. Consider a rock mass that possesses 5 joint sets. The joint counts normal to each set are 12 per 10 m for joint set 1, 9 per 6 m for joint set 2, 8 per 10 m for joint set 3, 7 per 5 m for joint set 4, and 12 per 4 m for joint set 5. The RQD for this rock mass, as estimated from the newer *RQD*-J_V correlation, is greater than 92%.

3.( ) One group of investigators heated two marble and three limestone samples from 100°C to 500°C in 100°C-increments and measured the effective porosity of the rocks at different temperatures. Their results indicated that the effective

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porosity either increases with higher temperature or first slightly decreases until 200°C and then increases with higher temperature. Based on best fitting of the test data, they proposed the following relationship between effective porosity and temperature,

$$n = n_i + aT^2 + bT$$

where n_i is the initial porosity, T is the temperature in °C, and a and b are the regression coefficients, which are listed below for the five tested rocks. Appealing to these data, a sample of porous limestone heated to 400°C will have a porosity greater than 3.75%.

Rock	n_i (%)	a ($\times 10^{-4}$)	b ($\times 10^{-4}$)
Intramicritic limestone	7.86	0.2	−13
Porous limestone (travertine)	3.43	0.4	−144
Crystalized limestone	0.66	0.1	−21
Fine grain (0.1 – 1 mm) marble	0.30	0.1	−5
Coarse grain (0.5 – 2 mm) marble	0.24	0.07	+5

4.( ) While the porosity of an intact rock mass tends to decrease with increasing depth, the density tends to increase as depth increases. One author has proposed the following relation for variation in density of sandstones and siltstones with depth,

$$\rho = 2.72 - 1.244\exp(-0.846z)$$

where ρ is in g/cm^3 and z is in km. From this relation, we can state that, in order for the density of the intact rock to be greater than 2.0 g/cm^3, the depth must be at least 750 m.

5.( ) It has been observed that, for similar specimen geometry, the uniaxial compressive strength of rock material varies with specimen volume. Generally, it is observed that the UCS increases with increasing specimen volume, except at very small specimen sizes where inaccuracy in specimen preparation and surface flaws or contamination may dominate behavior and cause a strength increase with decreasing specimen volume.

6.( ) Many researchers have studied the effect of water content on the strength of intact rocks. The unconfined compressive strength of intact rocks decreases as the water content increases and their relationship can be described by the negative exponential function

$$\sigma_c = ae^{-bw} + c$$

where w is the water content in %, and a, b and c are constants. Here, σ_c is given in MPa. The following table lists the values of a, b and c for some types of rocks. With reference to these data, we conclude that, if we were to increase the water content of a gypsum sample from 2% to 7%, its UCS would decrease by more than 8%.

A	*b*	*c*	Rock type
4.16 – 84.01	0.0752– 6.147	2.97 – 231	15 British sandstones
83.59	0.4433	0	Coal measures, mudrock (clayshale, mudstone, and mudshale)
14.68	0.8193	24.0	Gypsum

7.( ) The tensile strength of a rock mass can be estimated using empirical relations such as the one below,

$$\sigma_t = -0.029\gamma f_c Q^{0.3}$$

where $f_c = \sigma_c/100$ for $Q > 10$ and $\sigma_c > 100$ MPa, otherwise $f_c = 1$, and γ is the density of rock mass in g/cm^3. In this equation, σ_t is given in MPa. For a granite with density of 2.70 g/cm^3, UCS of 120 MPa and Q-rating of 15, the tensile strength as estimated from the relation above is greater than 250 kPa.

8.( ) Some intact rocks, such as those composed of parallel arrangements of flat minerals like mica, chlorite and clay, show strong strength anisotropy. The maximum strength of a rock mass is generally found when the major principal stress is nearly perpendicular or parallel to the stratification plane.

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9.(   ) Bieniawski studied seven projects and suggested the following correlation for estimating the rock mass deformation modulus from the Rock Mass Rating (RMR),

$$E_m = 2RMR - 100$$

A few years later, Serafim and Pereira proposed the relation

$$E_m = 10^{(RMR-10)/40}$$

Both of these correlations were proposed before 1989. Accordingly, the RMR score to be used with these expressions is the rating from the 1976 version of the RMR system, which, as a rule of thumb, is to be taken as the score from the 1989 version of the RMR plus 5.

10.(   ) The *Q*-system, as originally conceived by the Norwegian Geotechnical Institute in the 1970s, does not include parameters such as joint orientation, joint persistence, and joint aperture.

PROBLEM 3

A tunnel is to be driven through a weathered quartz with the following characteristics:

→ A point-load strength index of 4 MPa.
→ A RQD of 65%.
→ Spacing between joints is 450 mm.
→ Joints are slightly rough and have a separation less than 1 mm. Walls are highly weathered.
→ Tunneling conditions are anticipated to be damp.
→ The dominant joint set dips at 30° against the direction of the drive.

What is the RMR of this rock mass?

A) *RMR* = 29

B) *RMR* = 48

C) *RMR* = 63

D) *RMR* = 80

PROBLEM 4

A dam foundation is to be built upon a strong limestone with the following characteristics:

→ Uniaxial compressive strength of 80 MPa.
→ A RQD of 80%.
→ Spacing between joints is 80 mm.
→ The joint set has been described as follows:
  → Average discontinuity length is 4 m.
  → Separation of joints is generally less than 0.1 mm.
  → Joints are slightly rough and moderately weathered.
  → Joints have no infilling.
→ Groundwater conditions are assumed to be dripping.
→ The discontinuity orientation was deemed fair.

What is the RMR of this rock mass?

A) *RMR* = 31

B) *RMR* = 53

C) *RMR* = 70

D) *RMR* = 89

PROBLEM 5

A small cavern will be excavated in a mass of norite wiith UCS of 160 MPa. The overburden stress is expected to reach 10 MPa at most, and the ratio of the horizontal principal stress to the vertical principal stress is K = 4/3. Other characteristics of the rock mass are listed below:

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→ The RQD is 80%.
→ The rock mass contains one joint set controlling stability.
→ Joints are rough, irregular and undulating.
→ There are unaltered joint walls with some surface staining.
→ There is medium hydraulic inflow with occasional outwash of joint fillings.
→ Stress conditions were deemed favorable to the excavation.

What is the rating of this rock mass in the *Q*-system? Use the average stress reduction factor (SRF) if Table 6 happens to suggest a range of values. Further, use the *new* SRF values if the stress field category is associated with old and new values. The latter values were taken from the 1993 review of the system.

A) Q = 0.86

B) Q = 15.1

C) Q = 79.2

D) Q = 141

PROBLEM 6

A small cavern will be excavated a mass of limestone wiith UCS of 39 MPa. The characteristics of the rock mass are as follows:

→ The RQD is 60%.
→ The rock mass contains one joint set controlling stability plus a subset of random joints.
→ Joints are smooth and planar.
→ Rock wall contact before 10 cm shear. Strong overconsolidation. There are thin, continuous fillings of non-softening clay mineral.
→ The excavation intersects multiple weakness zones containing non-swelling clay minerals and disintegrated rock.
→ There is large hydraulic inflow with substantial outwash of joint fillings.

What is the rating of this rock mass in the *Q*-system? Use the average stress reduction factor (SRF) if Table 6 happens to suggest a range of values. Further, use the *new* SRF values if the stress field category is associated with old and new values. The latter values were taken from the 1993 review of the system.

A) Q = 0.11

B) Q = 5.8

C) Q = 67.5

D) Q = 120

PROBLEM 7

The following characteristics apply to a siltstone rock:

→ Uniaxial compressive strength of 25 MPa.
→ Block volume of 0.002 m^3.
→ Smooth joint surfaces with slightly undulating joint plane.
→ Presence of a pronounced 2-m continuous joint.
→ Joint is thinly coated with a portion of sand and silt, but not clay.

Determine the Rock Mass Index (RMi) of the rock.

A) RMi = 0.503

B) RMi = 0.907

C) RMi = 5.64

D) RMi = 10.6

PROBLEM 8

The following characteristics apply to a limestone rock:

→ Uniaxial compressive strength of 50 MPa.
→ Block volume of 0.005 m^3.
→ Very rough, strongly undulating joints.
→ Presence of several 0.1- to 0.4-m continuous joints.
→ Fresh joint walls.

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Determine the Rock Mass Index (RMi) of the rock.

A) *RMi* = 0.414

B) *RMi* = 1.52

C) *RMi* = 4.78

D) *RMi* = 14.1

PROBLEM 9

A point load test was carried out on a 70-mm cross-section cylindrical sandstone core. The sample withstood a load of 14.7 kN. The following table lists correlations between the corrected point load index and the uniaxial compressive strength for some types of rocks. The UCS for the rock in question is:

Correlation	Rock Type
σ_c = 23.5$I_{s(50)}$	Amphibolite
σ_c = 23.3$I_{s(50)}$	Gabbro
σ_c = 15.8$I_{s(50)}$	Khondalite
σ_c = 21.9$I_{s(50)}$	Sandstone
σ_c = 14.4$I_{s(50)}$	Shale

A) σ_c = 19.8 MPa

B) σ_c = 40.6 MPa

C) σ_c = 76.4 MPa

D) σ_c = 91.5 MPa

PROBLEM 10

Laboratory test data show that unconfined compressive and tensile strengths for a rock are 105 MPa and 12 MPa, respectively. Field measurements indicate a joint persistence of 0.75. Further laboratory testing shows that joint cohesion and friction angle are 0.09 MPa and 26°, respectively. Using the Terzaghi jointed rock mass model, estimate the cohesion and angle of internal friction for intact rock tested in the laboratory, then determine rock mass values of friction angle, cohesion, unconfined compressive strength, and tensile strength. True or false?

1.( ) The angle of internal friction of the rock mass is greater than 40°.

2.( ) The cohesion of the rock mass is greater than 4 MPa.

3.( ) The unconfined compressive strength of the rock mass is greater than 60 MPa.

4.( ) The tensile strength of the rock mass is greater than 20 MPa.

PROBLEM 11

The rock block in biaxial stress is subjected to compressive stresses σ_x = −24 MPa and σ_y = −12 MPa (the negative sign denotes compression). The corresponding strains are ϵ_x = −440 × 10^{-6} and ϵ_y = −80 × 10^{-6} (the negative sign denotes shortening). Determine the modulus of elasticity for the rock and, referring to the table below, answer: which type of rock is the block made of?

Rock Type	Mean Elastic Modulus (GPa)
Granite	53
Gabbro	76
Diabase	88
Quartzite	66
Marble	45
Slate	10
Schist	34
Sandstone	15
Limestone	39
Dolostone	29

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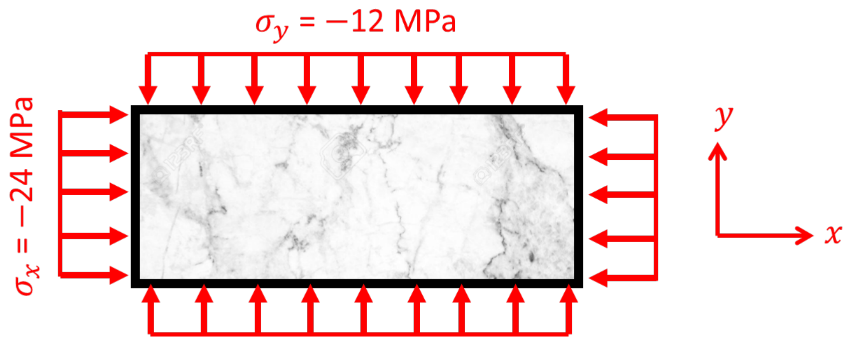


A) Schist.

B) Marble.

C) Granite.

D) Gabbro.

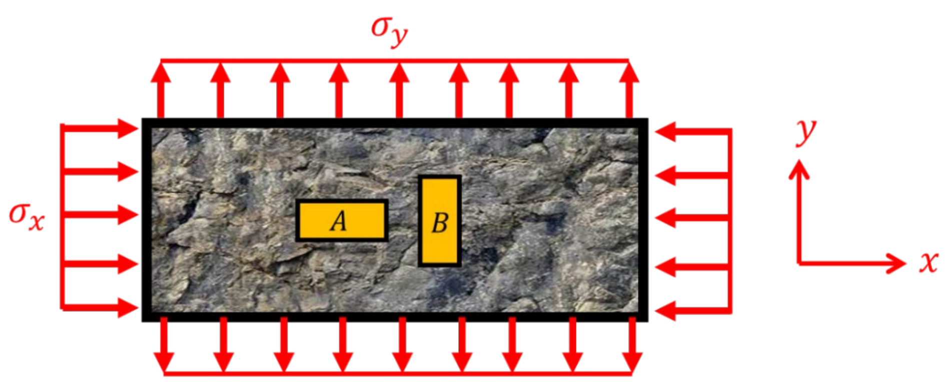
PROBLEM 12

The block shown below has a dimension t = 5 m perpendicular to the plane of the figure and is made of granite with density 2.75 g/cm^3. The block is subjected to uniform normal stresses σ_x and σ_y. Strain gages A and B oriented in the x- and y-directions, respectively, are attached to the block and give normal strains ϵ_x = −2 × 10^{-4} (shortening) and ϵ_y = 5 × 10^{-5} (elongation). Find the change in the z-dimension of the block, i.e. perpendicular to the plane of the figure. Bear in mind that tensile stresses and elongation strains are positive, while compressive stresses and shortening strains are negative. Select a mean Poisson's ratio from the following table and estimate Young's modulus from the Deere and Miller correlation

$$E = 64.64\rho_d - 115.4$$

where ρ_d is the density in g/cm^3 and E is in GPa.

Rock Type	Poisson's ratio
Basalt	0.23
Diabase	0.29
Gabbro	0.18
Granite	0.20
Limestone	0.23
Sandstone	0.20
Schist	0.12
Shale	0.09



A) Δt = −0.352 mm

B) Δt = −0.189 mm

C) Δt = +0.189 mm

D) Δt = +0.352 mm

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PROBLEM 13A

Consider a cemented breccia with Geological Strength Index (GSI) of 75. With regard to the Hoek-Brown failure criterion, the *m*-parameter for the intact rock is m_i = 16.3 and the uniaxial compressive strength is σ_{ci} = 51 MPa. Estimate the UCS of the rock mass. Let *D* = 0.

A) σ_{cm} = 12.7 MPa

B) σ_{cm} = 19.6 MPa

C) σ_{cm} = 26.5 MPa

D) σ_{cm} = 33.1 MPa

PROBLEM 13B

In 2001, Marinos and Hoek published the following correlation to estimate the uniaxial compressive strength of a rock mass,

$$\sigma_{\text{cm}} = \sigma_{\text{ci}} \times 0.0034 m_i^{0.8} \times \left[1.029 + 0.025 \exp\left(-0.1 m_i\right)\right]^{\text{GSI}}$$

Use this correlation to compute the UCS of the rock mass considered in the previous part.

A) σ_{cm} = 12.4 MPa

B) σ_{cm} = 19.7 MPa

C) σ_{cm} = 26.3 MPa

D) σ_{cm} = 33.0 MPa

PROBLEM 13C

Estimate the tensile strength of the rock mass.

A) σ_{tm} = 15.8 kPa

B) σ_{tm} = 30.4 kPa

C) σ_{tm} = 44.9 kPa

D) σ_{tm} = 68.7 kPa

PROBLEM 13D

Estimate the Mohr-Coulomb failure criterion properties ϕ' and *c'* of the rock mass.

A) ϕ' = 23.2° and c' = 5.67 MPa

B) ϕ' = 23.2° and c' = 10.3 MPa

C) ϕ' = 35.1° and c' = 5.67 MPa

D) ϕ' = 35.1° and c' = 10.3 MPa

In the following shaft-related problems, compressive stresses are positive. Notations σ_V, σ_H and σ_h refer to the vertical, major horizontal and minor horizontal stresses, respectively. Assume compass coordinates, i.e. *x* = east, *y* = north and *z* = up.

PROBLEM 14A (Pariseau, 2007, w/ permission)

A 6 m diameter circular shaft is planned in massive rock. Laboratory tests on core from exploration drilling show that the compressive strength of the rock is σ_c = 152 MPa, the tensile strength is σ_t = 8.3 MPa, the unit weight of rock is γ = 23 kN/m^3, Young's modulus of the rock E = 34.5 GPa, and the shear modulus of the rock is *G* = 13.8 GPa. The depth of the excavation is 915 m. Site measurements show that no tectonic stresses are present. Determine the factors of safety with respect to failure in compression in the vertical and horizontal directions.

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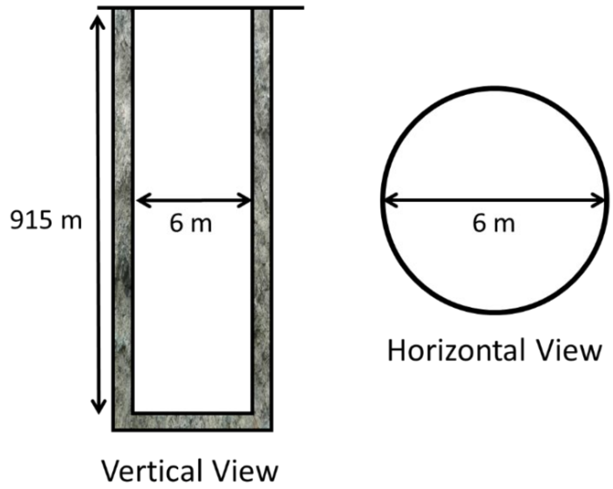


A) $(FS_c)_V$ = 3.62 and $(FS_c)_H$ = 5.45
B) $(FS_c)_V$ = 3.62 and $(FS_c)_H$ = 10.9
C) $(FS_c)_V$ = 7.24 and $(FS_c)_H$ = 5.45
D) $(FS_c)_V$ = 7.24 and $(FS_c)_H$ = 10.9

PROBLEM 14B

If the opening in the previous problem is in an *in situ* stress field such that σ_V = σ_H, what is the new factor of safety with respect to compression in the horizontal?

A) $(FS_c)_H$ = 1.79
B) $(FS_c)_H$ = 3.62
C) $(FS_c)_H$ = 5.45
D) $(FS_c)_H$ = 7.37

PROBLEM 15 (Pariseau, 2007, w/ permission)

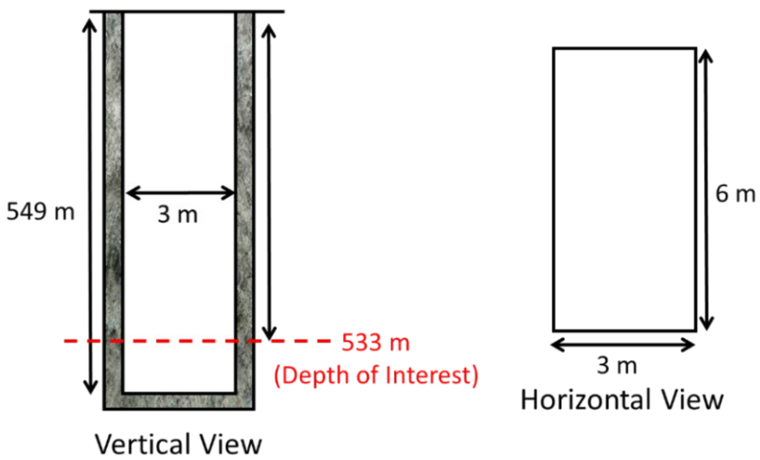
A rectangular shaft 3 × 6 m with the long axis parallel to the N-S line exists at a depth of 290 m. The mining plan calls for deepening the shaft to 549 m. Estimate safety factors for the shaft wall at a depth of 533 m. The rock has unconfined compressive strength σ_c = 164 MPa, tensile strength σ_t = 10.2 MPa, Young's modulus E = 36.5 GPa, Poisson's ratio ν = 0.27, and unit weight γ = 26.6 kN/m^3. The premining stress state relative to compass coordinates is

$$\sigma_E = 2414 + 4.5h$$

$$\sigma_N = 2897 + 7.9h$$

$$\sigma_V = 25.3h$$

where stresses are in kPa, h is the depth in meters, and E, N and V refer to compass coordinates. Premining shear stresses are nil relative to compass coordinates.



A) $(FS_c)_V$ = 6.52 and $(FS_c)_H$ = 4.43
B) $(FS_c)_V$ = 6.52 and $(FS_c)_H$ = 8.18
C) $(FS_c)_V$ = 12.1 and $(FS_c)_H$ = 4.43
D) $(FS_c)_V$ = 12.1 and $(FS_c)_H$ = 8.18

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PROBLEM 16A (Pariseau, 2007, w/ permission)

A rectangular shaft 3 m wide by 6 m long is sunk vertically in ground where the state of pre-mining stress involves the following stress variables.

$$\sigma_{xx} = 7.9 \text{ MPa}$$

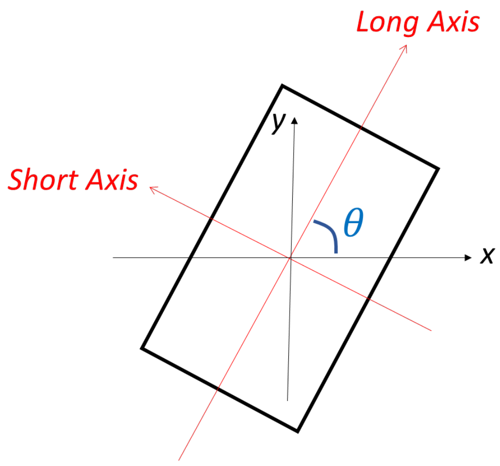
$$\sigma_{yy} = 14.2 \text{ MPa}$$

$$\sigma_{zz} = 11.0 \text{ MPa}$$

$$\tau_{xy} = 1.5 \text{ MPa}$$

$$\tau_{yz} = \tau_{zx} = 0$$

with compression positive, x = east, y = north, and z = up. Rock properties are Young's modulus E = 31.0 GPa, Poisson's ratio ν = 0.20, compressive strength σ_c = 103 MPa and tensile strength σ_t = 6.2 MPa. Find the most favorable orientation of the shaft. As your answer, provide the angle θ that the long axis should make with the x-axis, as illustrated below.



A) θ = 32.5°
B) θ = 47.1°
C) θ = 62.0°
D) θ = 77.3°

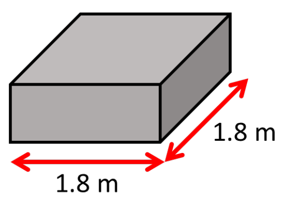
PROBLEM 16B

Find the factor of safety with respect to compression in the horizontal at the optimum orientation.

A) $(FS_c)_H$ = 1.55
B) $(FS_c)_H$ = 3.72
C) $(FS_c)_H$ = 5.94
D) $(FS_c)_H$ = 7.11

PROBLEM 17A (Pariseau, 2017, w/ permission)

Consider a square footing on a jointed rock mass that contains 3 orthogonal joint sets. The strata dip 45 degrees, although the foundation surface is flat. A footing 1.8 × 1.8 m square is subjected to a vertical stress of 12 MPa. Estimate the settlement expected when joint spacing is 0.9 m. Properties of the rock are given in Table 14.



A) δ = 5.78 mm
B) δ = 10.4 mm
C) δ = 15.2 mm
D) δ = 20.8 mm

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PROBLEM 17B

Estimate the settlement expected if the joint spacing for Part A is 0.18 m. Properties of the rock remain as given in Table 14.

A) δ = 10.7 mm

B) δ = 15.9 mm

C) δ = 20.8 mm

D) δ = 25.6 mm

PROBLEM 17C

Estimate the settlement expected in Part A if, instead of the properties given in Table 14, the rock has E = 42 GPa and ν = 0.23.

A) δ = 1.78 mm

B) δ = 3.93 mm

C) δ = 5.94 mm

D) δ = 7.72. mm

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▸ ADDITIONAL INFORMATION

Table 1 Rock Mass Rating (RMR) system

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS

	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4 - 10 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100 - 250 MPa	50 - 100 MPa	25 - 50 MPa	5 - 25 MPa	1 - 5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6 - 2 . m	200 - 600 mm	60 - 200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10 - 25	25 - 125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1, - 0.2	0.2 - 0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)

Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS

Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES

Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300 - 400	200 - 300	100 - 200	< 100
Friction angle of rock mass (deg)	> 45	35 - 45	25 - 35	15 - 25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions

Discontinuity length (persistence)	< 1 m	1 - 3 m	3 - 10 m	10 - 20 m	> 20 m
Rating	6	4	2	1	0
Separation (aperture)	None	< 0.1 mm	0.1 - 1.0 mm	1 - 5 mm	> 5 mm
Rating	6	5	4	1	0
Roughness	Very rough	Rough	Slightly rough	Smooth	Slickensided
Rating	6	5	3	1	0
Infilling (gouge)	None	Hard filling < 5 mm	Hard filling > 5 mm	Soft filling < 5 mm	Soft filling > 5 mm
Rating	6	4	2	2	0
Weathering	Unweathered	Slightly weathered	Moderately weathered	Highly weathered	Decomposed
Ratings	6	5	3	1	0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**

Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip - Dip 45 - 90°	Drive with dip - Dip 20 - 45°	Dip 45 - 90°	Dip 20 - 45°
Very favourable	Favourable	Very unfavourable	Fair
Drive against dip - Dip 45-90°	Drive against dip - Dip 20-45°	Dip 0-20 - Irrespective of strike°	
Fair	Unfavourable	Fair	

* Some conditions are mutually exclusive . For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.
** Modified after Wickham et al (1972).

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Table 2 Joint set number (J_n) for use with the *Q*-system

	Condition	J_n
A	Massive, no or few joints	0.5–1.0
B	One joint set	2
C	One joint set plus random	3
D	Two joint sets	4
E	Two joint sets plus random	6
F	Three joint sets	9
G	Three joint sets plus random	12
H	Four or more joint sets, random, heavily jointed, "sugar cube," etc.	15
J	Crushed rock, earth-like	20

For intersections use ($3.0 \cdot J_n$). For portals use ($2.0 \cdot J_n$).

Table 3 Joint roughness number (J_r) for use with the *Q*-system

	Condition	J_r
	(a) Rock wall contact and ***(b) Rock wall contact before 10 cm shear***	
A	Discontinuous joint	4.0
B	Rough or irregular, undulating	3.0
C	Smooth, undulating	2.0
D	Slickensided, undulating	1.5
E	Rough or irregular, planar	1.5
F	Smooth, planar	1.0
G	Slickensided, planar	0.5
	(c) No rock wall contact when sheared	
H	Zone containing clay minerals thick enough to prevent rock wall contact	1.0
J	Sandy, gravelly, or crushed zone thick enough to prevent rock wall contact	

Descriptions refer to small-scale features and intermediate-scale features, in that order. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m. $J_r = 0.5$ can be used for planar, slickensided joints having lineation, provided the lineations are favorably oriented. J_r and J_a classification is applied to the joint set or discontinuity that is least favorable for stability both from the point of view of orientation and shear resistance, τ.

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Table 4 Joint alteration number (J_a) for use with the *Q*-system

	Condition	ϕ_r approx. (degree)	J_a
	(a) Rock wall contact (no mineral filling, only coating)		
A	Tightly healed, hard, non-softening, impermeable filling, i.e., quartz or epidote		0.75
B	Unaltered joint walls, surface staining only	25–35	1.0
C	Slightly altered joint walls; non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	25–30	2.0
D	Silty or sandy clay coatings, small clay fraction (non-softening)	20–25	3.0
E	Softening or low friction clay mineral coatings, i.e., kaolinite and mica; also chlorite, talc, gypsum, and graphite, etc., and small quantities of swelling clays (discontinuous coatings, 1–2 mm or less in thickness)	8–16	4.0
	(b) Rock wall contact before 10 cm shear (thin mineral fillings)		
F	Sandy particles, clay-free disintegrated rock, etc.	25–30	4.0
G	Strongly over-consolidated, non-softening clay mineral fillings (continuous, <5 mm in thickness)	16–24	6.0
H	Medium or low over-consolidation, softening, clay mineral fillings (continuous, <5 mm in thickness)	12–16	8.0
J	Swelling clay fillings, i.e., montmorillonite (continuous, <5 mm in thickness); value of J_a depends on percent of swelling clay-size particles, and access to water, etc.	6–12	8–12
	(c) No rock wall contact when sheared (thick mineral fillings)		
K, L, M	Zones or bands of disintegrated or crushed rock and clay (see G, H, J for description of clay condition)	6–24	6, 8, or 8–12
N	Zones or bands of silty or sandy clay, small clay fraction (non-softening)	—	5.0
O, P, R	Thick, continuous zones or bands of clay (see G, H, J for description of clay condition)	6–24	10, 13, or 13–20

Table 5 Joint water reduction factor (J_w) for use with the *Q*-system

/

	Condition	Approx. water pressure (MPa)	J_w
A	Dry excavation or minor inflow, i.e., 5 lt./min locally	<0.1	1
B	Medium inflow or pressure, occasional outwash of joint fillings	0.1–0.25	0.66
C	Large inflow or high pressure in competent rock with unfilled joints	0.25–1.0	0.5
D	Large inflow or high pressure, considerable outwash of joint fillings	0.25–1.0	0.33
E	Exceptionally high inflow or water pressure at blasting, decaying with time	>1.0	0.2–0.1
F	Exceptionally high inflow or water pressure continuing without noticeable decay	>1.0	0.1–0.05

Factors C to F are crude estimates. Modify Jw if drainage measures are installed.
Special problems caused by ice formation are not considered.
For general characterization of rock masses distant from excavation influences, the use of J_w = 1.0, 0.66, 0.5, 0.33, etc., as depth increases from, say, 0–5, 5–25, 25–250 to >250 m is recommended, assuming that RQD/J_n is low enough (e.g., 0.5–25) for good hydraulic conductivity. This will help to adjust Q for some of the effective stress and water softening effects in combination with appropriate characterization values of SRF. Correlations with depth-dependent static modulus of deformation and seismic velocity will then follow the practice used when these were developed.

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Table 6 Stress reduction factor (*SRF*) for use with the *Q*-system

	Conditions			SRF	
	(a) Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated				
A	Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)			10.0	
B	Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation $\leq$50 m)			5.0	
C	Single-weakness zones containing clay or chemically disintegrated rock (depth of excavation $>$50 m)			2.5	
D	Multiple-shear zones in competent rock (clay-free), loose surrounding rock (any depth)			7.5	
E	Single-shear zones in competent rock (clay-free) (depth of excavation $\leq$50 m)			5.0	
F	Single-shear zones in competent rock (clay-free) (depth of excavation $>$50 m)			2.5	
G	Loose, open joints, heavily jointed or "sugar cube," etc. (any depth)			5.0	
	(b) Competent rock, rock stress problems				
		q_c/σ_1	**σ_θ/q_c**	**SRF (old)**	**SRF (new)**
H	Low stress, near surface, open joints	$>$200	$<$0.01	2.5	2.5
J	Medium stress, favorable stress condition	200–10	0.01–0.3	1.0	1.0
K	High stress, very tight structure; usually favorable to stability, may be unfavorable to wall stability	10–5	0.3–0.4	0.5–2.0	0.5–2.0
L	Moderate slabbing after $>$1 hour in massive rock	5–3	0.5–0.65	5–9	5–50
M	Slabbing and rock burst after a few minutes in massive rock	3–2	0.65–1.0	9–15	50–200
N	Heavy rock burst (strain-burst) and immediate dynamic deformations in massive rock	$<$2	$>$1	15–20	200–400
	(c) Squeezing rock; plastic flow of incompetent rock under the influence of high rock pressures				
O	Mild squeezing rock pressure			1–5	5–10
P	Heavy squeezing rock pressure			$>$5	10–20
	(d) Swelling rock; chemical swelling activity depending on presence of water				
Q	Mild swelling rock pressure			5–10	
R	Heavy swelling rock pressure			10–15	

Reduce these SRF values by 25–50% if the relevant shear zones only influence but do not intersect the excavation. This will also be relevant for characterization.
For strongly anisotropic virgin stress field (if measured): when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce q_c to 0.75 q_c; when $\sigma_1/\sigma_3 > 10$, reduce q_c to 0.50 q_c (where q_c is unconfined compressive strength), σ_1 and σ_3 are major and minor principal stresses, and σ_θ is the maximum tangential stress (estimated from elastic theory).
Few case records available where depth of crown below surface is less than span width; suggest SRF increase from 2.5 to 5 for such cases (see H).
Cases L, M, and N are usually most relevant for support design of deep tunnel excavation in hard massive rock masses, with RQD/J_n ratios from about 50–200.
For general characterization of rock masses distant from excavation influences, the use of SRF = 5, 2.5, 1.0, and 0.5 is recommended as depth increases from, say, 0–5, 5–25, 25–250, >250 m. This will help to adjust Q for some of the effective stress effects, in combination with appropriate characterization values of J_w. Correlations with depth-dependent static modulus of deformation and seismic velocity will then follow the practice used when these were developed.
Cases of squeezing rock may occur for depth $H > 350Q^{1/3}$ (Singh & Goel, 2006). Rock mass compressive strength can be estimated from $q_{cmass} \approx 7\gamma (Q)^{1/3}$ (MPa); γ is the rock density in t/m^3, and q_{cmass} = rock mass compressive strength.

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Table 7 Classification of rock mass based on Q-values

Q	Group	Classification
0.001–0.01		Exceptionally poor
0.01–0.1	3	Extremely poor
0.1–1		Very poor
1–4	2	Poor
4–10		Fair
10–40		Good
40–100	1	Very good
100–400		Extremely good
400–1000		Exceptionally good

Table 8 Joint roughness factor (j_R) for use with the RMi system

Small-scale smoothness* of joint surface	**Large-scale waviness of joint plane**				
(The ratings in bold are similar to Jr in the Q-system)	***Planar***	***Slightly undulating***	***Undulating***	***Strongly undulating***	***Stepped or interlocking***
Very rough	2	3	4	6	6
Rough	**1.5**	2	**3**	4.5	6
Smooth	**1**	1.5	**2**	3	4
Polished or slickensided*	**0.5**	1	**1.5**	2	3
For filled joints: jR = 1; for irregular joints a rating of jR = 6 is suggested					

**For slickensided surfaces the ratings given cover possible movement along the lineation. (For movements across lineation, a rough or very rough rating should be applied for the surface.)*

Table 9 Joint length and continuity factor (j_L) for use with the RMi system

			jL	
Joint length (m)	**Term**	**Type**	***Continuous joints***	***Discontinuous joints****
<0.5	Very short	Bedding/foliation parting	3	6
0.1–1.0	Short/small	Joint	2	4
1–10	Medium	Joint	1	2
10–30	Long/large	Joint	0.75	1.5
>30	Very long/ large	Filled joint, seam or shear*	0.5	1

**Often a singularity (special feature), and should in these cases be treated separately.*
***Discontinuous joints end in massive rock mass.*

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Table 10 Joint alteration factor (j_A) for use with the RMi system

Term	Description	jA
	A. Contact between rock wall surfaces	
Clean joints		
Healed or welded joints	Softening, impermeable filling (quartz, epidote, etc.)	0.75
Fresh joint walls	No coating or filling on joint surface, except from staining (rust)	1
Alteration of joint wall		
i. 1 grade more altered	The joint surface exhibits one class higher alteration than the rock	2
ii. 2 grade more altered	The joint surface shows two classes higher alteration than the rock	4
Coating or thin filling		
Sand, silt, calcite, etc.	Coating of friction materials without clay	3
Clay, chlorite, talc, etc.	Coating of softening and cohesive minerals	4

B. Filled joints with partial or no contact between the joint wall surfaces

Type of filling material	Description	Partial wall contact (thin filling <5 mm*)	No wall contact (thick filling or gouge)
Sand, silt, calcite, etc. (non-softening)	Filling of friction material without clay	4	8
Compacted clay materials	"Hard" filling of softening and cohesive materials	6	6–10
Soft clay materials	Medium to low over-consolidation of filling	8	12
Swelling clay materials	Filling material exhibits clear swelling properties	8–12	13–20

**Based on joint thickness division in the RMR system (Bieniawski, 1973).*

Table 11 Classification of rock based on RMi values

Term		
For RMi	*Related to rock mass strength*	RMi value
Extremely low	Extremely weak	<0.001
Very low	Very weak	0.001–0.01
Low	Weak	0.01–0.1
Moderate	Medium	0.1–1.0
High	Strong	1.0–10.0
Very high	Very strong	10–100
Extremely high	Extremely strong	>100

Table 12 Principal stresses for selected pre-shaft stress ratios (*M* values) (Compression is considered positive)

Case M	*Principal stresses*	*Physical description*
-1	$S_3 = -S_1$	Pure shear
0	$S_3 = 0$	Uniaxial compression
$+1$	$S_3 = +S_1$	Hydrostatic compression
$-\infty$	$S_1 = 0$	Uniaxial tension

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Figure 1 Stress concentrations at the wall of a circular shaft at 5° intervals for a range of principal stress ratios (*M* values)

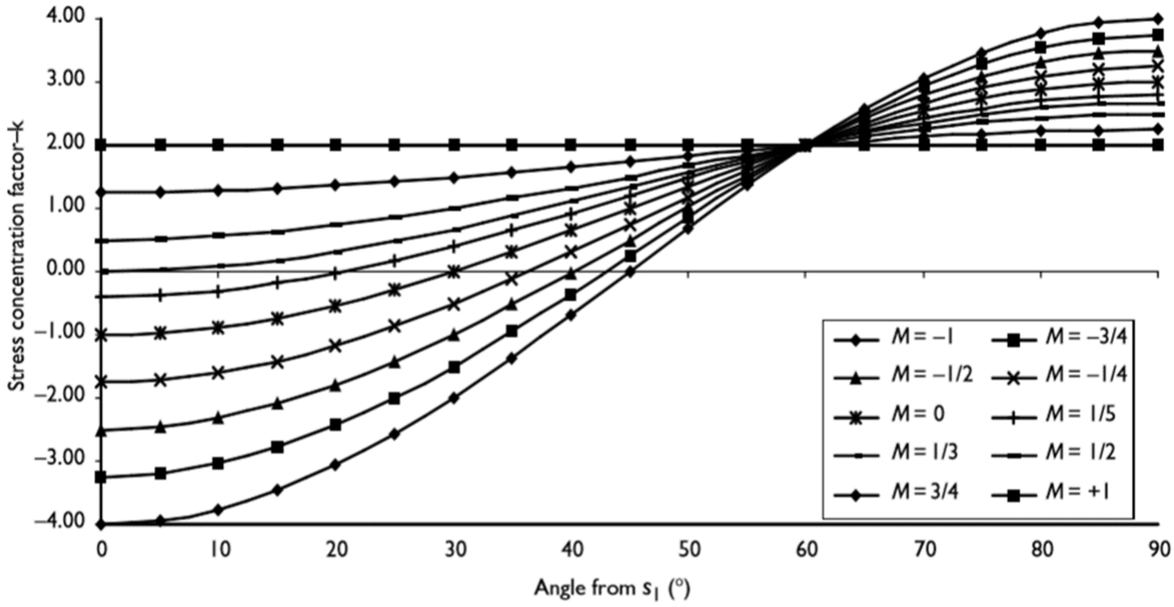

Table 13 Stress concentration factors about a rectangular section

k / k'	M = 0		1/5		1/3		1/2		3/4		1
	K_{max}	K_{min}	K_{max}	K_{min}	K_{max}	K_{min}	K_{max}	K_{min}	K_{max}	K_{min}	K_{max} / K_{min}
1/8	3.70	−0.84	5.30	−0.18	6.62	0.20	8.28	0.64	11.1	0.42	14.04
8	11.8	−1.01	12.08	−0.77	12.4	−0.60	12.8	−0.40	13.4	−0.10	0.18
1/4	2.96	−0.81	3.72	−0.29	4.36	0.03	5.28	0.42	6.80	0.56	8.43
4	6.96	−0.96	7.21	−0.71	7.37	−0.53	7.58	−0.32	7.99	0.00	0.32
1/2	3.28	−0.82	3.70	−0.41	4.05	−0.14	4.57	0.19	5.48	0.69	6.46
2	4.72	−0.92	4.97	−0.64	5.15	−0.44	5.44	−0.20	5.92	0.16	0.52
1	3.58	−0.85	3.83	−0.52	4.06	−0.29	4.39	−0.02	4.97	0.40	5.65
											0.80

Notes
M = principal stress ratio (S_3/S_1).
k = aspect ratio (b/a), S_1 is applied parallel to the a-axis (long axis of the section).
$k' = 1/k$ corresponding to application of S_1 parallel to the short axis of the section.
$K(\max)$ = maximum stress concentration factor.
$K(\min)$ = minimum stress concentration factor.

Table 14 Properties for guideline computations involving foundations on jointed rock

PROPERTY MATERIAL	E (10^6psi/GPa)	ν (–)	C_o (psi/MPa)	T_o (psi/MPa)
ROCK	4.8/33.1	0.20	9,600/66.2	800/5.52
JOINTS	0.048/0.33	0.20	480/3.3	4.8/0.033
FOOTING	4.8/33.1	0.20	4,800/33.1	400/2.76

E = Young's modulus, v = Poisson's ratio, C_o = unconfined compressive strength, T_o = tensile strength.

Table 15 Square foundation stiffness for various combinations of joint spacing and strata dip. (B = foundation width, S = joint spacing)

SPACING *DIP (deg)*	*WIDE* *B/S=2*	*MEDIUM* *B/S=5*	*CLOSE* *B/S=10*	*VERY CLOSE* *B/S=25*
0/90	6.48/1.76	4.37/1.19	2.98/0.81	1.54/0.42
15/75	5.68/1.54	3.50/0.95	2.05/0.56	1.06/0.29
30/60	6.04/1.64	3.76/1.02	2.00/0.54	1.06/0.29
45/45	6.37/1.73	3.56/0.97	1.78/0.48	0.94/0.25

Notes
* Units of stiffness are kpsi/inch / MPa/mm. Conversion: (MPa/mm) = 0.272(kpsi/inch).

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▸ SOLUTIONS

P.1 ■ Solution

The RQD is given by

$$RQD = \frac{\Sigma \text{Length of core pieces} > 10 \text{ cm}}{\text{Total length of core run}}$$

Accordingly,

$$RQD = \frac{24+40+41+44}{200} = \boxed{74.5\%}$$

This RQD indicates a rock with fair to good quality.

▸ The correct answer is **C**.

P.2 ■ Solution

1. True. Palmström and other investigators verified that, indeed, there is a poor correlation between the RQD and other approaches to block size measurement. As a result, the use of RQD as a standalone parameter in rock engineering calculations may lead to inaccuracy or errors. Bienlawski has observed that *while the RQD is a practical parameter for core logging, it is not sufficient on its own to provide an accurate description of a rock mass*. This parameter should, therefore, be used with great care.

2. False. The volumetric joint count for the rock mass in question is

$$J_V = \frac{1}{(10/12)} + \frac{1}{(6/9)} + \frac{1}{(10/8)} + \frac{1}{(5/7)} + \frac{1}{(4/12)} = 7.9 \text{ joints/m}^3$$

The corresponding *RQD* follows as

$$RQD = 110 - 2.5 \times 7.9 = 90.25\%$$

3. True. The coefficients to use for a porous limestone such as the one in question are n_i = 3.43%, *a* = 0.4×10^{-4} and *b* = –144×10^{-4}. Substituting *T* = 400°C in the correlation brings to

$$n = 3.43 + 0.4 \times 10^{-4} \times 400^2 - 144 \times 10^{-4} \times 400 = 4.07\%$$

4. False. We are looking for the depth at which the density becomes 2.0 g/cm^3. With reference to the correlation received, we write

$$2.72 - 1.244 \exp(-0.846z) = 2.0$$

Solving this equation gives $z_{\min}$ = 0.646 km; that is, the rock reaches the prescribed density at a depth of about 650 m.

5. False. In general, the opposite is true, i.e., the UCS tends to decrease with increasing specimen volume. For very small specimen sizes, the opposite effect is observed, that is, there may be a decrease with decreasing specimen volume. This latter effect, coupled with the ISRM's requirement that the specimen diameter should be at least 10 times the size of the largest grain, provides a reason for using specimen diameters of approximately 50 mm in laboratory compression tests.

6. True. The coefficients for use with the type of rock in question are *a* = 14.68, *b* = 0.8193, and *c* = 24.0, which leads to the relation

$$\sigma_c = 14.68e^{-0.8193w} + 24.0$$

For a water content of 2%, we get

$$(\sigma_c)_1 = 14.68 \times \exp(-0.8193 \times 2) + 24.0 = 26.85 \text{ MPa}$$

while, for *w* = 5%,

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$$\left(\sigma_c\right)_2 = 14.68 \times \exp\left(-0.8193 \times 7\right) + 24.0 = 24.05 \text{ MPa}$$

which represents a decrease of 10.43%.

7. False. Factor f_c = 120/100 = 1.20. Substituting this and other pertaining variables gives

$$\sigma_t = -0.029 \times 2.70 \times 1.0 \times 15^{0.3} = -0.176 \text{ MPa} = -176 \text{ kPa}$$

8. True. This is observed in Section 7.8 of Zhang's *Engineering Properties of Rocks*. In addition, the minimum strength is obtained when the angle β between the major principal stress and the stratification plane is at 30-60 degrees. The degree of strength anisotropy is commonly quantified by the strength anisotropy ratio, R_c, defined as

$$R_c = \frac{\sigma_{c,\max}}{\sigma_{c,\min}}$$

where $\sigma_{c,\max}$ and $\sigma_{c,\min}$ are the maximum and minimum compressive strengths at a given confining pressure, respectively. A rock is said to exhibit *high* anisotropy if $4.0 < R_c \leq 6.0$, and *very high* anisotropy if $R_c > 6.0$.

9. False. The 1976 RMR equals the 1989 RMR *minus*, not plus, 5. The RMR underwent several modifications since it was first proposed in 1973. For example, in 1974 the number of classification parameters was reduced from 8 to 6; in 1975, ratings were adjusted and recommended support requirements were reduced; in 1976, class boundaries were reduced to even multiples of 20; in 1979, the ISRM rock mass description proposed in the previous year was adopted.

10. True. Joint orientation, for one, was not found to be an important, general parameter. If joint orientations had been included, the *Q* classification system would have been less general, and its essential simplicity lost. Other features not included in the Q-system are direct measurements of rock strength (although a 2002 review of the system does take the UCS into account), joint size, joint persistence, and joint aperture. The latter is used in the RMR, GSI and RMi systems.

P.3 ■ Solution

For a point-load strength index of 4 MPa, Table 1-A1 gives a rating of 12. For a RQD between 50 and 75%, Table 1-A2 gives a rating of 13. For a discontinuity spacing of 450 mm, Table 1-A3 gives a rating of 10. For slightly rough joints with the specified separation and highly weathered walls, Table 1-A4 gives a rating of 20. For the specified groundwater conditions, Table 1-A5 gives a rating of 10. Lastly, Table 1-F gives a description of 'favorable' if the tunnel is to be driven against the dip of a set of joints dipping at 30°. Referring to Table 1-B with this classification, we read a rating adjustment of –2. Gathering this information, the RMR is found as

$$RMR = 12 + 13 + 10 + 20 + 10 - 2 = \boxed{63}$$

According to Table 1-C, the mass in question fits into the 'good rock' category.

► The correct answer is **C**.

P.4 ■ Solution

For a UCS of 80 MPa, Table 1-A1 gives a rating of 12. For a RQD of 80%, Table 1-A2 gives a rating of 17. For a discontinuity spacing of 80 mm, Table 1-A3 gives a rating of 8. Since detailed information on the joint set is available, we may resort to the detailed classification in Table 1-E instead of using the generalizations in row 1-A4. Accordingly, a discontinuity persistence of 4 m contributes +2 to the rating; an aperture < 0.1 mm adds +5; a slightly rough structure adds +3; the absence of infilling adds +6; a moderately weathered structure adds +3; then, the overall rating increase due to discontinuity conditions

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is 2 + 5 + 3 + 6 + 3 = 19. For dripping groundwater conditions, Table 1-A5 gives a rating of +4. For 'fair' discontinuity conditions, Table 1-B gives a penalty rating of –7. Gleaning this information, the RMR is found as

$$RMR = 12 + 17 + 8 + 19 + 4 - 7 = \boxed{53}$$

As per Table 1-C, the mass considered belongs to the 'fair rock' category.

► The correct answer is **B**.

P.5 ■ Solution

For one set of joints governing stability, Table 2 gives a joint set number J_n = 2.0. For rough, irregular and undulating joints, Table 3 gives a joint roughness number J_r = 3.0. For unaltered joint walls with surface staining only, Table 4 gives a joint alteration number J_a = 1.0. For medium inflow with occasional outwash of joint fillings, Table 5 gives a joint water reduction number J_w = 0.66. The overburden stress will not surpass 10 MPa, which is to say that the major principal stress will be $\sigma_1 = K \times 10 = 4/3 \times 10 = 13.3$ MPa at most. This gives a ratio σ_c/σ_1 = 160/13.3 = 12.0. Referring to Table 6 with this quantity, we see that, for a ratio σ_c/σ_1 ranging from 10.0 to 200 and favorable stress conditions, the SRF should be taken as 1.0. We are now in position to evaluate the *Q*-value for this rock mass,

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} = \frac{80}{2.0} \times \frac{3.0}{1.0} \times \frac{0.66}{1.0} = \boxed{79.2}$$

According to Table 7, the mass in question is just above the threshold to be classified as 'very good' rock.

► The correct answer is **C**.

P.6 ■ Solution

For one set of joints governing stability plus random, Table 2 gives a joint set number J_n = 3.0. For smooth and planar joints, Table 3 gives a joint roughness number J_r = 1.0. For rock wall contact before 10 cm shear, strongly overconsolidated, non-softening clay mineral, Table 4 gives a joint alteration number J_a = 6.0. For large inflow with considerable outwash of joint fillings, Table 5 gives a joint water reduction J_w = 0.33. Since the excavation intersects multiple weakness zones containing clay and disintegrated rock, Table 6 indicates a SRF of 10.0. We can then establish the *Q*-value for this rock mass,

$$Q = \frac{RQD}{J_n} \times \frac{J_r}{J_a} \times \frac{J_w}{SRF} = \frac{60}{3.0} \times \frac{1.0}{6.0} \times \frac{0.33}{10.0} = \boxed{0.11}$$

According to Table 7, the mass in question is just above the threshold to be classified as 'very poor' rock.

► The correct answer is **A**.

P.7 ■ Solution

For smooth and strongly undulating joints, Table 8 gives j_R = 3. For a single medium continuous joint, Table 9 gives j_L = 1. For the joints coated with sand and silt, Table 10 gives j_A = 3. The joint condition factor is, accordingly,

$$j_C = \frac{j_L j_R}{j_A} = \frac{1 \times 3}{3} = 1.0$$

The value of *D* is

$$D = 0.37 \times j_C^{-0.2} = 0.37 \times 1.0^{-0.2} = 0.37$$

and the jointing parameter follows as

$$J_P = 0.2 \times j_C^{0.5} \times V_b^D = 0.2 \times 1.0^{0.5} \times 0.002^{0.37} = 0.0201$$

The RMi is then

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$$RMi = \sigma_c \times J_p = 25 \times 0.0201 = \boxed{0.503}$$

From Table 11, we see that the rock in question has medium mechanical strength.

▶ The correct answer is **A**.

P.8 ■ Solution

For very rough and strongly undulating joints, Table 8 gives j_R = 6. For very short continuous joints, Table 9 gives j_L = 3. For fresh joint walls, Table 11 gives j_A = 1. The joint condition factor is calculated as

$$j_C = \frac{j_L j_R}{j_A} = \frac{3 \times 6}{1} = 18.0$$

The value of *D* is

$$D = 0.37 \times j_C^{-0.2} = 0.37 \times 18.0^{-0.2} = 0.208$$

and the jointing parameter is computed as

$$J_P = 0.2 \times j_C^{0.5} \times V_b^D = 0.2 \times 18.0^{0.5} \times 0.005^{0.208} = 0.282$$

The RMi is determined to be

$$RMi = \sigma_c \times J_P = 50 \times 0.282 = \boxed{14.1}$$

Reading Table 11, we verify that the rock in question is very strong.

▶ The correct answer is **D**.

P.9 ■ Solution

The uncorrected point load index is

$$I_s = \frac{P}{D_e^2} = \frac{14.7 \times 10^3}{0.07^2} = 3 \text{ MPa}$$

The corrected index follows as

$$I_{s(50)} = I_s \times \left(\frac{D_e}{50}\right)^{0.45} = 3 \times \left(\frac{70}{50}\right)^{0.45} = 3.49 \text{ MPa}$$

With reference to the table, the correlation to use is $\sigma_c = 21.9 I_{s(50)}$. Accordingly,

$$\sigma_c = 21.9 I_{s(50)} = 21.9 \times 3.49 = \boxed{76.4 \text{ MPa}}$$

▶ The correct answer is **C**.

P.10 ■ Solution

1. False. The angle of internal friction for intact rock is

$$\sin\phi = \frac{\sigma_c - \sigma_t}{\sigma_c + \sigma_t} = \frac{105 - 12}{105 + 12} = 0.795$$

$$\therefore \phi = 52.7^\circ$$

The cohesion of the intact rock is

$$c = \frac{\sqrt{\sigma_c \sigma_t}}{2} = \frac{\sqrt{105 \times 12}}{2} = 17.7 \text{ MPa}$$

Turning to the rock mass, the angle of internal friction is determined as

$$\tan\phi_{rm} = (1 - p) \times \tan\phi + p \times \tan\phi_j = (1 - 0.75) \times \tan 52.7^\circ + 0.75 \times \tan 26^\circ = 0.694$$

$$\therefore \phi_{rm} = 34.8^\circ$$

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2. True. The cohesion of the rock mass is calculated as

$$c_{rm} = (1-p)\times c + p\times c_j = (1-0.75)\times 17.7 + 0.75\times 0.09 = 4.49 \text{ MPa}$$

3. True. The unconfined compressive strength of the rock mass is computed as

$$\sigma_{c,rm} = \frac{2c\cos\phi_{rm}}{1-\sin\phi_{rm}} = \frac{2\times 17.7\times\cos 34.8^{\circ}}{1-\sin 34.8^{\circ}} = 67.7 \text{ MPa}$$

4. False. The tensile strength of the rock mass is evaluated as

$$\sigma_{t,rm} = \frac{2c\cos\phi_{rm}}{1+\sin\phi_{rm}} = \frac{2\times 17.7\times\cos 34.8^{\circ}}{1+\sin 34.8^{\circ}} = 18.5 \text{ MPa}$$

As a final check on the validity of these results, note that the ratio of the compressive strength to the tensile strength of the rock mass should obey the relation

$$\frac{\sigma_{c,rm}}{\sigma_{t,rm}} = \frac{1+\sin\phi_{rm}}{1-\sin\phi_{rm}}$$

Evaluating the left-hand side gives

$$\frac{\sigma_{c,rm}}{\sigma_{t,rm}} = \frac{67.7}{18.5} = 3.66$$

As for the right-hand side,

$$\frac{1+\sin 34.8^{\circ}}{1-\sin 34.8^{\circ}} = 3.66$$

The equality is correct, and our results have been corroborated.

P.11 ■ Solution

The stresses and strains are related by the stress-strain relationships

$$\varepsilon_x = \frac{1}{E}\left(\sigma_x - v\sigma_y\right)$$

$$\varepsilon_y = \frac{1}{E}\left(\sigma_y - v\sigma_x\right)$$

Inserting our data yields

$$-440\times 10^{-6} = \frac{1}{E}\left[-24\times 10^{6} - v\times\left(-12\times 10^{6}\right)\right]$$

$$\therefore -440\times 10^{-6}E = -24\times 10^{6} + 12\times 10^{6}v \text{ (I)}$$

$$-80\times 10^{-6} = \frac{1}{E}\left[-12\times 10^{6} - v\times\left(-24\times 10^{6}\right)\right]$$

$$\therefore -80\times 10^{-6}E = -12\times 10^{6} + 24\times 10^{6}v \text{ (II)}$$

Solving equations (I) and (II) simultaneously gives v = 0.35 and E = 45 GPa. With reference to the table we were given, we conclude that the block is made of marble.

► The correct answer is **B**.

P.12 ■ Solution

Reading the table, we take a Poisson's ratio v = 0.20. For a density of 2.75 g/cm^3, Young's modulus is estimated as

$$E = 64.64\times 2.75 - 115.4 = 62.4 \text{ GPa}$$

The stress in the x-direction can be determined with the stress-strain relation for biaxial stress,

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$$\sigma_x = \frac{E}{1-\nu^2}\left(\varepsilon_x + \nu\varepsilon_y\right) = \frac{62.4\times10^9}{1-0.20^2}\times\left[-2\times10^{-4} + 0.20\times\left(5\times10^{-5}\right)\right] = -12.4\text{ MPa}$$

The negative sign of course denotes compression. As for the *y*-direction, we have

$$\sigma_y = \frac{E}{1-\nu^2}\left(\varepsilon_y + \nu\varepsilon_x\right) = \frac{62.4\times10^9}{1-0.20^2}\times\left[5\times10^{-5} + 0.20\times\left(-2\times10^{-4}\right)\right] = 0.650\text{ MPa}$$

The strain in the *z*-direction follows as

$$\varepsilon_z = -\frac{\nu}{E}\left(\sigma_x + \sigma_y\right) = -\frac{0.20}{62.4\times10^9}\times\left(-12.4\times10^6 + 0.65\times10^6\right) = 3.77\times10^{-5}$$

Finally, the change in the dimension in the *z*-direction is found as

$$\Delta t = \varepsilon_z \times t = \left(3.77\times10^{-5}\right)\times5000 = \boxed{+0.189\text{ mm}}$$

The positive sign of course indicates an increase in the dimension being considered.

► The correct answer is **C**.

P.13 ■ Solution

Part A: The uniaxial compressive strength of the rock mass can be estimated as

$$\sigma_{\text{cm}} = \sigma_{\text{ci}} \times s^a$$

Constant s is computed as

$$s = \exp\left(\frac{GSI - 100}{9-3D}\right) = \exp\left(\frac{75-100}{9-3\times0}\right) = 0.0622$$

Constant *a*, in turn, is determined as

$$a = \frac{1}{2} + \frac{1}{6}\times\left(e^{-GSI/15} - e^{-20/3}\right) = 0.5 + 0.167\times\left(e^{-75/15} - e^{-20/3}\right) = 0.501$$

Lastly, the uniaxial compressive strength of the rock mass is estimated as

$$\sigma_{\text{cm}} = \sigma_{\text{ci}} s^a = 51\times0.0622^{0.501} = \boxed{12.7\text{ MPa}}$$

► The correct answer is **A**.

Part B: Substituting the pertaining data gives

$$\sigma_{\text{cm}} = 51\times0.0034\times16.3^{0.8}\times\left[1.029 + 0.025\times\exp\left(-0.1\times16.3\right)\right]^{75} = \boxed{19.7\text{ MPa}}$$

► The correct answer is **B**.

Part C: The tensile strength of the rock mass can be estimated with the relation

$$\sigma_{\text{tm}} = -\sigma_{\text{cm}}\frac{\sqrt{m_m^2 + 4s} - m_m}{2}$$

where m_m is the *m*-parameter for the rock mass, which is given by

$$m_m = m_i \exp\left(\frac{GSI - 100}{28 - 14D}\right) = 28\times\exp\left(\frac{75-100}{28-14\times0}\right) = 11.5$$

Accordingly,

$$\sigma_{\text{tm}} = -12.7\times\left(\frac{\sqrt{11.5^2 + 4\times0.0622} - 11.5}{2}\right) = -0.0687$$

$$\therefore \boxed{\sigma_{\text{tm}} = 68.7\text{ kPa}}$$

► The correct answer is **D**.

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Part D: The first step is to appeal to the Hoek-Brown failure criterion curve and simulate some data points. The curve in question is of course

$$\sigma'_{1f} = \sigma'_{3f} + \sigma'_{ci}\left(m_m \frac{\sigma'_{3f}}{\sigma'_{ci}} + s\right)^a$$

which in the present case becomes

$$\sigma'_{1f} = \sigma'_{3f} + 51\left(11.5 \times \frac{\sigma'_{3f}}{51} + 0.0622\right)^{0.501}$$

$$\therefore \sigma'_{1f} = \sigma'_{3f} + 51\left(0.225\sigma'_{3f} + 0.0622\right)^{0.501}$$

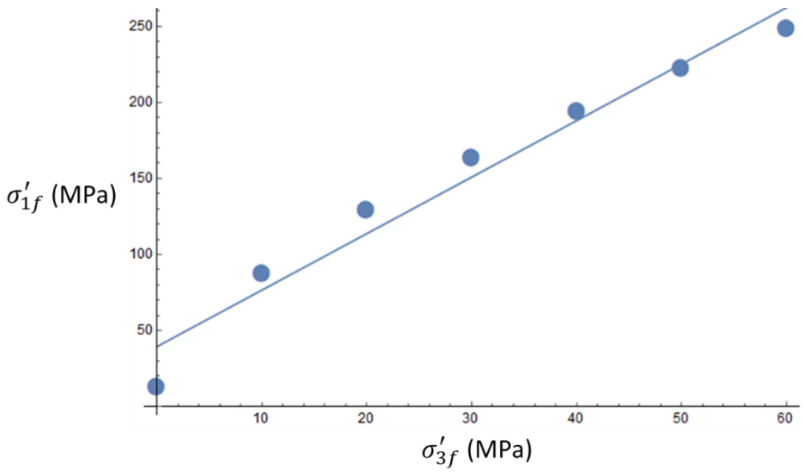
Using this equation, we can tabulate some values of the major principal stress σ'_{1f} for given values of the confining pressure σ'_{3f}, as shown.

σ'_{3f} (MPa)	σ'_{1f} (MPa)
0	12.7
10	87.6
20	129
30	163
40	194
50	222
60	248

The next step is to fit a line that passes through the data points generated above. Using the *LinearModelFit* function in Mathematica, we get a line of the form

$$\sigma'_{1f} = 3.71\sigma'_{3f} + 39.5$$

The coefficient of determination is 0.963 and indicates a good fit to the data. The line and the data points are plotted below.



Now, recall that, in the Mohr-Coulomb failure criterion, the principal stresses are related by

$$\sigma'_{1f} = \sigma'_{3f}\tan^2\left(45^\circ + \frac{\phi'}{2}\right) + 2c'\tan\left(45^\circ + \frac{\phi'}{2}\right)$$

Comparing this equation to that of the line obtained above, we see that

$$\tan^2\left(45^\circ + \frac{\phi'}{2}\right) = 3.71 \rightarrow \boxed{\phi' = 35.1^\circ}$$

and

$$2c'\tan\left(45^\circ + \frac{\phi'}{2}\right) = 39.5$$

$$\therefore 2c'\tan\left(45^\circ + \frac{35.1^\circ}{2}\right) = 39.5$$

$$\therefore \boxed{c' = 10.3 \text{ MPa}}$$

► The correct answer is **D**.

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P.14 ■ Solution

Part A: The overburden stress due to the excavation is $\sigma_1 = \gamma H$ = 23 × 915 = 21.0 MPa. The principal stress in question is also the vertical stress, i.e. $\sigma_1 = \sigma_V$ = 21.0 MPa. The factor of safety with respect to compression in this direction follows as

$$\left(FS_c\right)_V = \frac{\sigma_c}{\sigma_V} = \frac{152}{21.0} = \boxed{7.24}$$

Since the shaft is under hydrostatic compression relatively to the horizontal, we have $\sigma_h = \sigma_H$ and $M = \sigma_3/\sigma_1$ = 1.0 (Table 12). Referring to Figure 1 with this ratio, we read a stress concentration factor K_c = 2.0. The horizontal stresses are related to the vertical stress by the expression

$$\sigma_h = \sigma_H = \frac{\nu}{1-\nu} \times \sigma_V$$

where ν is Poisson's ratio, which, from strength of materials, can be estimated as

$$\nu = \frac{E}{2G} - 1 = \frac{34.5}{2 \times 13.8} - 1 = 0.25$$

Therefore,

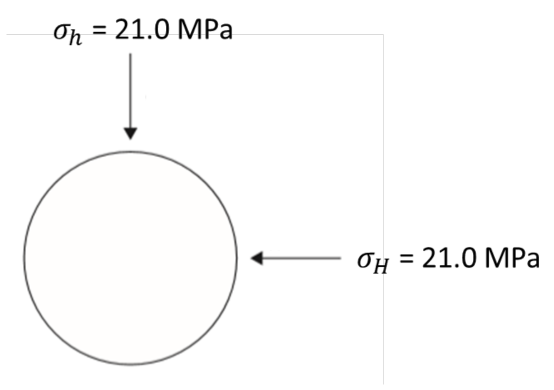
$$\sigma_h = \sigma_H = \frac{\nu}{1-\nu} \times \sigma_V = \frac{0.25}{1-0.25} \times 21.0 = 7.0 \text{ MPa}$$

The factor of safety against compression in the horizontal direction is determined to be

$$\left(FS_c\right)_H = \frac{\sigma_c}{K_c \times \sigma_H} = \frac{152}{2.0 \times 7.0} = \boxed{10.9}$$

► The correct answer is **D**.

Part B: The horizontal stress state is illustrated below.



We now have $\sigma_H = \sigma_V$ = 21.0 MPa. The updated factor of safety follows as

$$\left(FS_c\right)_H = \frac{\sigma_c}{K_c \times \sigma_H} = \frac{152}{2.0 \times 21.0} = \boxed{3.62}$$

The horizontal stress has increased relatively to the previous problem, and the factor of safety has decreased accordingly.

► The correct answer is **B**.

P.15 ■ Solution

The maximum vertical stress is

$$\sigma_V = 25.3 \times 533 = 13.5 \text{ MPa}$$

The eastward horizontal stress is, at most,

$$\sigma_E = 2414 + 4.5 \times 533 = 4.81 \text{ MPa}$$

The northward horizontal stress is, at most,

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$$\sigma_N = 2897 + 7.97 \times 533 = 7.11 \text{ MPa}$$

The ratio of principal stresses in the horizontal is $M = \sigma_3/\sigma_1 = 4.81/7.11 = 0.677$. From Table 13, we know that the maximum stress concentration factor is 4.57 for $M = 0.5$ and 5.48 for $M = 0.75$. Interpolating between these two quantities gives $K_{max} = 5.21$. We proceed to determine the factor of safety against compression in the horizontal direction,

$$(FS_c)_H = \frac{\sigma_c}{K_{\max} \times \sigma_1} = \frac{164}{5.21 \times 7.11} = \boxed{4.43}$$

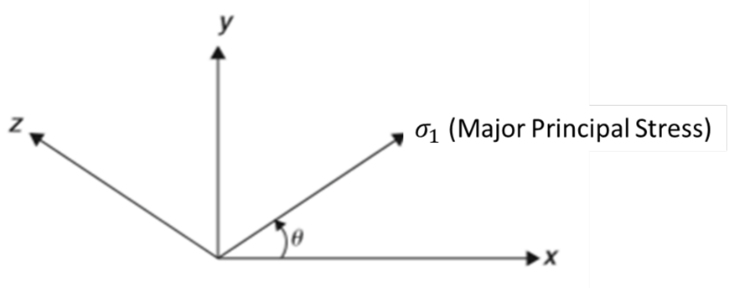
The factor of safety in the vertical direction is determined next,

$$(FS_c)_V = \frac{164}{13.5} = \boxed{12.1}$$

► The correct answer is **C**.

P.16 ■ Solution

Part A: As a rule, the most favorable orientation for the shaft is to have it be with the long axis parallel to the major compressive stress, as illustrated below.



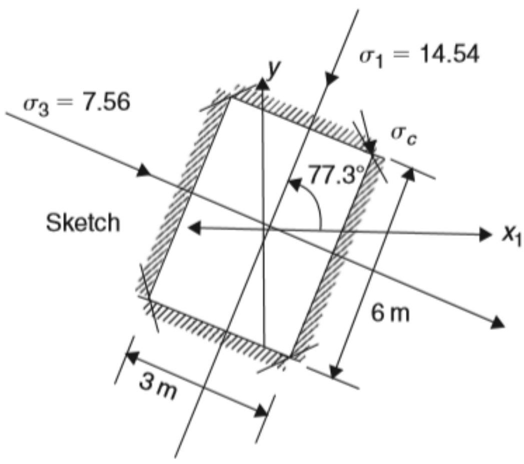
Angle θ can be obtained with the stress transformation

$$\tan 2\theta = \frac{\tau_{xy}}{\frac{1}{2}(\sigma_{xx} - \sigma_{yy})} = \frac{1.5}{\frac{1}{2} \times (7.9 - 14.2)} = -0.476$$

$$\therefore 2\theta = -25.5^\circ, 154.5^\circ$$

$$\therefore \theta = -12.8^\circ, 77.3^\circ$$

We take the positive solution and conclude that the long axis should make an angle of 77.3° with the x-axis.



► The correct answer is **D**.

Part B: The major principal stress is given by the stress transformation

$$\sigma_1 = \frac{1}{2}(\sigma_{xx} + \sigma_{yy}) + \left[\left(\frac{\sigma_{xx} - \sigma_{yy}}{2}\right)^2 + \tau_{xy}^2\right]^{1/2} = \frac{1}{2} \times (7.9 + 14.2) + \left[\left(\frac{14.2 - 7.9}{2}\right)^2 + 1.5^2\right]^{1/2} = 14.5 \text{ MPa}$$

while the minor principal stress follows as

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$$\sigma_3 = \frac{1}{2}\left(\sigma_{xx}+\sigma_{yy}\right) - \left[\left(\frac{\sigma_{xx}-\sigma_{yy}}{2}\right)^2 + \tau_{xy}^2\right]^{1/2} = \frac{1}{2}\times(7.9+14.2) - \left[\left(\frac{14.2-7.9}{2}\right)^2 + 1.5^2\right]^{1/2} = 7.56\ \text{MPa}$$

The stress ratio *M* is, accordingly,

$$M = \frac{\sigma_3}{\sigma_1} = \frac{7.56}{14.5} = 0.521 \approx 0.5$$

Referring to Table 13 with this value of *M* and an aspect ratio *k* = 1/2, we read a stress concentration factor in compression $K_{max} = K_c = 4.57$. The factor of safety with respect to compression in the horizontal is then

$$\left(FS_c\right)_H = \frac{\sigma_c}{K_c \times \sigma_1} = \frac{103}{4.57\times 14.5} = \boxed{1.55}$$

▶ The correct answer is **A**.

P.17 ■ Solution

Part A: The ratio of footing width to joint spacing is B/S = 1.8/0.9 = 2.0. Referring to Table 15 with this value, we read a stiffness of 1.73 MPa/mm. Settlement is given by $\delta = P/K$, where *P* is the load and *K* is the stiffness. Substituting these data gives

$$\delta = \frac{P}{K} = \frac{10\times 10^6}{1.73\times 10^6} = \boxed{5.78\ \text{mm}}$$

▶ The correct answer is **A**.

Part B: The ratio B/S now becomes 1.8/0.18 = 10. Consulting Table 15 with this value, we read a stiffness of 0.48 MPa/mm. The ensuing settlement is then

$$\delta = \frac{P}{K} = \frac{10\times 10^6}{0.48\times 10^6} = \boxed{20.8\ \text{mm}}$$

Since the rock is much more jointed than that of the previous part, the strength of the material underlying the foundation is reduced, and the settlement is increased.

▶ The correct answer is **C**.

Part C: Since the properties are not the same as those in Table 14, we cannot readily apply $\delta = P/K$. Instead, we use propose a modified stiffness $K' = K(E'/E)$, where E' is a rectified Young's modulus given by

$$E' = \frac{(1-\nu)}{(1-2\nu)(1+\nu)}E = \frac{1-0.23}{(1-2\times 0.23)\times(1+0.23)}\times 42 = 48.7\ \text{GPa}$$

The settlement is, accordingly,

$$\delta = \frac{P}{K'} = \frac{P}{K\left(E'/E\right)} = \frac{10\times 10^6}{\left(1.73\times 10^6\right)\times\left(48.7/33.1\right)} = \boxed{3.93\ \text{mm}}$$

▶ The correct answer is **B**.

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ANSWER SUMMARY

Problem 1		C
Problem 2		T/F
Problem 3		C
Problem 4		B
Problem 5		C
Problem 6		A
Problem 7		A
Problem 8		D
Problem 9		C
Problem 10		T/F
Problem 11		B
Problem 12		C
Problem 13	13A	A
	13B	B
	13C	D
	13D	D
Problem 14	14A	D
	14B	B
Problem 15		C
Problem 16	16A	D
	16B	A
Problem 17	17A	A
	17B	C
	17C	B

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