FINAL

Devils Slide Tunnel

Geotechnical Design Report

Part 1: Rock Mass Parameters
Part 2: Rock Mass Behavior

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1 DESIGN APPROACH

Tunnel design is governed by the fact that "*Rock masses are so variable in nature that the chance for ever finding a common set of parameters and a common set of constitutive equations valid for all rock masses is quite remote."* (T.L. Brekke and T.R. Howard, 1972). Therefore it has to be taken into account that, prior to tunneling, any design represents a prediction which is either (a) verified on site in the event that all design assumptions are confirmed or (b) adjusted insitu to suit actual conditions. Figure 1 indicates the design approach which has been adopted. The first step of the design procedure is to establish geological data in those sections along the tunnel profile with consistent characteristics and then to summarize the geological series with similar mechanical properties. Further, the boundary conditions such as virgin stresses, size, shape and orientation of the opening have to be taken into account in order to establish a possible failure mechanism, thereby establishing the behavior of the opening. Different failure mechanisms require different support measures as well as models of analysis to design the support measures. In order to simplify procedures at the site, support categories are established which are applicable for the various types of behavior of the opening.

The subject report deals with the determination of rock mass types including characteristic rock mass parameters for the South Block, the Central Block and the North Block.

2 GEOLOGICAL INPUT DATA

2.1 Description of Geology

Two stratigraphic series are expected, namely the Montara granodiorite on the south and Paleocene age sedimentary rocks on the north. The contact between these two units has been reported to be depositional in areas and faulted in other areas, but was interpreted in GIR [8] as a fault contact. Montara granodiorite is commonly coarse-grained and may range in composition to quartz diorite and granite, containing pegmatitic veins and hornblende-rich intrusions. The sedimentary rocks include interbedded sandstones, siltstone and claystone and conglomerates with individual layers ranging from less than half a meter to perhaps several tens of meters thick. The sedimentary rocks have been extensively folded and faulted and near the ground surface these rocks are extensively weathered.

Geological/geotechnical input data have been derived from EMI documents [5] to [9]. Based on this data the Rock Mass Types for the South, the Central and the North Block have been defined.

Geotechnical Design

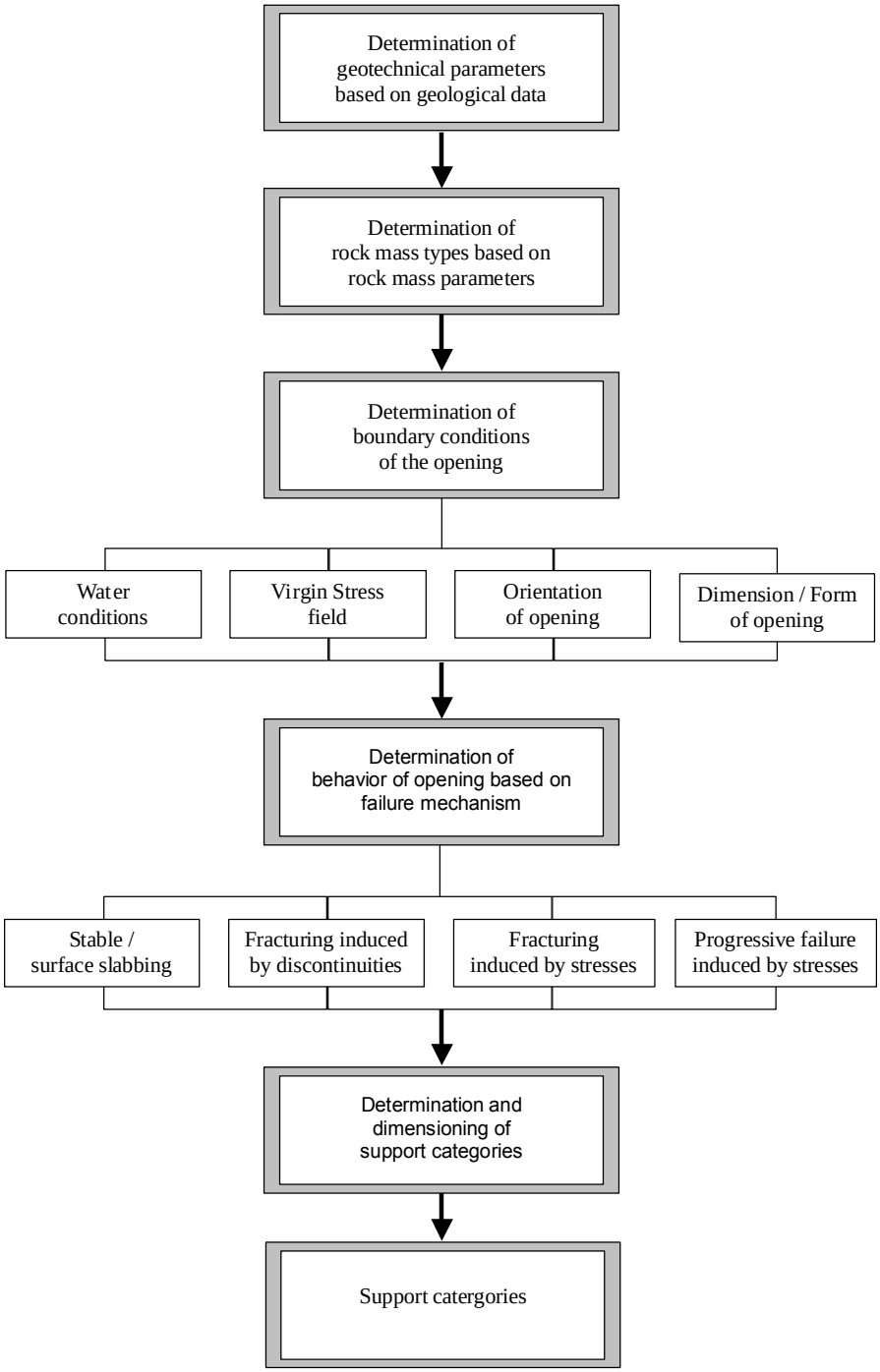

Figure 1: Flowchart for the Geotechnical Design

2.2 Lithological Units

Five lithological units are identified along the proposed alignment of the two Devil´s Slide Tunnel bores:

1. Granitic rock
2. Claystone
3. Siltstone
4. Sandstone
5. Conglomerate

The mineralogy, intact rock properties, and discontinuities identified for those units are summarized in EMI documents [5] to [9]. The rock mass description below is a brief compilation of this data.

2.2.1 Granitic Rock

Moderately to very intensely fractured granitic rock. Rock mass is fresh to strongly weathered with partly disintegration of the rock mass. Within the highly fractured rock sections, the appearance of clayey fault gouge is common. Spacing of discontinuities is varying between < 3 to 50 cm. Discontinuity surface properties are changing from planar to irregular, smooth to rough. The persistence of the discontinuities is in a range of <3 to >6 m. Where clayey infillings occur, groundwater can have a significant impact on the shear strength of discontinuities.

2.2.2 Claystone

Slightly to intensively fractured claystone. Rock mass is fresh to intensely weathered, in highly fractured and sheared sections, intensely weathered to disintegrated. Spacing of discontinuities is varying between <3 and 40 cm. Discontinuity surface properties are changing from planar to irregular, smooth to rough. The persistence is in the range of <3 to >6 m. Highly fractured and sheared sections show soil like rock mass characteristics. In those sections groundwater has a significant impact on rock mass behavior.

2.2.3 Siltstone

Slightly to intensively fractured siltstone. Rock mass is fresh to intensely weathered, in highly fractured and sheared sections, intensely weathered to disintegrated. Spacing of discontinuities is varying between <3 and 40 cm. Discontinuity surface properties are changing from planar to irregular,

smooth to rough. The persistence is in a range from <3 to >6 m. Highly fractured and sheared sections show soil like rock mass characteristics. In those sections groundwater has a significant impact on rock mass behavior.

2.2.4 Sandstone

Slightly to intensively fractured, fine to coarse grained sandstone. The degree of weathering is basically controlled by the intensity of fracturing and varies from fresh to intensely fractured. Spacing of discontinuities is varying between 3 and 40 cm. Discontinuity surface properties are changing from planar to irregular, slightly rough to rough. The persistence is in a range from <3 to 6 m.

2.2.5 Conglomerate

Slightly to intensively fractured, fine to coarse grained conglomerate with transition to coarse grained sandstones. The grain size of the matrix is varying from clay to sand, characterizing predominantly the unconfined compressive strength of the rock mass. The degree of weathering is controlled by the intensity of fracturing and varies from fresh to intensely fractured. Spacing of discontinuities is varying between 3 and 40 cm. Discontinuity surface properties are changing from planar to irregular, slightly rough to rough. The persistence is in a range from <3 to >6 m.

3 ROCK MASS TYPES

The rock mass types (RT) are defined using relevant geotechnical rock volumes including lithology, discontinuities and tectonic structures. The characteristics of the rock mass types are governed by:

- Lithology
- Properties and type of discontinuities
- Strength parameters of intact rock
- Conditions affecting parameters of intact rock and of rock mass

Eight characteristic geotechnical parameters are used to define ten rock mass types for the South, Central and the North Block of the Devil´s Slide Tunnel area which are summarized in the following table.

Rock Mass Characterization

	Rock Mass Types (RT)				
	RT-G 1	RT-G 2	RT-G 3	RT-SS 1	RT-SS 2
Lithology	Granitic Rock, slightly to moderately fractured	Granitic Rock, intensely to very intensely fractured	Granitic Rock, very intensely fractured - fault zone	Sandstone, slightly to moderately fractured	Sandstone, intensely to very intensely fractured
Weathering	fresh to slightly weathered	moderately to strongly weathered	moderately to strongly weathered, with disintegration of rock, occurrence of clayey gouge common	fresh to slightly weathered	moderately to intensely weathered
Spacing of Discontinuities [cm]	10 to 50	3 to 10	<3	10 to 40	3 to 10
Persistence [m]	<3 to >6	<3	<3	<3 to >6	<3
Opening/infilling	slightly to moderately open, predominantly clay, few calcite and sand	slightly to moderately open, predominantly clay, few calcite and sand	slightly to moderately open, predominantly clay, some sand	tight to slightly open, predominantly calcite, few clay	tight to slightly open, predominantly calcite, few clay
UCS [Mpa]	28.4	6.5	1.3	110	10
Density [KN/m³]	25.9	24	24	25.9	25.9
RQD [%]	50 to 90	15 to 50	0 to 15	50 to 80	15 to 50

	Rock Mass Types (RT)				
	RT-SH 1	RT-SH 2	RT-SH 3	RT-C 1	RT-C 2
Lithology	Interbedding of Siltstone and Claystone with Sandstone and Conglomerate interbeds possible	Interbedding of Siltstone and Claystone with Sandstone and Conglomerate interbeds possible	heavily sheared and disintegrated claystone and siltstone (fault gouge) with more competent blocks of sandstone and conglomerate (block in matrix structure)	Conglomerate, slightly to moderately fractured	Conglomerate, intensely to very intensely fractured
Weathering	fresh to slightly weathered	moderately to intensely weathered	intensely to disintegrated	fresh to slightly weathered	moderately to intensely weathered
Spacing of Discontinuities [m]	10 to 40	3 to 10	NA	10 to 50	3 to 10
Persistence [m]	<3 to >6	<3	NA	<3 to >6	<3
Opening/infilling	tight to slightly open, predominantly calcite, few clay	tight to slightly open, predominantly calcite, few clay	NA	tight to slightly open, predominantly calcite, few clay	tight to slightly open, predominantly calcite, few clay
UCS [Mpa]	29	10	0.01	49	10
Density [KN/m³]	26.2	26.2	23.9	26	26
RQD [%]	50 to 80	15 to 50	0 to 15	50 to 90	15 to 50

Table 1: Rock mass types

4 GEOTECHNICAL PARAMETERS

The mechanical properties of the various rock mass types are derived using several different methods in order to define an appropriate range of parameters:

- Hoek and Brown's mass law [10], [12] for deformation and strength parameters of the rock
- Serafim & Pereira (1983) and Boyd (1993), both in [15], for the determination of Young's Modulus
- K.W. John [11] for strength parameters

4.1 The Hoek-Brown Method (2002 Edition)

The geotechnical parameters are derived based on Hoek-Brown's mass law described in detail in [10]. The derivation is summarized below. The general form of Hoek-Brown's failure criterion is:

$$\sigma_1 = \sigma_3 + \sigma_{ci} \cdot \left(m_b \cdot \frac{\sigma_3}{\sigma_{ci}} + s \right)^a$$

σ_1, σ_3	major and minor principal effective stresses
m_b	Hoek-Brown constant for rock masses
s, a	parameters describing rock mass properties
σ_{ci}	uniaxial compressive strength of the intact rock

The Hoek-Brown criterion thus establishes a connection between the principal effective stresses. The rock mass parameters m_b, *a* and *s* can be derived by means of the following parameters:

- Hoek-Brown constant for intact rock m_i
- Geological Strength Index *GSI*

The Hoek-Brown constant m_i can be derived from triaxial tests. Relevant results of these tests are not given in the project documents; therefore this parameter is derived from relevant literature. The *GSI* is a parameter introduced by Hoek in 1994, providing a numerical rating of the rock masses based on the structure and surface of the rock mass. These values enable the parameters m_b, *s* and *a* to be defined as follows:

$$m_b = m_i \cdot \exp\left(\frac{GSI - 100}{28\text{-}14D} \right)$$

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

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

D (Disturbance Factor) is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses. Guidelines for the selections of *D* are given in [10].

The Hoek-Brown criterion serves to derive the Mohr-Coulomb parameters ϕ' and c'. Furthermore, the modulus of elasticity using the parameters σ_{ci} and *GSI* can also be determined.

$$\phi' = \sin^{-1}\left[\frac{6am_b(s + m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a) + 6am_b(s + m_b\sigma'_{3n})^{a-1}}\right]$$

$$c' = \frac{\sigma_{ci}\left[(1+2a)s + (1-a)m_b\sigma'_{3n}\right](s + m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1 + (6am_b(s + mb\sigma'_{3n})^{a-1})/((1+a)(2+a)}}$$

where $\sigma_{3n} = \sigma'_{3\max} / \sigma_{ci}$

The value of $\sigma'_{3\max}$ is the upper limit of confining stress over which the relationship between the Hoek-Brown and the Mohr-Coulomb criteria is considered.

$$Em(GPa) = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}}10^{((GSI-10)/40}$$ for $\sigma_{ci} \leq 100$

$$Em(GPa) = \left(1 - \frac{D}{2}\right)10^{((GSI-10)/40}$$ for $\sigma_{ci} > 100$

The Hoek-Brown Failure Criterion 2002 edition allows the consideration of overburden and rock mass density in the calculation of rock mass parameters. The variation of these parameters influences the resulting shear strength of the rock mass, but has no impact on Young's Modulus. The rock mass types to be encountered in the Central and North Block were calculated with an average overburden to be encountered in the Central Block. Since the overburden in the North Block is significantly lower this has to be considered in the interpretation of the results (a decrease of the overburden will increase the angle of internal friction and decrease the cohesion). Therefore, the upper bound of the

friction angles and the lower bound of the cohesion for the North Block are recommended for calculations.

The results of these calculations are listed in table 2.

Determination of Rock mass parameters according to Hoek-Brown

		RT-G 1	RT-G 2	RT-G 3	RT-SS 1	RT-SS 2
Input Parameters	GSI	45	30	25	45	30
	UCS - intact Rock [MPa]	28.4	6.5	1.3	110	10
	mi	20	17	15	17	15
	Overburden [m]	100	100	100	130	130
	Disturbance Faktor	0.5	0.5	0.5	0.5	0.5
	Specific Weight [KN/m³]	26	24	24	26	26
Calculated Parameters	c [MPa]	0.41	0.15	0.07	0.78	0.21
	ϕ [°]	44	25	14	51	25
	E [GPa]	3.0	0.6	0.2	5.6	0.8

		RT-SH 1	RT-SH 2	RT-SH 3	RT-C 1	RT-C 3
Input Parameters	GSI	33	22	not applicable	45	30
	UCS - intact Rock [MPa]	29	10		49	10
	mi	10	8		19	16
	Overburden [m]	130	130		130	130
	Disturbance Faktor	0.5	0.5		0.5	0.5
	Specific Weight [KN/m³]	26	26		26	26
Calculated Parameters	c [MPa]	0.29	0.13		0.58	0.22
	ϕ [°]	31	17		46	26
	E [GPa]	1.5	0.5		3.9	0.8

Table 2: Determination of rock mass parameters according to Hoek-Brown

4.2 Determination of Young's Modulus

In addition to the Hoek-Brown Failure method, Young's Modulus for the rock mass types was determined using the equation of Serafim & Pereira (1983)

$$E_m(GPa) = 10^{(RMR-10)/40} \text{ (for 35<RMR<55)}$$

and the equation of Boyd (1993)

$$E_m(GPa) = 3.5 * RMR^{3.75} / 10^6 \text{ (for RMR<35)}$$

where RMR is the Rock Mass Rating according to Bieniawski, 1999.

The results of these calculations are listed in table 3.

Determination of Young's Modulus according to Serafim & Pereira (1983) and Boyd (1993)

	RT-G 1	RT-G 2	RT-G 3	RT-SS 1	RT-SS 2
Rock Mass Rating (Bieniawski)	37	27	18	49	28
Young's Modulus (Boyd)	-	0.8	0.2	-	0.9
Young's Modulus (Serafim & Pereira)	4.7	-	-	9.4	-

	RT-SH 1	RT-SH 2	RT-SH 3	RT-C 1	RT-C 3
Rock Mass Rating (Bieniawski)	30	20	not applicable	41	28
Young's Modulus (Boyd)	-	-		-	0.9
Young's Modulus (Serafim & Pereira)	1.2	0.3		6.0	-

Table 3: Determination of Young's Modulus according to Serafim & Pereira (1983) and Boyd (1993)

4.3 Method by K.W. John

The method by K. W. John [12] for determining the shear strength of jointed rock, is based on failure criteria, namely shear failure of intact rock and sliding along a joint plane regarding various angles of the joint plane.

The ratio of the minor to the major principal stress is taken into consideration in the calculation using the following factor:

$$\psi = \frac{\sigma_3}{\sigma_1} = 0$$

The stresses at the joint plane which is inclined in respect to the first principal stress at an angle α, can be expressed as follows:

$$\sigma_n = \sigma_1 \left[\frac{1+\psi}{2} + \frac{1-\psi}{2} \cos(2\alpha) \right]$$

$$\tau_n = \sigma_1 \frac{1-\psi}{2} \sin(2\alpha)$$

By inserting these parameters in the failure criterion of the joint plane:

$$\tau_n \leq c_k + \sigma_n \cdot \tan \varphi_k$$

the maximum sustainable principal effective stress can be expressed by the cohesion c_k and the friction angle φ_k of the joint by:

$$\sigma_1 \leq \frac{c_k}{\frac{1-\psi}{2} \sin(2\alpha) - \left[\frac{1+\psi}{2} + \frac{1-\psi}{2} \cos(2\alpha) \right] \tan \varphi_k}$$

The failure criterion in the block limits the principal effective stress to the following maximum value, with c and φ representing the failure parameters in the block:

$$\sigma_1 \leq \frac{c}{\frac{1-\psi}{2} \cos(\varphi) - \left[\frac{1+\psi}{2} + \frac{1-\psi}{2} \sin \varphi \right] \tan \varphi}$$

The angle of the joint planes varies between 0 and 180°. The reduction factor α is determined by integrating the curve of the function which describes the minimum failure.

The mathematical value for the uniaxial compressive strength of the rock mass is determined as follows:

$$\square_{rm} = \square_{intact\ rock} * \square$$

For each rock type two discontinuity sets are regarded. The angle between these joints is determined at the tunnel face which is perpendicular to the tunnel axis striking N25E.

The cohesion of the rock mass can be expressed as follows on account of the friction angle and the uniaxial compressive strength:

$$c_{Geb} = \sigma_{GEB} \left[\frac{1 - \sin(\varphi_{Geb})}{2 \cos(\varphi_{Geb})} \right]$$

Experience shows that the friction angle of the rock mass is only slightly less than the friction of the intact rock. Thus, the friction angle of the rock mass is derived by slight reductions of the friction angle of the intact rock. The friction angle of the rock mass and the compressive strength of the rock mass can be used to calculate the cohesion of the rock mass. The results of this exercise are listed in table 4.

Determination of Rock mass parameters according K.W. John

		Shear Parameters	RT-G 1	RT-G 2	RT-G 3	RT-SS 1	RT-SS 2
Input parameters	Intact Rock	c [MPa]	4.9	1.4	not applicable	22.78	not applicable
		ϕ [°]	45	30		45	
	Joint set 1	c [MPa]	0.08	0.04		0.02	
		ϕ [°]	34	22		30	
	Joint set 2	c [MPa]	0.08	0.04		0.02	
		ϕ [°]	34	22		30	
Calculated Parameters		UCS [MPa]	1.9	0.5		3.2	
		c [MPa]	0.49	0.15		0.83	
		ϕ [°]	35	25		35	

		Shear Parameters	RT-SH 1	RT-SH 2	RT-SH 3	RT-C1	RT-C 2
Input parameters	Intact Rock	c [MPa]	9.63	not applicable	not applicable	11.42	not applicable
		ϕ [°]	30			40	
	Joint set 1	c [MPa]	0.01			0.1	
		ϕ [°]	29			37	
	Joint set 2	c [MPa]	0.01			0.1	
		ϕ [°]	29			37	
Calculated Parameters		UCS [MPa]	0.9			3.4	
		c [MPa]	0.25			0.89	
		ϕ [°]	30			35	

Table 4: Determination of rock mass parameters according to K.W. John

4.4 Range of Geotechnical Parameters

It is to be noted that, in addition to the applied methods for choosing the final rock mass parameters mentioned above, engineering judgment based on experience is still required. The results of this procedure are presented in Table 5.

		Rock mass Parameters according to Hoek-Brown (2002 Edition)									
		RT-G 1	RT-G 2	RT-G 3	RT-SS 1	RT-SS 2	RT-SH 1	RT-SH 2	RT-SH 3	RT-C 1	RT-C 2
Calculated Parameters	c [MPa]	0.41	0.15	0.07	0.78	0.21	0.29	0.13	NA	0.58	0.22
	ϕ [°]	44	25	14	51	25	31	17		46	26
	E [GPa]	3.0	0.6	0.2	5.6	0.8	1.5	0.5		3.9	0.8

		Young's Modulus according to Serafim & Pereira (1983)/Boyd (1993)									
		RT-G 1	RT-G 2	RT-G 3	RT-SS 1	RT-SS 2	RT-SH 1	RT-SH 2	RT-SH 3	RT-C 1	RT-C 2
Calculated Parameters	RMR	37	27	18	49	28	30	20	NA	41	28
	E [GPa] (Boyd)		0.8	0.2		0.9	1.2	0.3			0.9
	E [GPa] (Serafim & Pereira)	4.7			9.4					6.0	

		Shear Strength of Rock Mass - K.W. John (1969)									
		RT-G 1	RT-G 2	RT-G 3	RT-SS 1	RT-SS 2	RT-SH 1	RT-SH 2	RT-SH 3	RT-C 1	RT-C 2
Calculated Parameters	UCS [MPa]	1.9	0.5	NA	3.2	NA	0.9	NA	NA	3.4	NA
	c [MPa]	0.49	0.15		0.83		0.25			0.89	
	ϕ [°]	35	25		35		30			35	

		Proposed Rock Mass Parameters									
		RT-G 1	**RT-G 2**	**RT-G 3**	**RT-SS 1**	**RT-SS 2**	**RT-SH 1**	**RT-SH 2**	**RT-SH 3**	**RT-C 1**	**RT-C 2**
Proposed Parameters	**E [GPa]**	**2-4**	**0.4-0.8**	**0.2-0.4**	**4-6**	**0.5-1.0**	**1-2**	**0.5-1.0**	**0.03-0.05**	**3-5**	**0.5-1.0**
	c [MPa]	**0.4-0.6**	**0.15-0.25**	**0.05-0.15**	**0.7-1.0**	**0.2-0.3**	**0.25-0.35**	**0.15-0.25**	**0.01-0.02**	**0.6-1.0**	**0.2-0.3**
	ϕ [°]	**35-40**	**25-30**	**15-25**	**40-45**	**20-30**	**25-30**	**20-25**	**20-25**	**40-45**	**25-30**

Table 5: Proposed rock mass parameters

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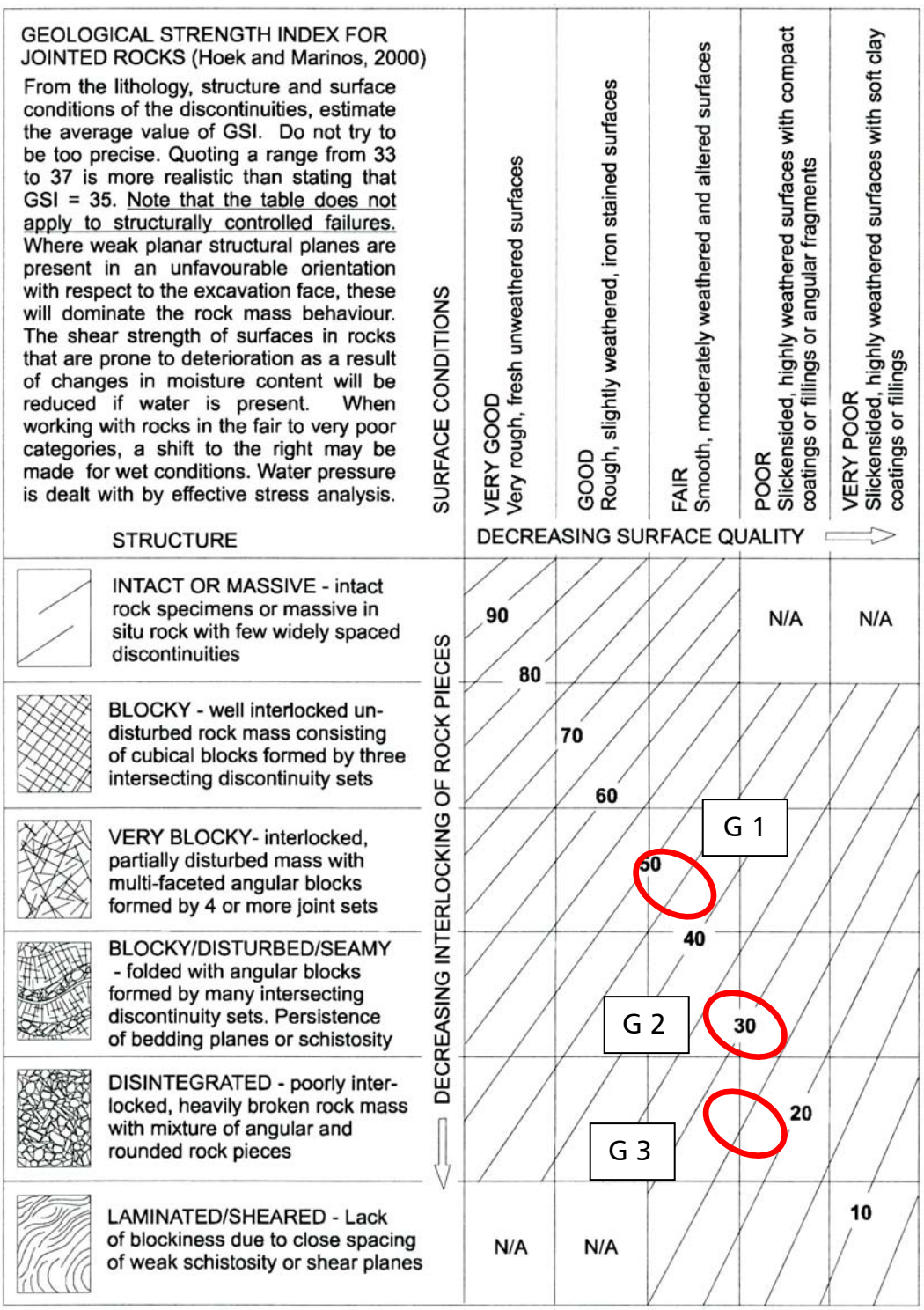
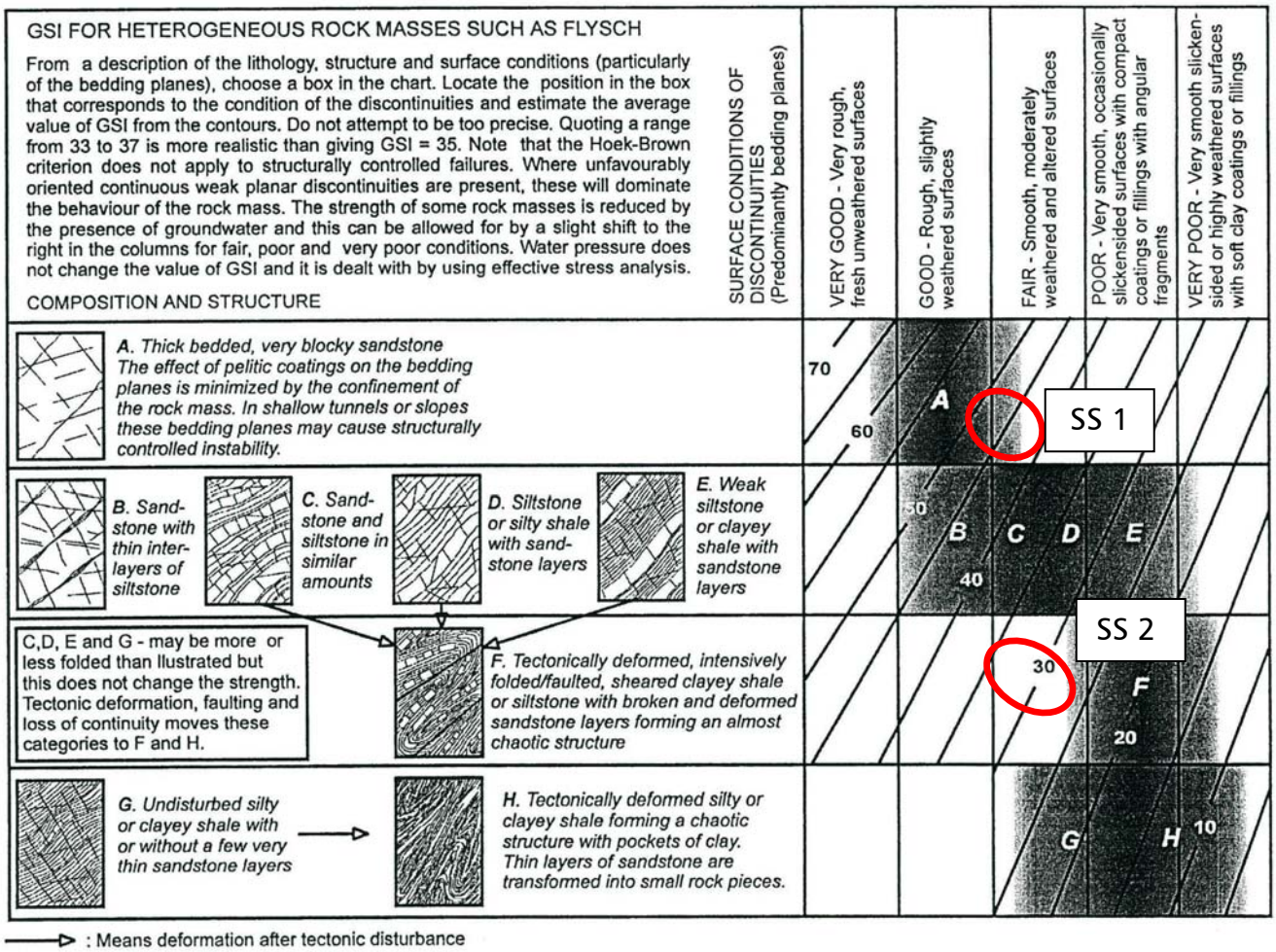
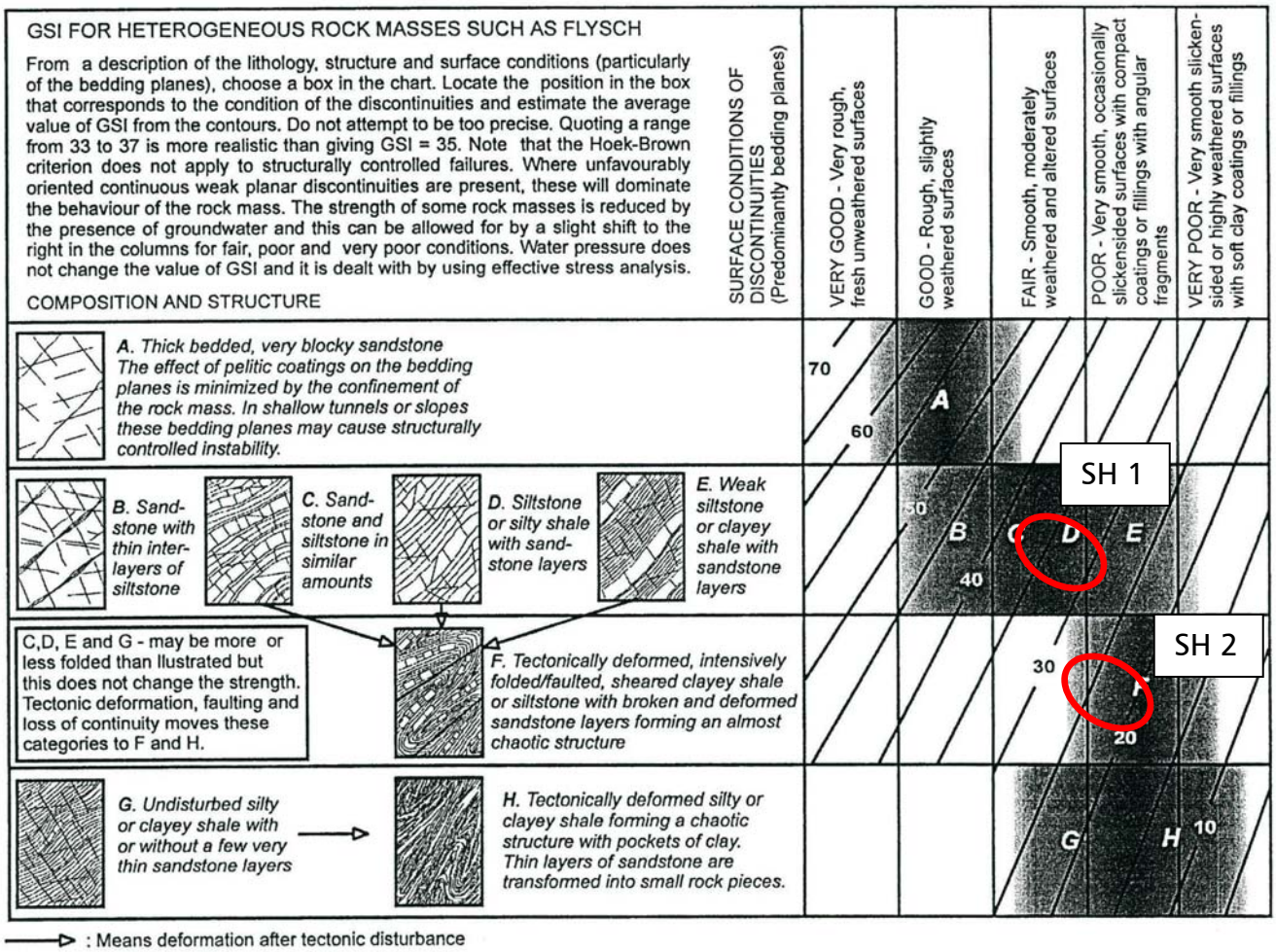
GSI Chart Granitic Rock



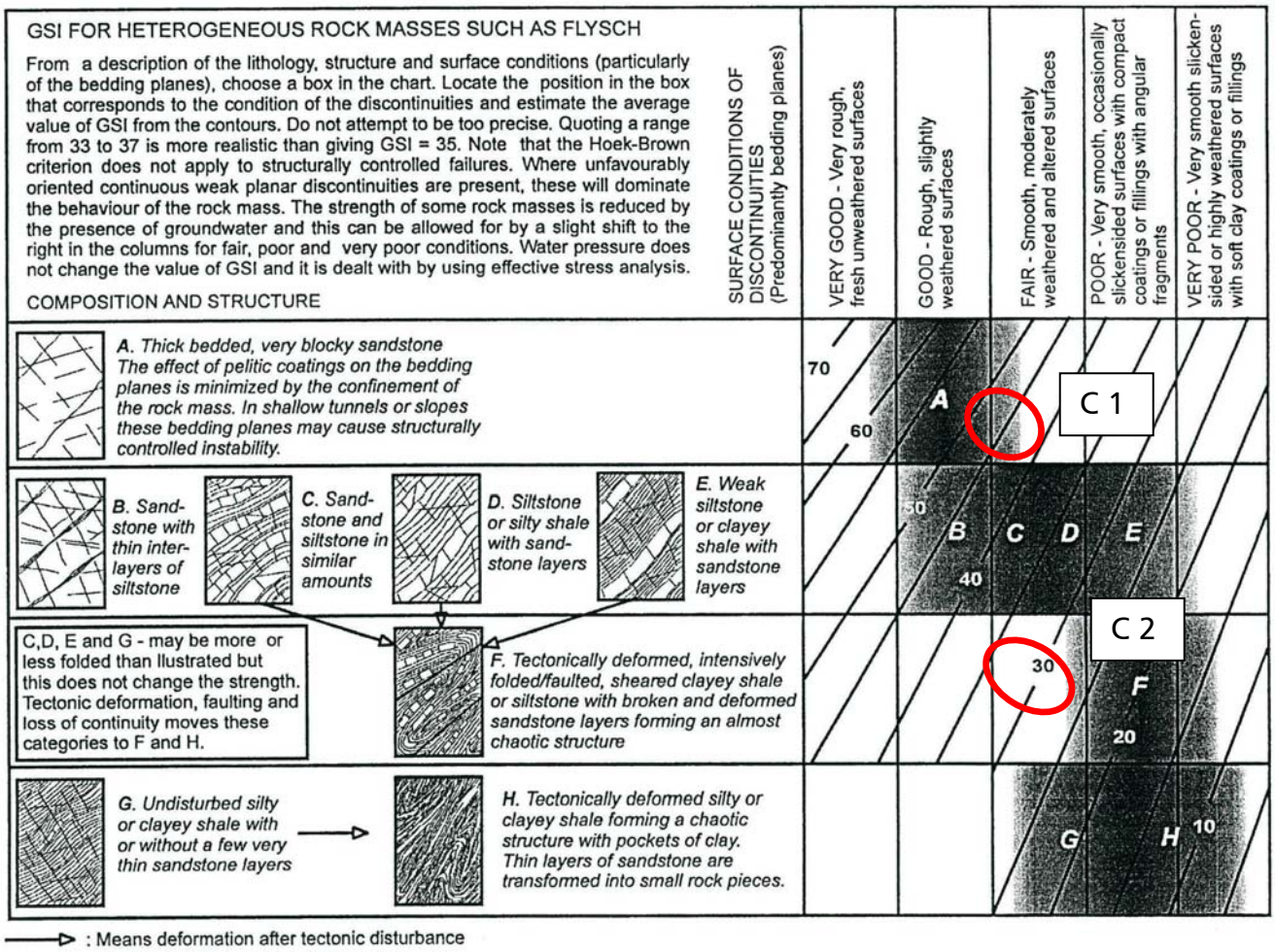
GSI Chart Sandstone



GSI Chart Siltstone/Claystone



GSI Chart Conglomerate



m_i Chart

Rock type	Class	Group	Texture: Coarse	Texture: Medium	Texture: Fine	Texture: Very fine
SEDIMENTARY	Clastic		Conglomerate (22)	Sandstone 19	Siltstone 9	Claystone 4
				Greywacke (18)		
	Non-Clastic	Organic		Chalk 7		
				Coal (8-21)		
		Carbonate	Breccia (20)	Sparitic Limestone (10)	Micritic Limestone 8	
		Chemical		Gypstone 16	Anhydrite 13	
METAMORPHIC	Non Foliated		Marble 9	Hornfels (19)	Quartzite 24	
	Slightly foliated		Migmatite (30)	Amphibolite 25 - 31	Mylonites (6)	
	Foliated*		Gneiss 33	Schists 4 - 8	Phyllites (10)	Slate 9
IGNEOUS	Light		Granite 33		Rhyolite (16)	Obsidian (19)
			Granodiorite (30)		Dacite (17)	
	Dark		Diorite (28)		Andesite 19	
			Gabbro 27	Dolerite (19)	Basalt (17)	
			Norite 22			
	Extrusive pyroclastic type		Agglomerate (20)	Breccia (18)	Tuff (15)	

Hoek-Brown Failure Criterion (2002 Edition) RT-G 1

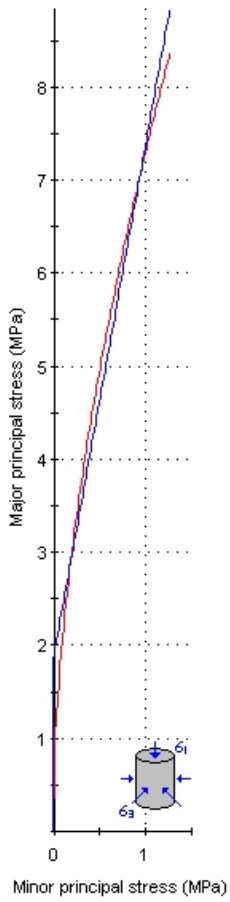
Analysis of Rock Strength using RocLab

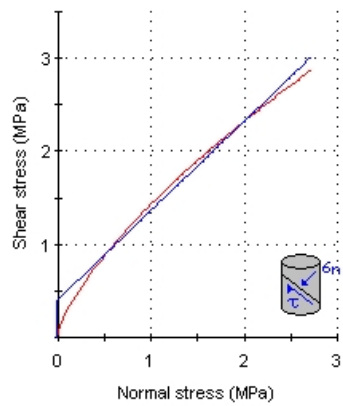
Hoek-Brown Classification
intact uniaxial compressive strength = 28.4 MPa
GSI = 45 mi = 20 Disturbance factor = 0.5

Hoek-Brown Criterion
mb = 1.457 s = 0.0007 a = 0.508

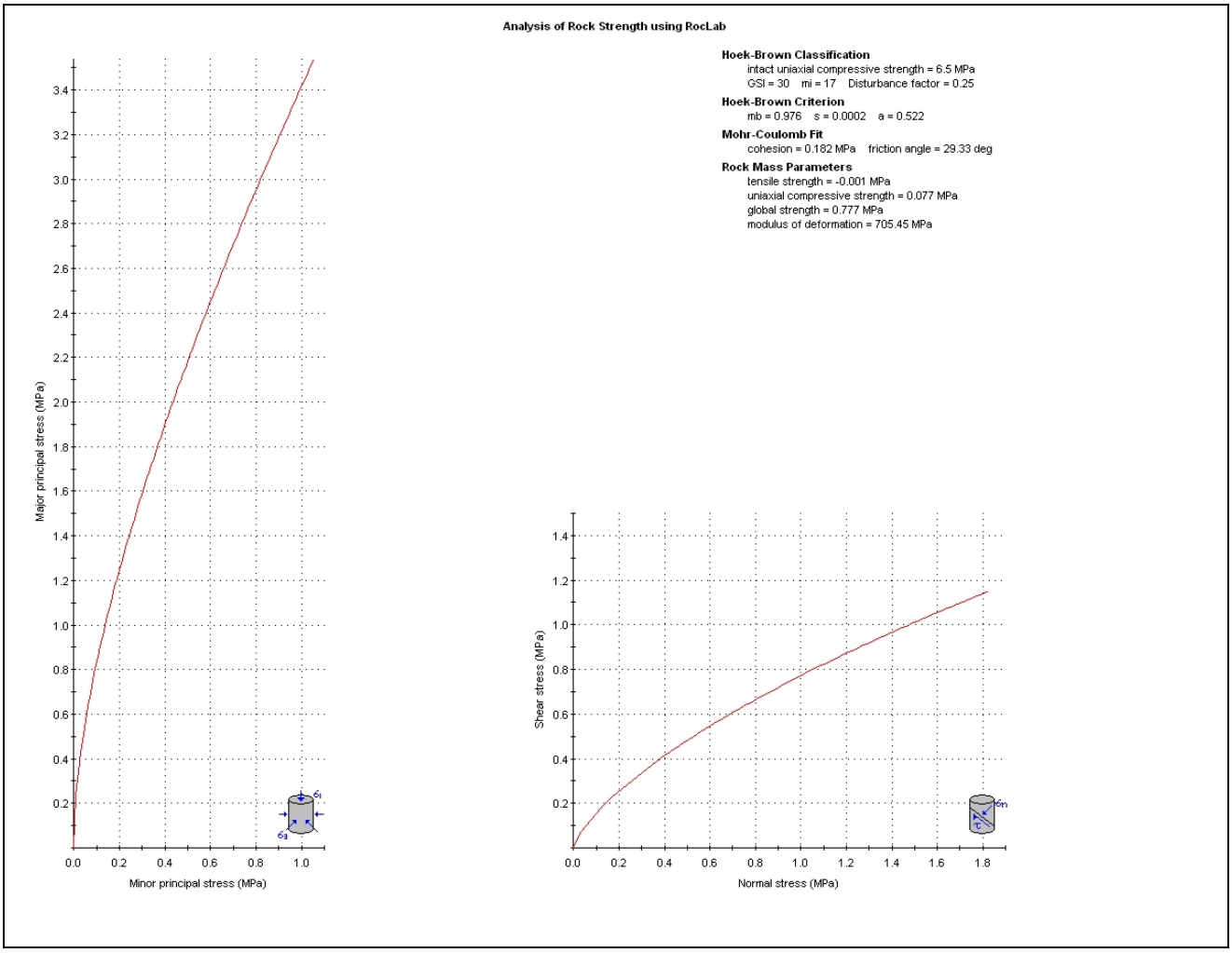
Mohr-Coulomb Fit
cohesion = 0.407 MPa friction angle = 43.78 deg

Rock Mass Parameters
tensile strength = -0.013 MPa
uniaxial compressive strength = 0.684 MPa
global strength = 4.451 MPa
modulus of deformation = 2997.23 MPa

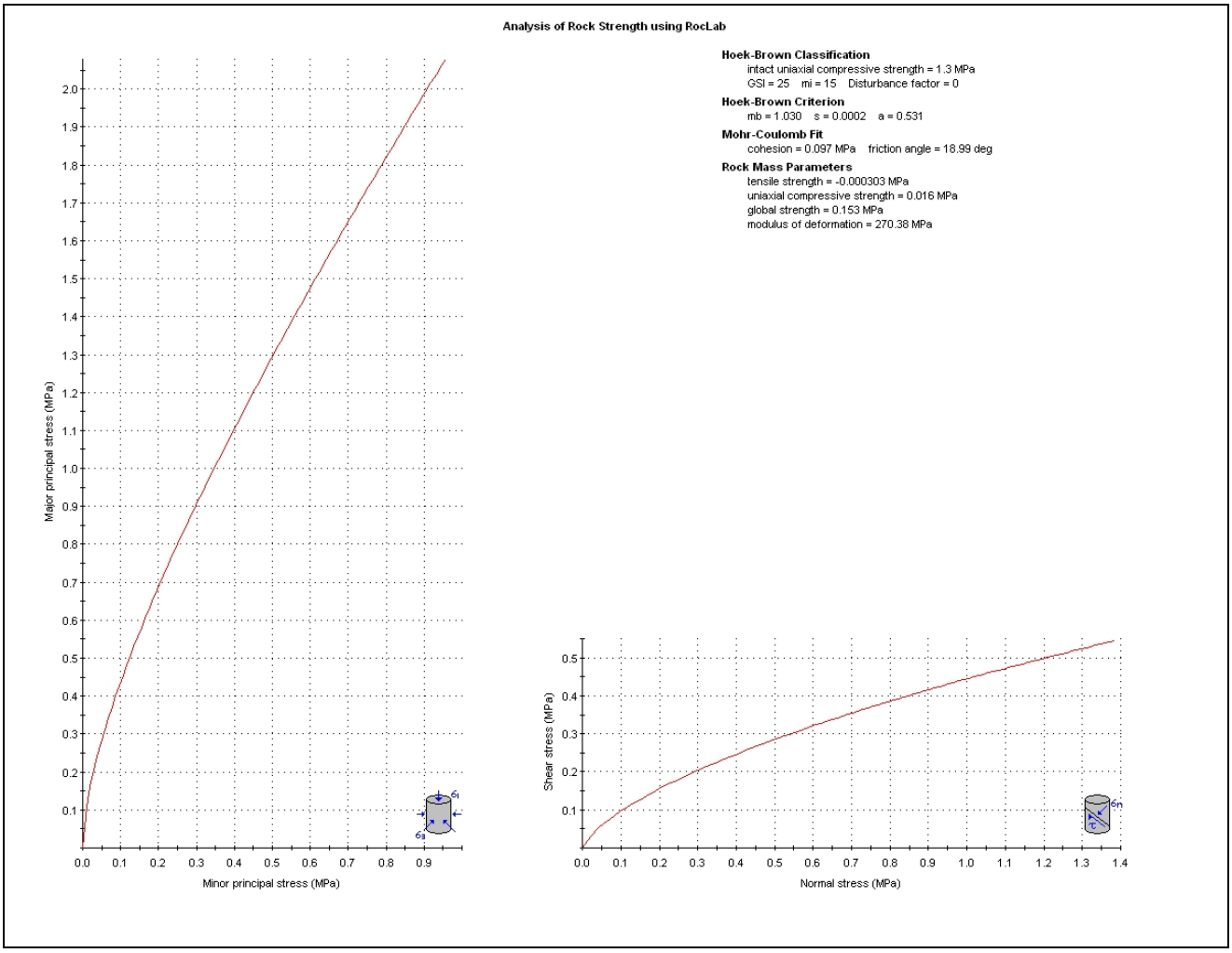




Hoek-Brown Failure Criterion (2002 Edition) RT-G 2

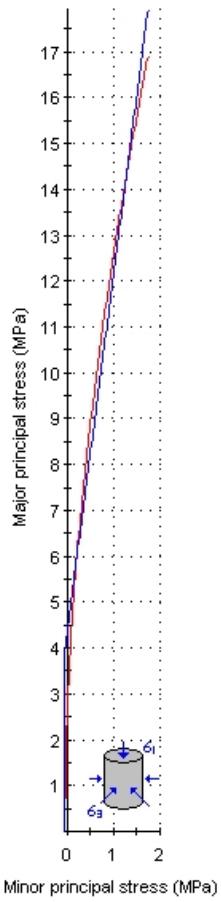


Hoek-Brown Failure Criterion (2002 Edition) RT-G 3



Hoek-Brown Failure Criterion (2002 Edition) RT-SS 1

Analysis of Rock Strength using RocLab

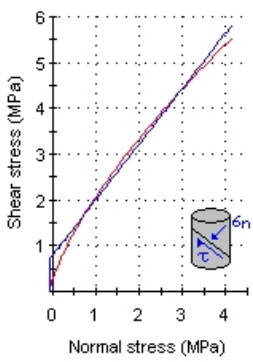


Hoek-Brown Classification
intact uniaxial compressive strength = 110 MPa
GSI = 45 mi = 17 Disturbance factor = 0.5

Hoek-Brown Criterion
mb = 1.239 s = 0.0007 a = 0.508

Mohr-Coulomb Fit
cohesion = 0.778 MPa friction angle = 50.58 deg

Rock Mass Parameters
tensile strength = -0.058 MPa
uniaxial compressive strength = 2.650 MPa
global strength = 15.892 MPa
modulus of deformation = 5624.21 MPa



Hoek-Brown Failure Criterion (2002 Edition) RT-SS 2

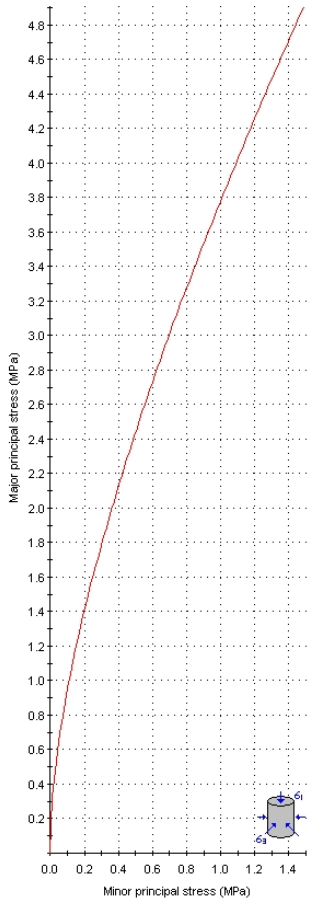
Analysis of Rock Strength using RocLab

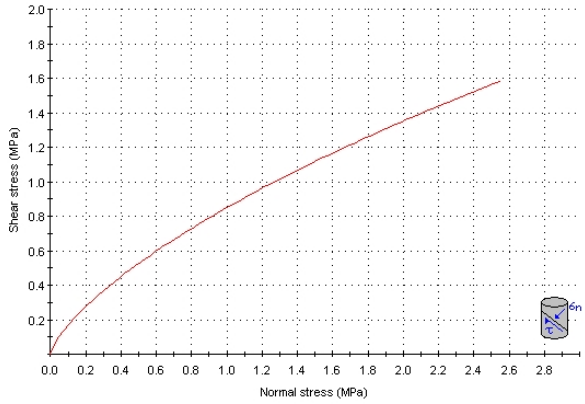
Hoek-Brown Classification
intact uniaxial compressive strength = 10 MPa
GSI = 30 mi = 15 Disturbance factor = 0.25

Hoek-Brown Criterion
mb = 0.861 s = 0.0002 a = 0.522

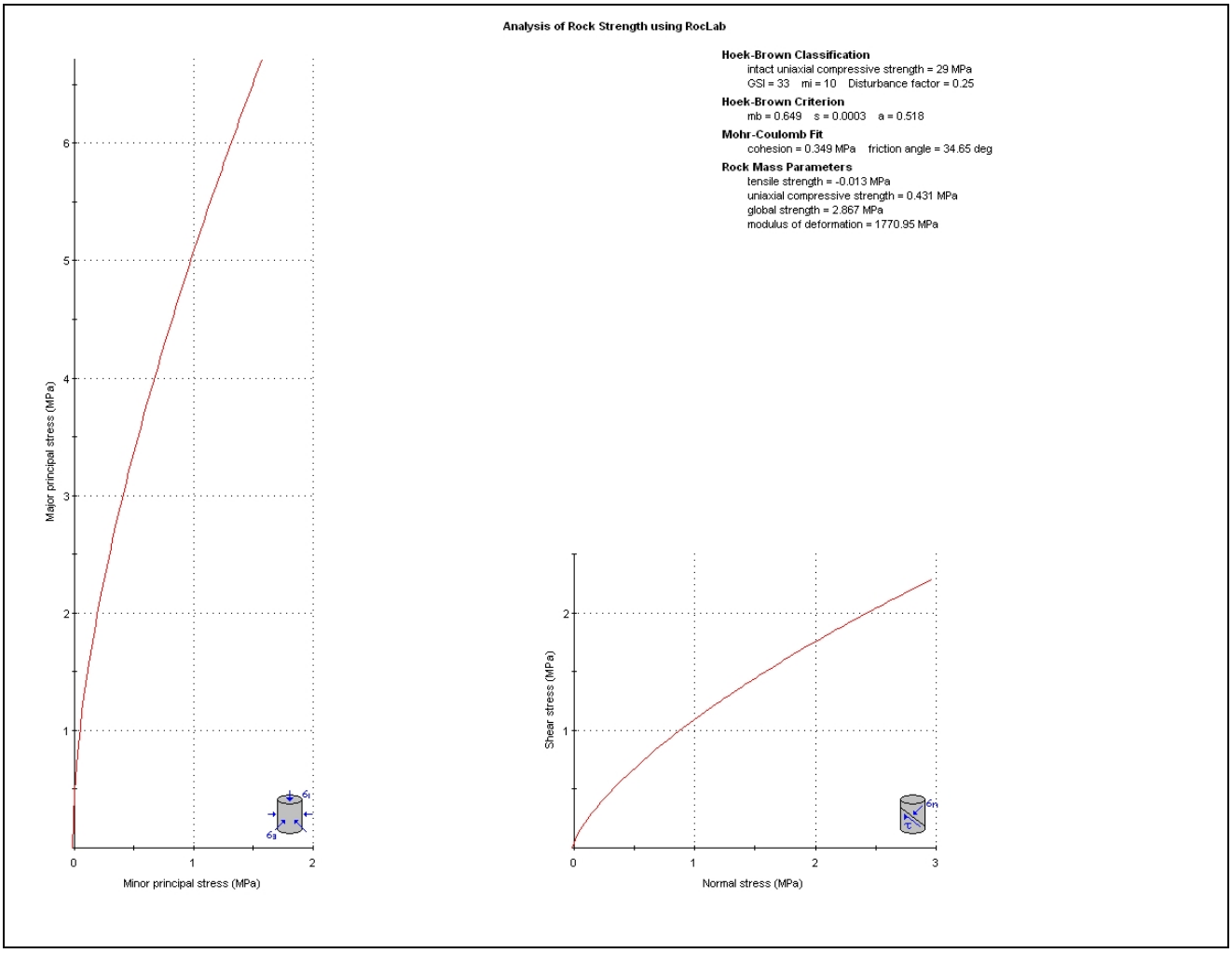
Mohr-Coulomb Fit
cohesion = 0.252 MPa friction angle = 28.94 deg

Rock Mass Parameters
tensile strength = -0.002 MPa
uniaxial compressive strength = 0.119 MPa
global strength = 1.120 MPa
modulus of deformation = 875.00 MPa

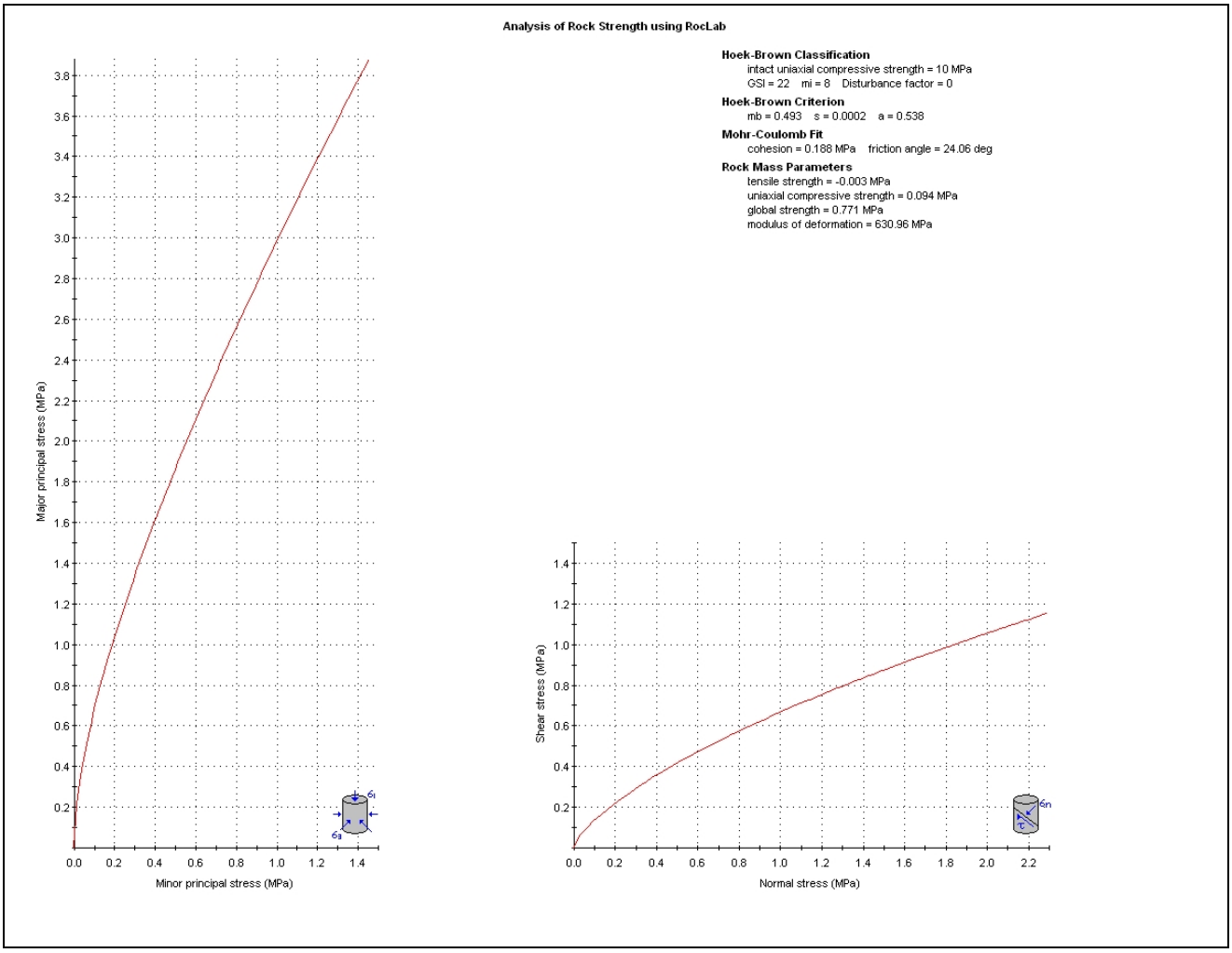




Hoek-Brown Failure Criterion (2002 Edition) RT-SH 1

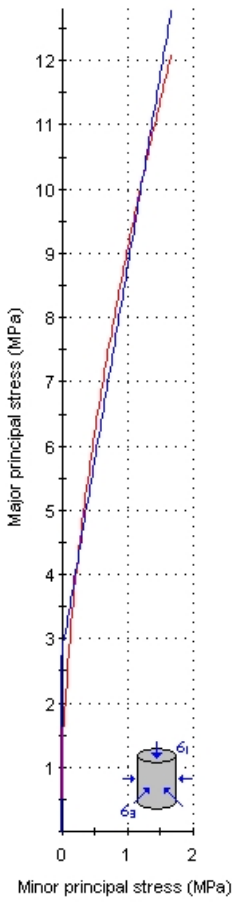


Hoek-Brown Failure Criterion (2002 Edition) RT-SH 2



Hoek-Brown Failure Criterion (2002 Edition) RT-C 1

Analysis of Rock Strength using RocLab

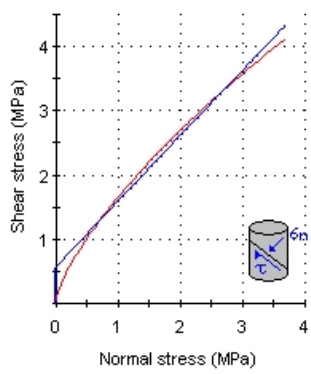


Hoek-Brown Classification
intact uniaxial compressive strength = 49 MPa
GSI = 45 mi = 19 Disturbance factor = 0.5

Hoek-Brown Criterion
mb = 1.385 s = 0.0007 a = 0.508

Mohr-Coulomb Fit
cohesion = 0.579 MPa friction angle = 45.53 deg

Rock Mass Parameters
tensile strength = -0.023 MPa
uniaxial compressive strength = 1.180 MPa
global strength = 7.485 MPa
modulus of deformation = 3936.94 MPa



Hoek-Brown Failure Criterion (2002 Edition) RT-C 2

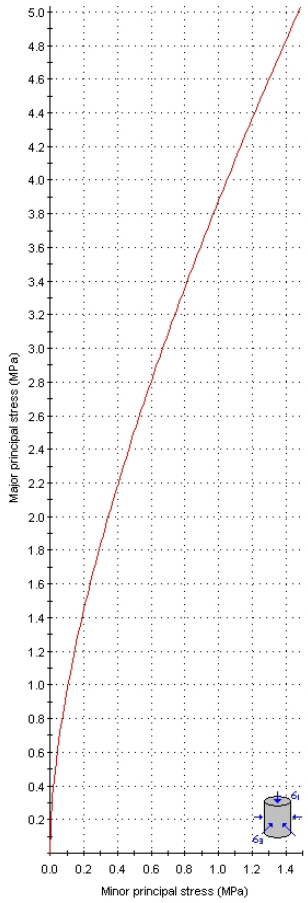
Analysis of Rock Strength using RocLab

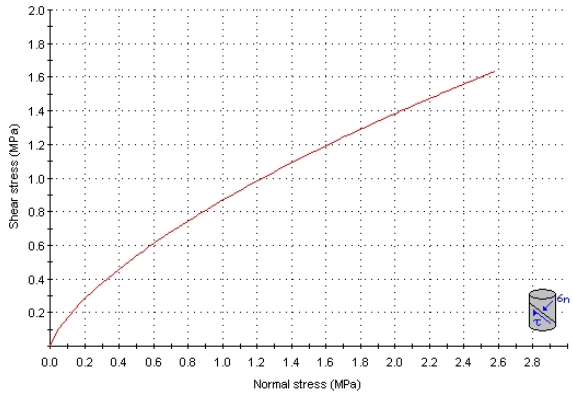
Hoek-Brown Classification
intact uniaxial compressive strength = 10 MPa
GSI = 30 mi = 16 Disturbance factor = 0.25

Hoek-Brown Criterion
mb = 0.919 s = 0.0002 a = 0.522

Mohr-Coulomb Fit
cohesion = 0.258 MPa friction angle = 29.48 deg

Rock Mass Parameters
tensile strength = -0.002 MPa
uniaxial compressive strength = 0.119 MPa
global strength = 1.158 MPa
modulus of deformation = 875.00 MPa





Rock Mass Type: G 1

Rock Mass Type (RT)	G-1
Lithology	granitic rock

Geological short description	slightly to moderately fractured, massive granitic rock
Spacing	10 to 50
Surface Shape/Roughness	planar to irregular, slightly rough to rough
Persistence	<3 to >6 meters
Opening/Infilling (% of in-filled joints)	generally slightly to moderately open (1-3mm), predominantly clay (80%), calcite (15%), sand (5%), some iron oxide staining
Weathering	fresh to slightly weathered/altered

	Des.	Dim	Mean Value	Deviation from Mean Value (+/-)	Comment
Parameters of Intact Rock					
Density		KN/m^3	25.9	0.4	9 tests
Uniaxial Compressive Strength	UCS	MPa	28.4	15.9	34 tests*
Hoek-Constant	mi	-	20		
Young's Modulus	E	GPa	13.7		
Poisson's Ratio	ν	-	0.24		1 test
Parameters of Discontinuities					
Angle of Internal Friction	ϕ	degree	38	3	10 tests
Cohesion	c	MPa	0,08	0.04	10 tests
Rock Mass Parameters (Hoek-Brown)					
Geological Strength Index	GSI	-	45	5	
Density		KN/m^3	26.0		
Disturbance Factor	D		0.5		
Tunnel Depth		m	100.0		
Cohesion	c	MPa	0.41		
Angle of Internal Friction	ϕ	degree	44		
Young's Modulus	E	GPa	3.0		
Young's Modulus (Serafim& Pereira, 1983/Boyd, 1993)					
RMR	–	-	37		
Young's Modulus	E	GPa	4.7		Serafim & Pereira
Rock Mass Parameters (K.W. John)					
Uniaxial Compressive Strength	UCS	MPa	1.9		
Cohesion	c	MPa	0.49		
Angle of Internal Friction	ϕ	degree	35		

Proposed Rock Mass Parameters					
Density	γ	KN/m^3	26	1	
Young's Modulus	E	GPa	3	1	
Cohesion	c	MPa	0.5	0.1	
Angle of Internal Friction	ϕ	degree	37.5	2.5	
Uniaxial Compressive Strength	UCS	MPa	2.0	0.5	
Poisson's Ratio	ν	-	0.25		

	estimated values
	calculated values

* including 25 Point Load Tests

Rock Mass Type: G 2

Rock Mass Type (RT)	G-2
Lithology	granitic rock

Geological short description	intensely to very intensely fractured, massive granitic rock
Spacing	3 to 10 cm
Surface Shape/Roughness	planar to irregular, slightly rough to smooth
Persistence	<3 meters
Opening/Infilling (% of in-filled joints)	generally slightly to moderately open (1-3mm), predominantly clay (85%) some sand, iron oxide staining is also common due to degree of weathering
Weathering	moderately to strongly weathered/altered

	Des.	Dim	Mean Value	Deviation from Mean Value (+/-)	Comment
Parameters of Intact Rock					
Density		KN/m^3	23.7	1.1	9 tests
Uniaxial Compressive Strength	UCS	MPa	6.5	2.3	8 tests
Hoek-Constant	mi	-	17		
Young's Modulus	E	GPa			
Poisson's Ratio	ν	-	0.30		1 test
Parameters of Discontinuities					
of Internal Friction	ϕ	degree	22		
Cohesion	c	MPa	0.04		
Rock Mass Parameters (Hoek-Brown)					
Geological Strength Index	GSI	-	30	5	
Density		KN/m^3	24.0		
Disturbance Factor	D		0.25		
Tunnel Depth		m	100.0		
Cohesion	c	MPa	0.19		
Angle of Internal Friction	ϕ	degree	29		
Young's Modulus	E	GPa	0.7		
Young's Modulus (Serafim& Pereira, 1983/Boyd, 1993)					
RMR	–	-	27		
Young's Modulus	E	GPa	0.8		Boyd
Rock Mass Parameters (K.W. John)					
Uniaxial Compressive Strength	UCS	MPa	0.5		
Cohesion	c	MPa	0.15		
Angle of Internal Friction	ϕ	degree	25		

Proposed Rock Mass Parameters					
Density	γ	KN/m^3	24	1.5	
Young's Modulus	E	GPa	0.7	0.2	
Cohesion	c	MPa	0.2	0.05	
Angle of Internal Friction	ϕ	degree	27.5	2.5	
Uniaxial Compressive Strength	UCS	MPa	0.7	0.2	
Poisson's Ratio	ν	-	0.25		

	estimated values
	calculated values

Rock Mass Type: G 3

Rock Mass Type (RT)	G-3
Lithology	granitic rock- fault zone

Geological short description	very intensely fractured granitic rock, rock mass is disintegrated due to weathering and tectonic strain, occurrence of clayey gouge common, this rock mass type is bound to fault zones
Spacing	<3 cm
Surface Shape/Roughness	planar to irregular, slightly rough to smooth
Persistence	<3 meters
Opening/Infilling (% of in-filled joints)	generally slightly to moderately open (1-3mm), predominantly clay (85%) some sand
Weathering	moderately to strongly weathered/altered with partly disintegration of rock

	Des.	Dim	Mean Value	Deviation from Mean Value (+/-)	Comment
Parameters of Intact Rock					
Density		KN/m^3	23.7	1.1	9 tests
Uniaxial Compressive Strength	UCS	MPa	1.3	0.7	9 tests
Hoek-Constant	mi	-	15		
Young's Modulus	E	GPa			
Poisson's Ratio	ν	-			
Parameters of Discontinuities					
of Internal Friction	ϕ	degree			
Cohesion	c	MPa			
Rock Mass Parameters (Hoek-Brown)					
Geological Strength Index	GSI	-	25	5	
Density		KN/m^3	24.0		
Disturbance Factor	D		0.0		
Tunnel Depth		m	100.0		
Cohesion	c	MPa	0.10		
Angle of Internal Friction	ϕ	degree	19		
Young's Modulus	E	GPa	0.3		
Young's Modulus (Serafim& Pereira, 1983/Boyd, 1993)					
RMR	–	-	18		
Young's Modulus	E	GPa	0.2		Boyd
Rock Mass Parameters (K.W. John)					
Uniaxial Compressive Strength	UCS	MPa	NA		
Cohesion	c	MPa	NA		
Angle of Internal Friction	ϕ	degree	NA		

Proposed Rock Mass Parameters					
Density	γ	KN/m^3	24	1.5	
Young's Modulus	E	GPa	0.5	0.1	
Cohesion	c	MPa	0.1	0.05	
Angle of Internal Friction	ϕ	degree	20	5	
Uniaxial Compressive Strength	UCS	MPa	0.3	0.1	
Poisson's Ratio	ν	-	0.35		

	estimated values
	calculated values

Rock Mass Type: SS 1

Rock Mass Type (RT)	SS-1
Lithology	sandstone

Geological short description	moderately to slightly fractured, fine to coarse grained sandstone, massive to thick bedded, with few siltstone/claystone interbeds possible
Spacing	10 to 40 cm
Surface Shape/Roughness	planar to irregular, slightly rough to rough
Persistence	<3 to >6 meters
Opening/Infilling (% of in-filled joints)	generally tight to slightly open (0-1mm), predominantly calcite, few clay (10%) fillings
Weathering	fresh to slightly weathered

	Des.	Dim	Mean Value	Deviation from Mean Value (+/-)	Comment
Parameters of Intact Rock					
Density		KN/m^3	25.9	0.2	36 tests
Uniaxial Compressive Strength	UCS	MPa	110	31.2	15 tests
Hoek-Constant	mi	-	17		
Young's Modulus	E	GPa	48	12.6	8 tests
Poisson's Ratio	ν	-	0.20	0.04	7 tests
Parameters of Discontinuities					
Angle of Internal Friction	ϕ	degree	30	5.3	3 tests
Cohesion	c	MPa	0.02	0.02	3 tests
Rock Mass Parameters (Hoek-Brown)					
Geological Strength Index	GSI	-	45	5	
Density		KN/m^3	26.0		
Disturbance Factor	D		0.5		
Tunnel Depth		m	130		
Cohesion	c	MPa	0.78		
Angle of Internal Friction	ϕ	degree	51		
Young's Modulus	E	GPa	5.6		
Young's Modulus (Serafim& Pereira, 1983/Boyd, 1993)					
RMR	–	-	49		
Young's Modulus	E	GPa	9.4		Serafim & Pereira
Rock Mass Parameters (K.W. John)					
Uniaxial Compressive Strength	UCS	MPa	3.2		
Cohesion	c	MPa	0.83		
Angle of Internal Friction	ϕ	degree	35		

Proposed Rock Mass Parameters					
Density	γ	KN/m^3	26	1	
Young's Modulus	E	GPa	5	1	
Cohesion	c	MPa	0.85	0.15	
Angle of Internal Friction	ϕ	degree	42.5	2.5	
Uniaxial Compressive Strength	UCS	MPa	3.9	1	
Poisson's Ratio	ν	-	0.25		

	estimated values
	calculated values

Rock Mass Type: SS 2

Rock Mass Type (RT)	SS-2
Lithology	sandstone

Geological short description	intensely fractured, fine to coarse grained sandstone
Spacing	3 to 10 cm
Surface Shape/Roughness	planar to irregular, slightly rough to rough
Persistence	< 3 meters
Opening/Infilling (% of in-filled joints)	generally tight to slightly open (0-1mm), predominantly calcite, few clay (10%) fillings
Weathering	moderately to intensely weathered

	Des.	Dim	Mean Value	Deviation from Mean Value (+/-)	Comment
Parameters of Intact Rock					
Density		KN/m^3	25.9	0.2	36 tests
Uniaxial Compressive Strength	UCS	MPa	10		
Hoek-Constant	mi	-	15		
Young's Modulus	E	GPa	2.3	3.0	3 tests
Poisson's Ratio	ν	-	0.33	0.04	3 tests
Parameters of Discontinuities					
Angle of Internal Friction	ϕ	degree	30	5.3	3 tests
Cohesion	c	MPa	0.02	0.02	3 tests
Rock Mass Parameters (Hoek-Brown)					
Geological Strength Index	GSI	-	30	5	
Density		KN/m^3	26.0		
Disturbance Factor	D		0.25		
Tunnel Depth		m	130		
Cohesion	c	MPa	0.25		
Angle of Internal Friction	ϕ	degree	29		
Young's Modulus	E	GPa	0.9		
Young's Modulus (Serafim& Pereira, 1983/Boyd, 1993)					
RMR	–	-	28		
Young's Modulus	E	GPa	0.9		Boyd
Rock Mass Parameters (K.W. John)					
Uniaxial Compressive Strength	UCS	MPa	NA		
Cohesion	c	MPa	NA		
Angle of Internal Friction	ϕ	degree	NA		

Proposed Rock Mass Parameters					
Density	γ	KN/m^3	26	1	
Young's Modulus	E	GPa	1.0	0.2	
Cohesion	c	MPa	0.25	0.05	
Angle of Internal Friction	ϕ	degree	25	5	
Uniaxial Compressive Strength	UCS	MPa	0.8	0.2	
Poisson's Ratio	ν	-	0.25		

	estimated values
	calculated values

Rock Mass Type: SH 1

Rock Mass Type (RT)	SH 1
Lithology	interbedding of claystone, siltstone and sandstone

Geological short description	moderately to slightly fractured interbedding of claystone, siltstone and sandstone, the amount of the lithological units varies between similar to a slight predominance of the fine grained units, conglomerate interbeds possible
Spacing	10 to 40 cm
Surface Shape/Roughness	planar to irregular, smooth to rough
Persistence	<3 to >6 meters
Opening/Infilling (% of in-filled joints)	generally tight to slightly open (0-1mm), predominantly calcite, few clay (10%) fillings
Weathering	fresh to slightly weathered

	Des.	Dim	Mean Value	Deviation from Mean Value (+/-)	Comment
Parameters of Intact Rock					
Density	γ	KN/m^3	26.2	0.4	29 tests
Uniaxial Compressive Strength	UCS	MPa	29	10.2	20 tests
Hoek-Constant	mi	-	10		
Young's Modulus	E	GPa	20	18.5	10 tests
Poisson's Ratio	ν	-	0.26	0.00	2 tests
Parameters of Discontinuities					
Angle of Internal Friction	ϕ	degree	29	14.6	7 tests
Cohesion	c	MPa	0.01	0.005	6 tests
Rock Mass Parameters (Hoek-Brown)					
Geological Strength Index	GSI	-	33	5	
Density	-	KN/m^3	26.0		
Disturbance Factor	D	-	0.25		
Tunnel Depth	-	m	130		
Cohesion	c	MPa	0.35		
Angle of Internal Friction	ϕ	degree	35		
Young's Modulus	E	GPa	1.8		
Young's Modulus (Serafim& Pereira, 1983/Boyd, 1993)					
RMR	–	-	30		
Young's Modulus	E	GPa	1.2		Boyd
Rock Mass Parameters (K.W. John)					
Uniaxial Compressive Strength	UCS	MPa	0.9		
Cohesion	c	MPa	0.25		
Angle of Internal Friction	ϕ	degree	30		

Proposed Rock Mass Parameters					
Density	γ	KN/m^3	26	1	
Young's Modulus	E	GPa	1.5	0.5	
Cohesion	c	MPa	0.3	0.05	
Angle of Internal Friction	ϕ	degree	27.5	2.5	
Uniaxial Compressive Strength	UCS	MPa	1.0	0.2	
Poisson's Ratio	ν	-	0.25		

	estimated values
	calculated values

Rock Mass Type: SH 2

Rock Mass Type (RT)	SH 2
Lithology	interbedding of claystone and siltstone

Geological short description	intensely to very intensely fractured interbedding of claystone, siltstone and sandstone, occurrence conglomerate interbeds possible, rock mass commonly sheared and crushed
Spacing	3 to 10 cm
Surface Shape/Roughness	planar to irregular, smooth to rough
Persistence	< 3 meters
Opening/Infilling (% of in-filled joints)	generally tight to slightly open (0-1mm), predominantly calcite, few clay (10%) fillings
Weathering	moderately to intensely weathered

	Des.	Dim	Mean Value	Deviation from Mean Value (+/-)	Comment
Parameters of Intact Rock					
Density	γ	KN/m^3	26.2	0.4	29 tests
Uniaxial Compressive Strength	UCS	MPa	10		
Hoek-Constant	mi	-	8		
Young's Modulus	E	GPa			
Poisson's Ratio	ν	-	0.31	0.00	2 Tests
Parameters of Discontinuities					
Angle of Internal Friction	ϕ	degree			
Cohesion	c	MPa			
Rock Mass Parameters (Hoek-Brown)					
Geological Strength Index	GSI	-	22	5	
Density	-	KN/m^3	26.0		
Disturbance Factor	D	-	0.0		
Tunnel Depth	-	m	130		
Cohesion	c	MPa	0.19		
Angle of Internal Friction	ϕ	degree	24		
Young's Modulus	E	GPa	0.6		
Young's Modulus (Serafim& Pereira, 1983/Boyd, 1993)					
RMR	–	-	20		
Young's Modulus	E	GPa	0.3		Boyd
Rock Mass Parameters (K.W. John)					
Uniaxial Compressive Strength	UCS	MPa	NA		
Cohesion	c	MPa	NA		
Angle of Internal Friction	ϕ	degree	NA		

Proposed Rock Mass Parameters					
Density	γ	KN/m^3	26	1	
Young's Modulus	E	GPa	0.4	0.1	
Cohesion	c	MPa	0.2	0.05	
Angle of Internal Friction	ϕ	degree	22.5	2.5	
Uniaxial Compressive Strength	UCS	MPa	0.6	0.2	
Poisson's Ratio	ν	-	0.3		

	estimated values
	calculated values

Rock Mass Type SH 3

Rock Mass Type (RT)	SH 3
Lithology	Fault Gouge

Geological short description	heavily sheared claystone and siltstone disintegrated to soil like rock mass with more competent sandstone/conglomerate blocks/layers in between (block in matrix structure), when in contact with water rock mass is softening
Spacing	not applicable
Surface Shape/Roughness	not applicable
Persistence	not applicable
Opening/Infilling (% of in-filled joints)	not applicable
Weathering	intensely weathered to disintegrated

	Des.	Dim	Mean Value	Deviation from Mean Value (+/-)	Comment
Parameters of Intact Rock					
Density	γ	KN/m^3	23.9	0.8	2 tests
Uniaxial Compressive Strength	UCS	MPa	0.01		1 test
Hoek-Constant	mi	-			
Angle of Internal Friction	ϕ	degree	15.5	2.5	1 test
Cohesion	c	kPa	55	14	1 test
Young's Modulus	E	GPa			
Poisson's Ratio	ν	-			

Proposed Rock Mass Parameters					
Density	γ	KN/m^3	24	1	
Young's Modulus	E	MPa	40	10	
Cohesion	c	kPa	15	5	
Angle of Internal Friction	ϕ	degree	22.5	2.5	
Poisson's Ratio	ν	-	0.4		

estimated values

Rock Mass Type: C 1

Rock Mass Type (RT)	C-1
Lithology	conglomerate

Geological short description	conglomerate to coarse grained sandstone, massive to thick bedded, slightly to moderately fractured, with few siltstone/claystone interbeds possible
Spacing	10 to 50 cm
Surface Shape/Roughness	planar to irregular, slightly rough to rough
Persistence	<3 to >6 m
Opening/Infilling (% of in-filled joints)	generally tight to slightly open (0-1mm), predominantly calcite, few clay (10%) fillings
Weathering	fresh to slightly weathered

	Des.	Dim	Mean Value	Deviation from Mean Value (+/-)	Comment
Parameters of Intact Rock					
Density	γ	KN/m^3	26.0	0.2	15 tests
Uniaxial Compressive Strength	UCS	MPa	49	14.0	13 tests
Hoek-Constant	mi	-	19		
Young's Modulus	E	GPa	59.1	-	1 test
Poisson's Ratio	ν	-	0.21	-	1 test
Parameters of Discontinuities					
Angle of Internal Friction	ϕ	degree	37		
Cohesion	c	MPa	0.10		
Rock Mass Parameters (Hoek-Brown)					
Geological Strength Index	GSI	-	45	5	
Density	γ	KN/m^3	26.0		
Disturbance Factor	D	-	0.5		
Tunnel Depth	-	m	130		
Cohesion	c	MPa	0.58		
Angle of Internal Friction	ϕ	degree	46		
Young's Modulus	E	GPa	3.9		
Young's Modulus (Serafim& Pereira, 1983/Boyd, 1993)					
RMR	–	-	41		
Young's Modulus	E	GPa	6.0		Serafim & Pereira
Rock Mass Parameters (K.W. John)					
Uniaxial Compressive Strength	UCS	MPa	3.4		
Cohesion	c	MPa	0.89		
Angle of Internal Friction	ϕ	degree	35		

Proposed Rock Mass Parameters					
Density	γ	KN/m^3	26	0.5	
Young's Modulus	E	GPa	4	1	
Cohesion	c	MPa	0.8	0.2	
Angle of Internal Friction	ϕ	degree	42.5	2.5	
Uniaxial Compressive Strength	UCS	MPa	3.6	1	
Poisson's Ratio	ν	-	0.25		

	estimated values
	calculated values

Rock Mass Type: C 2

Rock Mass Type (RT)	C-2
Lithology	conglomerate

Geological short description	conglomerate to coarse grained sandstone, intensely to very intensely fractured
Spacing	3 to 10 cm
Surface Shape/Roughness	planar to irregular, slightly rough to rough
Persistence	<3 m
Opening/Infilling (% of in-filled joints)	generally tight to slightly open (0-1mm), predominantly calcite, few clay (10%) fillings
Weathering	moderately to intensely weathered

	Des.	Dim	Mean Value	Deviation from Mean Value (+/-)	Comment
Parameters of Intact Rock					
Density	γ	KN/m^3	26.0	0.2	15 tests
Uniaxial Compressive Strength	UCS	MPa	10		
Hoek-Constant	mi	-	16		
Young's Modulus	E	GPa			
Poisson's Ratio	ν	-			
Parameters of Discontinuities					
Angle of Internal Friction	ϕ	degree			
Cohesion	c	MPa			
Rock Mass Parameters (Hoek-Brown)					
Geological Strength Index	GSI	-	30	5	
Density	γ	KN/m^3	26.0		
Disturbance Factor	D	-	0.25		
Tunnel Depth	-	m	130		
Cohesion	c	MPa	0.26		
Angle of Internal Friction	ϕ	degree	29		
Young's Modulus	E	GPa	0.9		
Young's Modulus (Serafim& Pereira, 1983/Boyd, 1993)					
RMR	–	-	28		
Young's Modulus	E	GPa	0.9		Boyd
Rock Mass Parameters (K.W. John)					
Uniaxial Compressive Strength	UCS	MPa	NA		
Cohesion	c	MPa	NA		
Angle of Internal Friction	ϕ	degree	NA		

Proposed Rock Mass Parameters					
Density	γ	KN/m^3	26	1	
Young's Modulus	E	GPa	0.8	0.2	
Cohesion	c	MPa	0.25	0.05	
Angle of Internal Friction	ϕ	degree	27.5	2.5	
Uniaxial Compressive Strength	UCS	MPa	0.8	0.2	
Poisson's Ratio	ν	-	0.3		

	estimated values
	calculated values

UCS - Granitic rock

Boring	Depth	UCS
	m	Mpa
02-05	213,36	5,1
02-13	10,15	1,2
02-14	37,55	0,6
P9-96	118,26	0,3
	121,01	0,5
P14-96	100,28	4,0
02-05	204,83	24,3
	208,51	16,6
02-21	142,80	10,0
P1-96	10,36	21,8
P2-96	10,06	25,8
P14-96	59,13	18,9
	62,79	27,7
	109,42	14,9
	112,17	17,7
P1-96	17,68	75,1
	23,47	77,0
	43,28	84,6
	43,59	75,8
mean		26,4
mean deviation		21,9

PLT - Granitic Rock

Boring	Depth	UCS	Is(50)
	m	Mpa	Mpa
P9-96	127,53	4,5	0,19
	155,23	4,5	0,19
P14-96	55,84	9,1	0,38
	59,25	9,1	0,38
	62,24	9,1	0,38
	82,63	6,8	0,28
	87,42	1,7	0,07
	88,85	2,3	0,1
	96,87	1,7	0,07
	99,94	1,7	0,07
	112,62	1,7	0,07
02-05	245,49	20,2	0,84
02_2B	47,73	24,2	1,01
	76,05	58,3	2,43
P1-96	17,37	56,9	2,37
	23,84	59,1	2,46
	35,36	56,9	2,37
	43,83	59,1	2,46
	49,62	34,1	1,42
	60,56	27,3	1,14
P2-96	10,67	22,7	0,94
P9-96	125,06	11,4	0,48
	156,82	18,2	0,76
	170,90	11,4	0,48
P14-96	55,14	13,6	0,57
	57,73	20,5	0,85
	58,13	34,1	1,42
	58,67	50,0	2,08
	60,11	27,3	1,14
	60,69	56,9	2,37
	61,66	31,8	1,32
	63,73	20,5	0,85
	66,45	13,6	0,57
	73,76	24,0	1
	79,00	20,5	0,85
	112,75	13,6	0,57
P1-96	19,96	91,0	3,79
	43,04	72,8	3,03
	43,04	95,8	3,99
02-2B	78,88	68,6	2,86
02-2A	48,55	91,4	3,81
	57,85	68,6	2,86
02-21	112,47	64,8	2,7

Boring	Depth	UCS	Is(50)
	m	Mpa	Mpa
02-21	111,34	226,3	9,43
mean		36,4	1,5
mean deviation		23,3	1,2

Table 1: Results of UCS and PLT tests

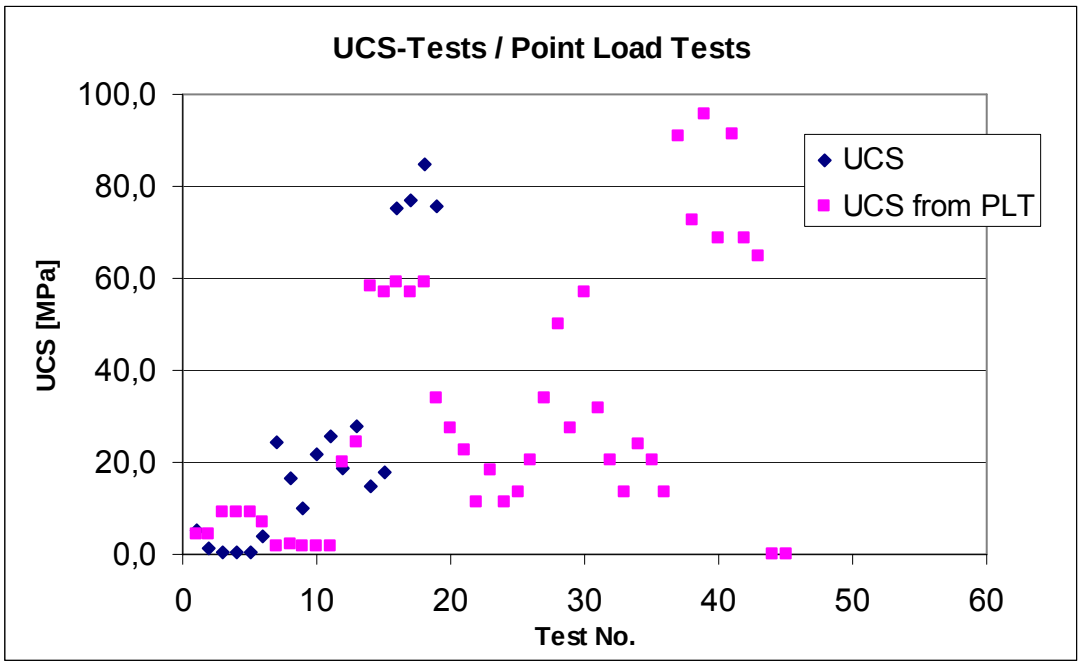


Figure 1: Results of UCS and PLT tests

UCS - Sandstone

Boring	Depth	UCS
	m	Mpa
P5-96	185,17	139,8
	194,46	43,7
P12-96	63,09	93,4
	64,92	189,8
	114,00	163,2
	121,31	56,8
	156,67	69,4
02-05	77,63	66,8
	93,27	150,9
	102,93	155,6
02-06	117,65	164,2
	118,41	117,5
	119,33	194,7
	122,40	131,4
	125,33	152,2
	136,06	160,3
02-06A	161,60	123,8
	162,43	146,0
02-07	111,62	106,1
	112,78	123,5
	122,68	168,3
02-11	81,02	163,1
mean		130,9
mean deviation		34,3

PLT - Sandstone

Boring	Depth	UCS	Is(50)
	m	Mpa	Mpa
P5-96	144,78	56,9	2,4
	145,02	52,1	2,2
	145,97	27,3	1,1
	184,62	81,9	3,4
P6-96	1,43	13,6	0,6
	1,58	20,5	0,9
	2,99	18,2	0,8
P8-96	29,90	22,7	0,9
	69,80	29,6	1,2
P12-96	61,75	11,4	0,5
	64,62	47,8	2,0
	66,39	75,1	3,1
	69,25	70,5	2,9
	72,48	66,0	2,8
	75,80	45,5	1,9
	91,26	45,5	1,9
	95,52	20,5	0,9
	99,73	72,8	3,0
	103,02	61,4	2,6
	106,07	63,7	2,7
	109,76	75,1	3,1
	112,87	66,0	2,8
	113,63	54,6	2,3
	115,31	59,1	2,5
	121,52	84,2	3,5
	137,01	72,8	3,0
	139,39	45,5	1,9
	142,52	45,5	1,9
	156,79	63,7	2,7
	157,34	54,6	2,3
	158,22	72,8	3,0
	158,62	56,9	2,4
	158,98	36,4	1,5
	182,30	59,1	2,5
P13-96	82,84	63,7	2,7
	85,22	66,0	2,8
	88,15	52,0	2,2
	131,77	25,9	1,1
	142,19	46,8	2,0
	153,01	89,0	3,7
	113,23	21,1	0,9
05-05	162,85	35,5	1,5
	63,00	23,5	1,0

Boring	Depth	UCS	Is(50)
	m	Mpa	Mpa
05-05	104,39	72,7	3,0
	106,31	51,8	2,2
	121,86	46,1	1,9
02-12	41,51	234,5	9,8
02-06	91,32	168,0	7,0
	174,07	192,0	8,0
	116,83	105,1	4,4
02-06A	53,86	56,6	2,4
02-07	58,22	123,6	5,2
	57,30	122,9	5,1
02-18B	10,33	72,7	3,0
02-06A	210,16	134,2	5,6
	215,80	103,4	4,3
02-06	134,90	177,8	7,4
	130,82	214,3	8,9
	126,08	149,5	6,2
	130,09	210,2	8,8
	128,14	266,9	11,1
	135,18	186,0	7,8
	135,39	194,2	8,1
	125,32	204,2	8,5
02-06A	159,96	69,6	2,9
	163,62	108,2	4,5
	170,08	56,6	2,4
	173,89	44,4	1,9
	182,67	154,6	6,4
	188,21	143,0	6,0
	196,50	215,0	9,0
02-05	57,91	80,9	3,4
02-11A	123,35	283,0	11,8
	121,95	234,5	9,8
02-08B	91,41	40,3	1,7
02-12	46,15	92,9	3,9
02-06	49,29	101,0	4,2
	47,76	157,7	6,6
	82,60	133,4	5,6
	84,34	113,3	4,7
02-11	80,59	125,3	5,2
02-08C	38,40	60,7	2,5
02-11	123,05	24,2	1,0
mean		90,3	3,8
mean deviation		51,1	2,1

Table 2: Results of UCS and PLT tests

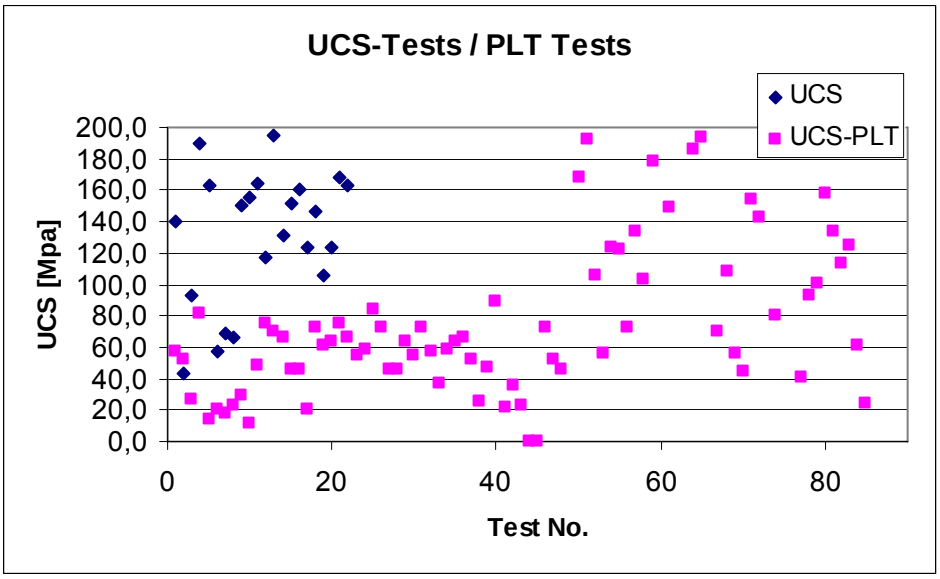


Figure 2: Results of UCS and PLT tests

UCS - Interbedding SS/S/CL

Boring	Depth	UCS
	m	Mpa
P5-96	196,29	5,5
	209,09	18,3
	209,70	11,8
P12-96	159,41	114,7
	176,78	30,3
P13-96	126,80	28,6
	163,68	20,4
	174,35	75,8
	181,36	29,2
02-06	106,50	26,4
02-11	38,74	18,0
	39,93	38,9
	41,76	27,7
	65,44	26,6
	73,61	30,1
	74,58	23,9
	75,29	21,3
	79,74	120,4
	81,02	163,1
	83,36	36,1
	84,58	15,0
02-11A	91,93	74,2
	93,36	72,3
	161,12	49,0
	168,25	61,0
	172,06	57,1
mean		46,0
mean deviation		28,7

PLT - Interbedding SS/S/CL

Boring	Depth	UCS	Is(50)
	m	Mpa	Mpa
P13-96	126,19	6,8	0,3
	161,94	43,2	1,8
02-11	75,04	43,7	1,8
	134,66	67,9	2,8
02-05	171,08	75,1	3,1
	171,85	64,8	2,7
	189,52	67,7	2,8
02-08C	28,80	34,1	1,4
	28,04	38,4	1,6
02-11	28,04	28,3	1,2
02-05	102,41	237,8	9,9
02-08B	80,99	94,6	3,9
02-08C	25,76	4,1	0,2
02-11	38,95	129,4	5,4
	39,75	61,4	2,6
	40,33	51,8	2,2
	68,21	4,1	0,2
	86,32	52,6	2,2
	90,34	95,0	4,0
02-08C	59,38	24,2	1,0
	64,34	48,5	2,0
02-11	85,25	40,3	1,7
	49,77	28,3	1,2
mean		58,4	2,4
mean deviation		32,1	1,3

Table 3: Results of UCS and PLT tests

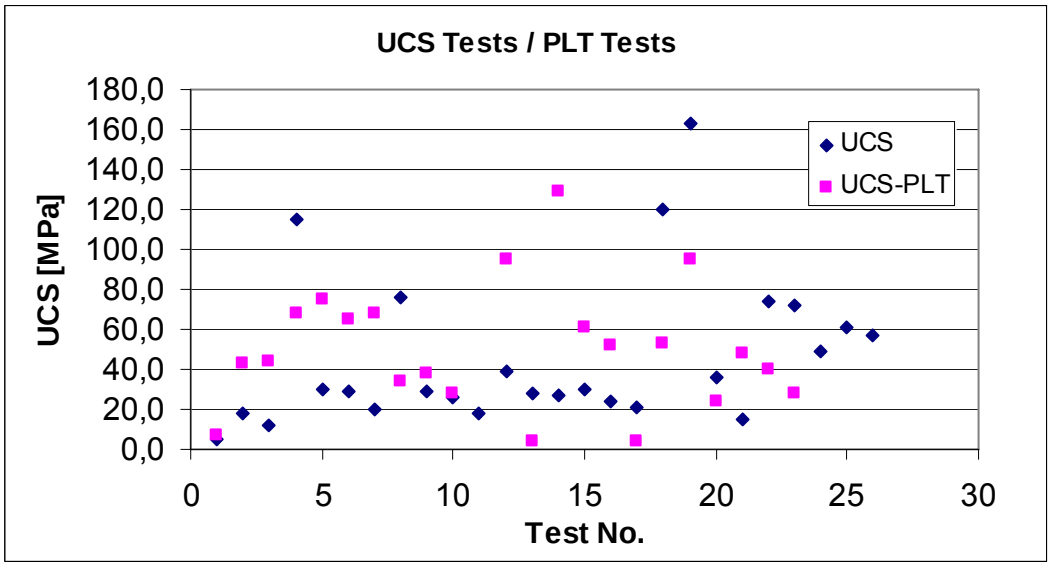


Figure 3: Results of UCS and PLT tests

UCS - Conglomerate

Boring	Depth	UCS
	m	Mpa
P5-96	128,63	49,8
	148,13	44,0
	150,88	43,4
P12-96	135,03	24,8
P13-96	136,25	55,2
02-05	104,52	
	108,57	26,2
02-06	100,58	46,0
	102,44	68,3
	104,73	82,2
02-08B	91,41	58,2
	93,73	54,2
	95,98	67,6
	99,91	19,8
mean		49,2
mean deviation		14,0

PLT - Conglomerate

Boring	Depth	UCS	Is(50)
	m	Mpa	Mpa
P5-96	125,18	45,5	1,9
	128,96	4,5	0,2
	129,17	11,4	0,5
	130,73	4,5	0,2
	131,46	4,5	0,2
	147,65	4,5	0,2
	148,47	22,7	0,9
	149,75	31,8	1,3
	149,87	4,5	0,2
	150,78	4,5	0,2
	151,24	31,8	1,3
	151,70	9,1	0,4
	152,92	4,5	0,2
	153,16	27,3	1,1
P6-96	23,10	50,0	2,1
	24,32	54,6	2,3
P9-96	98,39	4,5	0,2
P12-96	134,90	27,3	1,1
P13-96	99,49	72,8	3,0
	101,96	68,2	2,8
	102,11	41,4	1,7
	109,03	72,8	3,0
	136,06	34,1	1,4
	136,37	61,4	2,6
	138,29	47,8	2,0
	140,03	54,6	2,3
	141,95	59,1	2,5
	144,87	61,4	2,6
	146,70	56,9	2,4
	150,33	40,9	1,7
	153,01	38,7	1,6
	155,20	22,7	0,9
	155,69	11,4	0,5
	159,01	47,8	2,0
	165,57	4,5	0,2
	166,42	50,0	2,1
	169,04	13,6	0,6
02-05	131,77	25,9	1,1
	142,19	46,8	2,0
	153,01	89,0	3,7
	113,23	21,1	0,9
	115,85	3,1	0,1
	128,26	21,1	0,9

Boring	Depth	UCS	Is(50)
	m	Mpa	Mpa
02-05	144,23	4,1	0,2
02-06A	51,21	24,2	1,0
	51,33	30,0	1,3
	95,49	52,6	2,2
	94,34	22,6	0,9
	174,07	36,5	1,5
	178,46	186,0	7,8
	184,71	162,5	6,8
	182,15	59,0	2,5
	209,40	72,0	3,0
02-11A	121,31	239,3	10,0
02-08B	98,45	44,4	1,9
	98,57	16,1	0,7
	93,36	67,9	2,8
	96,53	72,7	3,0
02-12	48,31	44,4	1,9
	50,69	72,7	3,0
mean		43,7	1,8
mean deviation		27,8	1,2

Table 4: Results of UCS and PLT tests

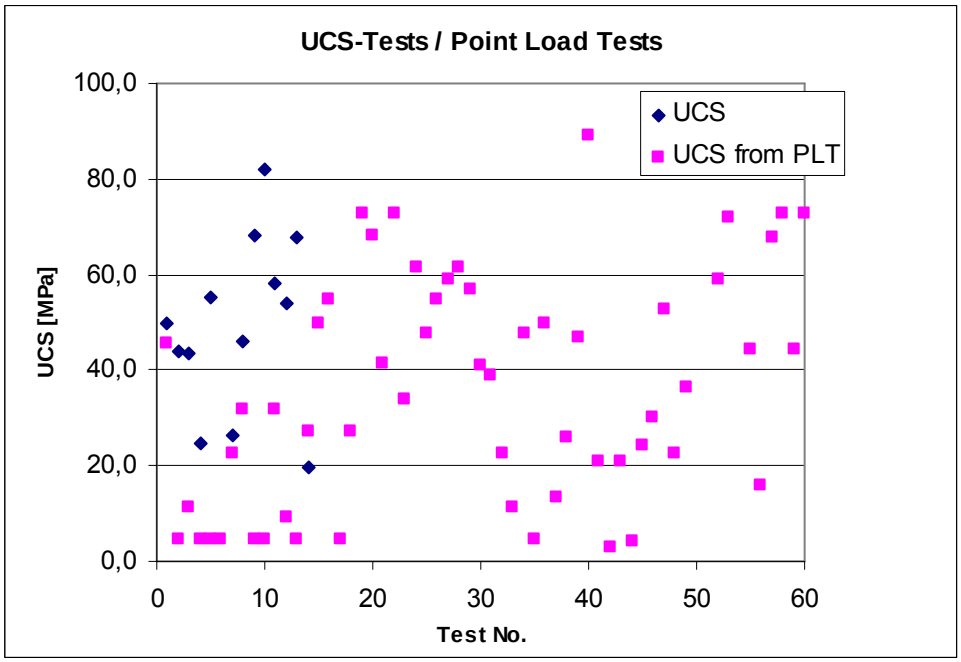


Figure 4: Results of UCS and PLT tests

COMMENTARY

Subject: Shannon & Wilson, Inc.: Review of Rock Mass Parameters for Devil's Slide Tunnels Pacifica, California. July 2, 2004

Comments/Reply:

Page 2: *Shannon & Wilson recommend studying the effects of alternate elastic moduli on analysis results.*

ILF reply: ILF purposely used the most realistic rock parameters for each rock mass type to define which support category is most appropriate.
Including a wide range of parameters results in different support categories, which would sacrifice the prognosis.

Page 5: *Shannon & Wilson make the point that the construction method can effect assumed values of rock shear strength and this should be investigated.*

ILF reply: The disturbance factor is just one out of several parameters to determine strength of the rock mass. This factor is applied in accordance to in-house experience. The construction method to be used, as well as blasting regulations, are included in the specifications. In order not to rely only on one method of determination of the rock mass parameters, two additional and different methods have been used, which confirm the ranges of parameters defined.

Page 6: *Shannon & Wilson question the use of the Mohr-Coulomb model where normal stresses are low.*

LIF reply: Averaging the rock mass shear strength is used as it is in the case for FE calculations, when parameters are not differentiated between near the opening or within the primary stress field. This simplification is generally accepted.

Page 7: *Shannon & Wilson question the use of Poisson's ratio without considering topographical and tectonic effects when determining insitu horizontal rock stresses.*

ILF reply: It is noted that topographical and geologic effects have been considered by calculating the vertical stress field along the tunnel route. A 3D model could not be applied due to lacking geologic features perpendicular to the tunnel route.
The Poisson's ratio was used to calculate the horizontal stress factor for sections with an overall elastic rock mass behaviour only.
According to Shannon & Wilson tectonic effects are not decisive.

Part 2: Rock Mass Behavior

5 VIRGIN STESS FIELD

5.1 Overview

In order to gain information on the virgin stress field the world stress map was reviewed. However few data are available in the vicinity of the project area. Those data which are available provide general information on the orientation of the stress field which shows clear evidence of the connection to the San Andreas Fault system which passes the project area a short distance to the east (see Figure 2). The available data give no information concerning the size of the virgin stress field or on the exact 3D orientation of the main stress axis. Due to the morphological conditions (stress relief caused by the nearby sea cliff) and the properties of the rock mass encountered in the project area (predominantly sheared to heavily sheared, weak rock mass, which are not likely to preserve high stress conditions) it was decided not to assume decisive tectonic stress conditions for the project area. It was assumed that the virgin stress field is governed by the topography and the geological structure.

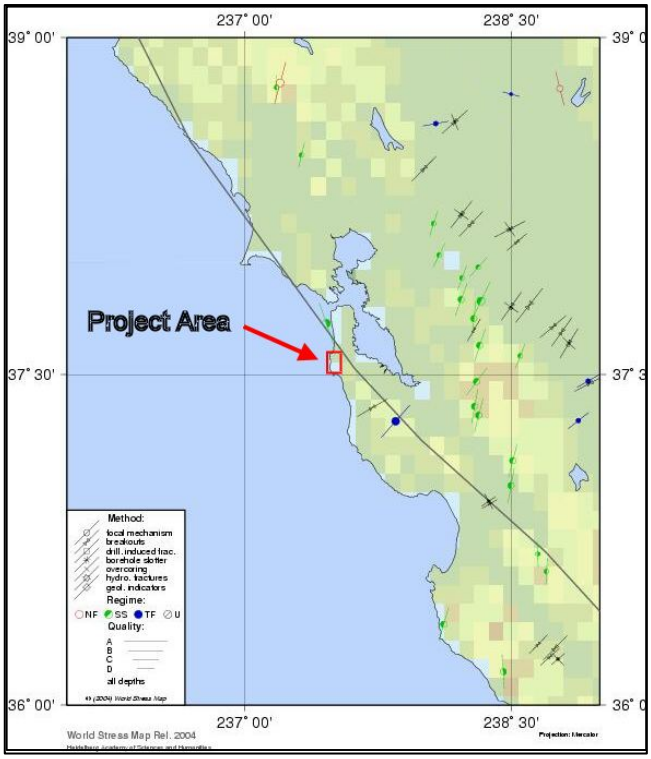


Figure 2: Section of the World Stress Map, from [20]

5.2 Vertical Stresses

The vertical stresses are determined by FE-analysis in order to consider the effect of the geological structure. In the TSR it was already reported that due to the different stiffnesses of the various geologic units and the structural architecture, stress distribution is not only controlled by the topography. Although this may be a three dimensional problem a 2D analysis was carried out, because the various geological units cross the tunnel alignment almost perpendicular. In addition not sufficient input data regarding geologic conditions perpendicular to the tunnel alignment are available. Plain strain conditions have been assumed.

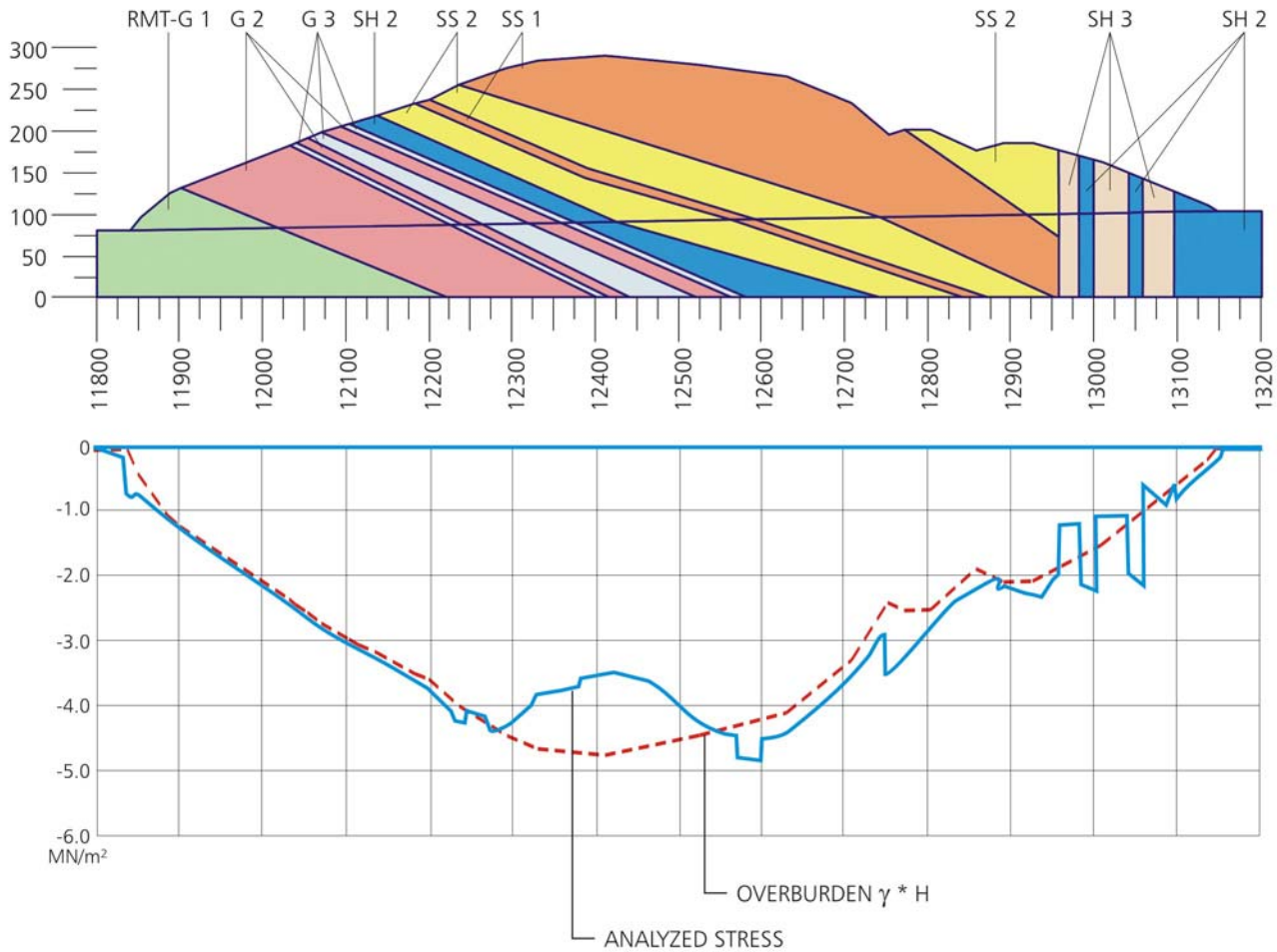


Figure 3: Comparison of overburden and vertical stresses along tunnel

Remark: The rock mass types shown in the above figure refer to the defined rock mass types. Rock mass types with similar rock mass parameters were combined where possible for the sake of con-

venience: SS 1 can also be C1 and SS 2 can also be SH 1 or C 2. If the tunnel section for one rock mass type was very short is was neglected in this calculation for the small impact on the result.

As a result it has been found that due to the large variety of stiffnesses caused by alternating sequences of soft and hard rock formations and their inclination of about 25 to 50 degree in the Central Block, for some tunnel sections the vertical stress state will not be governed by the height of overburden and the specific weight of the rock mass (see Figure 3). Regarding the 3D conditions may increase this effect due to stress relief in direction of the sea cliff.

5.3 Horizontal Stresses

Due to the topographical stress relief low horizontal stresses are expected. The horizontal virgin stress conditions (σ_3) are derived from the k_0 value (horizontal to vertical stress ratio).

For rock mass types which behave elastic, i.e. massive rock units, k_0 is derived from the Poisson's ratio (ν):

$$k_0 = \frac{\nu}{1-\nu}$$

ν	k_o
0,15	0,18
0,25	0,33
0,35	0,54
0,45	0,82

For rock mass types which are fractured but the angle of internal friction is larger than the shear stresses developed by overburden, k_0 is derived from the angle of internal friction:

$$k_0 = \frac{1+\sin\phi}{1-\sin\phi}$$

ϕ	k_o
15°	0,18
25°	0,33
35°	0,54
45°	0,82

For soil the angle of internal friction only is decisive:

$$k_0 = 1 - \sin\phi$$

ϕ	k_o
15°	0,74
25°	0,56
35°	0,43

6 ROCK MASS BEHAVIOR (BHT)

6.1 Approach

The stress ratio SR is introduced in order to determine the rock mass behavior around the tunnel opening. The stress ratio is defined by of the rock mass strength divided by the virgin vertical stresses:

$$SR = \frac{UCS_{RM}}{\sigma_I}$$

UCS_{RM}… Uniaxial compressive strength of rock mass

σ_1… Virgin vertical stress

The stress ratio is decisive with respect to fracturing around the opening. The classification of the stress ratio was chosen according to [19]:

- SR = >0,45 Behavior Type 1: Failure of rock blocks (few support problems)
- SR = 0.45 – 0,28 Behavior Type 2: Fracturing induced by stresses and/or discontinuities (minor squeezing problems)
- SR < 0.28 Behavior Type 3: Progressive failure induced by stresses (severe squeezing problems)
- Behavior Type 4: Failure induced ahead of the tunnel face, this BHT can occur under every stress condition in very weak to soil like rock mass below groundwater level

The fourth rock mass behavior type occurs in highly sheared, soil like siltstone/claystone. There the rock mass behavior is governed by the groundwater conditions and not on the virgin stress conditions. Therefore the above classification system is not applied.
The Figure 3 shows the relation between stress ratio and strain.

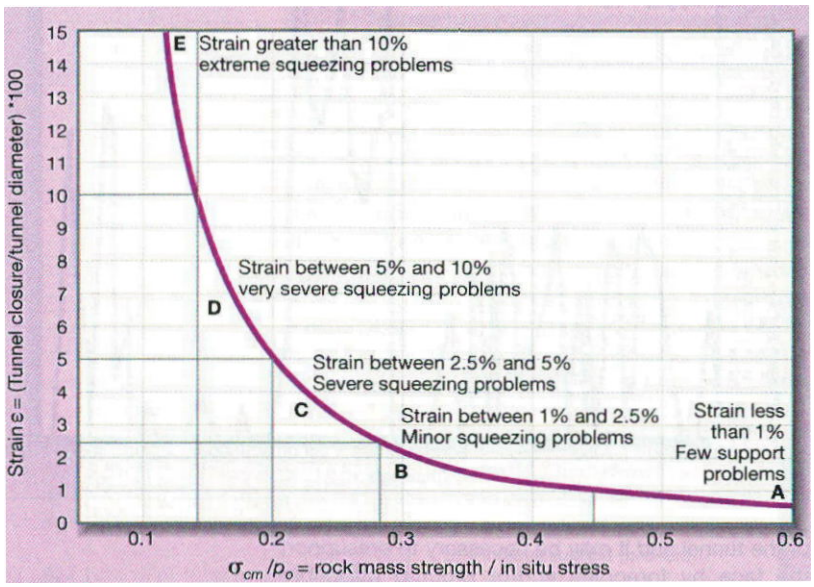


Figure 4: Stress Ratio versus Strain, from [19]

The vertical virgin stress field is determined by FE-analysis (see chap.5.2). Based on these vertical stresses and the UCS_{RM} for the various rock mass types, the stress ratios and the behavior types are allocated to the tunnel alignment. The following tables show the range of the vertical stress for the different rock behavior types, determined for the individual rock mass types and the allocation along the tunnel alignment.

RT	UCS (RM)	Range	BHT 1 (0.45<SR)	BHT 2 (0.28<SR<0.45)	BHT 3 (SR<0.28)
	[MPa]	[MPa]	Range of Sigma 1 [Mpa]		
G 1	2.0	0.5	<4.44	4.44-7.14	>7.14
G 2	0.7	0.2	<1.56	1.56-2.50	>2.50
G 3	0.3	0.1	<0.67	0.67-1.07	>1.07
SS 1	3.9	1.0	<8.67	8.67-13.93	>13.93
SS 2	0.8	0.2	<1.78	1.78-2.86	>2.86
SH 1	1.0	0.2	<2.22	2.22-3.57	>3.57
SH 2	0.6	0.2	<1.33	1.33-2.14	>2.14
C 1	3.6	1.0	<8.00	8.00-12.86	>12.86
C 2	0.8	0.2	<1.78	1.78-2.86	>2.86

Table 6: Stress Ranges for the Rock Mass Behavior Types of the individual Rock Mass Types (values in the column UCS (RM) are the mean values, values in the Range column are the ± values)

The behavior types are used to define sections of similar rock mass behavior during excavation. On the basis of the distribution of behavior types representative sections are chosen for which numerical analysis are carried out. By means of these calculations support categories are defined for those sections investigated. In turn these support categories are applicable at the relevant behavior types, refer to Figure 1 in the Geotechnical Design – Part 1.

Station SB		Station NB		RT	Sigma 1 [MPa]		UCS (RM)	Stress Ratio		BHT
from	to	from	to		lower bound	upper bound	[MPa]	lower bound	upper bound	
South Block										
118+51	119+46	118+44	119+34	G2	0.8	1.7	0.7	0.41	0.88	2[1]
119+46	120+25	119+34	120+12	G1	1.7	2.3	2.0	0.87	1.18	1
120+25	120+45	120+12	120+32	G2, G3	2.3	2.5	0.5	0.20	0.22	3
120+45	120+96	120+32	120+84	G1	2.5	2.9	2.0	0.69	0.80	1
120+96	122+12	120+84	122+00	G1, G2	2.9	3.4	1.3	0.38	0.45	2
122+12	123+68	122+00	123+56	G2	4.1	4.4	0.7	0.16	0.17	3
123+68	123+88	123+56	123+76	G2, G3	3.7	3.8	0.5	0.13	0.14	3
Central Block										
123+88	124+14	123+76	124+10	SH 2	3.5	3.7	0.6	0.16	0.17	3
124+14	124+37	124+10	124+33	SH 2, SH 3	3.5	3.5	0.3	0.09	0.09	3/4
124+37	124+94	124+33	124+72	SS 2, SH 2	3.5	3.8	0.7	0.18	0.20	3
124+94	125+86	124+72	125+61	C2[2]	3.8	4.5	1.0	0.22	0.26	2/3
125+86	126+07	125+61	125+85	C1	4.5	4.9	3.6	0.73	0.80	1
126+07	127+20	125+85	126+90	SS 2[2]	3.6	4.6	1.0	0.22	0.28	2/3
127+20	127+76	126+90	127+34	SS 1, SH 1	2.9	3.6	2.5	0.68	0.84	1/2
127+76	128+90	127+34	128+48	SS 1	2.1	3.3	3.9	1.18	1.86	1/2
128+90	129+28	128+48	128+86	SS 1, C 1	2.0	2.3	3.8	1.63	1.88	1/2
129+28	129+58	128+86	129+46	SS 2, C 2	2.0	2.4	0.8	0.33	0.40	2
North Block										
129+58	129+78	129+46	129+66	SH 3	1.9	2.0	0.1	0.05	0.05	4
129+78	130+54	129+66	130+42	SH 2, SH 3	1.2	1.6	0.3			3[1]
130+54	130+78	130+42	130+63	SH 2	1.0	1.2	0.6			3[1]
130+78	130+95	130+63	130+95	SH 2, SH 3	0.7	0.9	0.3			3[1]
		130+95	131+16	SS 2	0.4	0.7	0.8			3[1]

[1] stress factor was not used for definition of the rock mass behavior due to the special situation (shallow overburden, intensive weathering of rock mass close to surface)
[2] the upper bound of the UCS of rock mass was used

Table 7: Stress Ratio and Rock Mass Behavior Types along the Tunnel Alignment

Appendix 1 shows the geological/geotechnical profiles of the two tunnel tubes including the distribution of rock mass types, rock mass behavior types and support categories.

6.2 Water Conditions

Groundwater can have a major impact on tunnel construction especially in weak and highly sheared rock masses which are prone to erosion processes. As the groundwater level is above the tunnel opening throughout the tunnel alignment, it is planned to drain the tunnel ahead of the face in the critical sections by means of drainage borings. Hydrogeological modeling will be carried out in order to more accurately define the critical tunnel sections, to estimate the amount of water to be expected during tunnels construction and to estimate the impact of tunneling on the local groundwater regime.

6.3 Orientation of Opening

The orientation of the opening relative to the major discontinuity sets governs the stress relevant for the tunnel design. It also has a major impact on size, shape and stability of rock wedges formed by the intersection of discontinuities and the tunnel opening. Therefore the orientation of the opening is considered in the block stability analysis as well as the FE-analysis.

6.4 Dimension and Form of Opening

The distribution of stress around the tunnel opening is governed to a large extent by the size and cross section of the opening. They also affect size and shape of potentially unstable blocks during tunnel excavation. Therefore the size and cross section of the opening are considered in the block stability analysis as well as in the FE-analysis.

6.5 Rock Mass Behavior Types

The relevant behavior types identified along the tunnel alignment are characterized as follows:

6.5.1 Behavior Type 1: Failure of rock blocks

Rock mass behavior: The occurrence of fractures is mainly induced by discontinuities resulting in falling out of individual blocks.

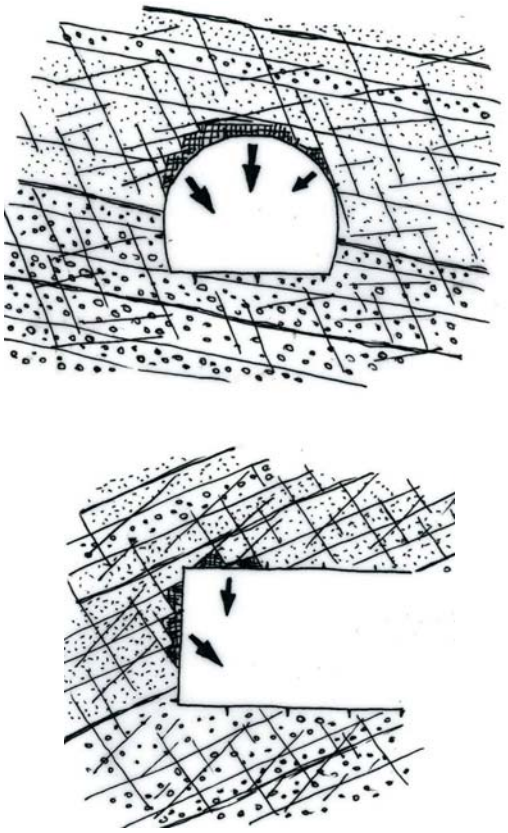
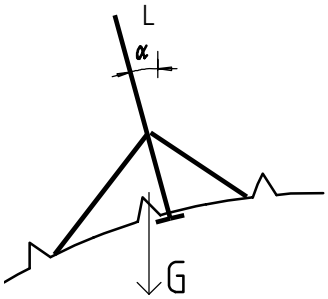
Failure Mode	Model of Analysis
	Block theory $L = G \cdot \eta / \cos(\alpha)$  L... load bearing capacity η… safety factor α… angle of bolting

Figure 5: Description of Behavior Type 1: Failure of rock blocks

6.5.2 Behavior Type 2: Fracturing induced by stresses and/or discontinuities

Rock mass behavior: The rock mass behavior is characterized by shear failure of the rock mass near the tunnel walls and along discontinuities. The rock mass is loosened due to fracturing. Mainly, vertical loads are acting on the lining (loosening pressure).

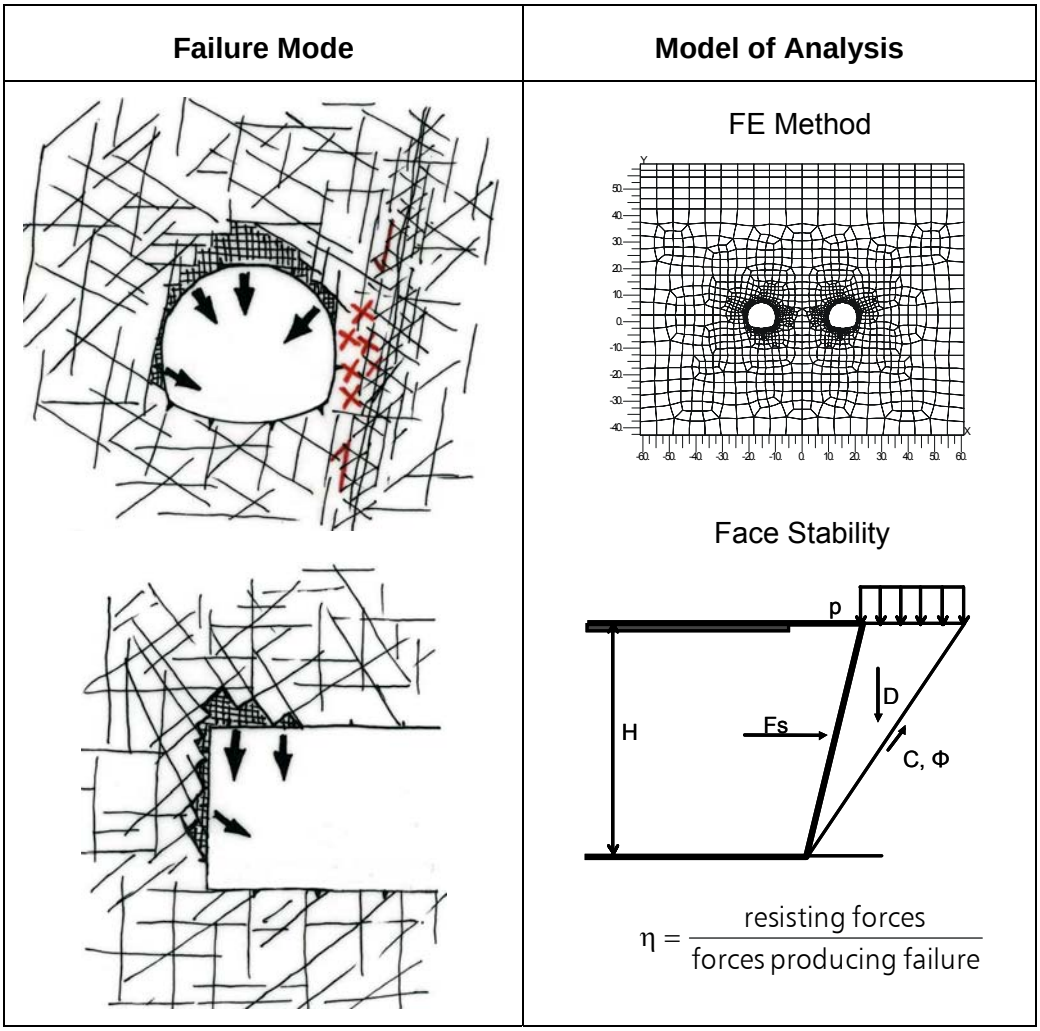


Figure 6: Description of Behavior Type 2: Fracturing induced by stresses and/or discontinuities

6.5.3 Behavior Type 3: Progressive failure induced by stresses

Rock mass behavior: Because the load bearing capacity of the rock mass is exceeded, the rock mass undergoes progressive shear failures, resulting in large plastic deformations.

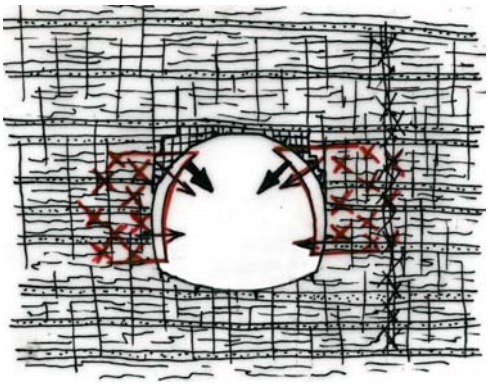
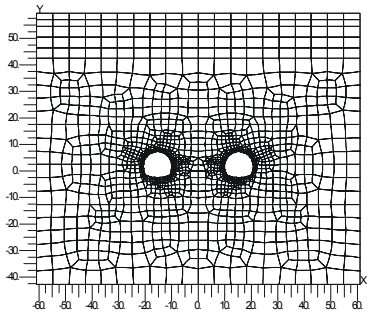
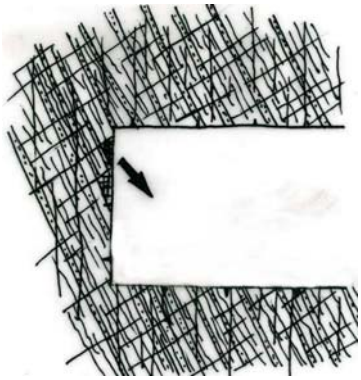
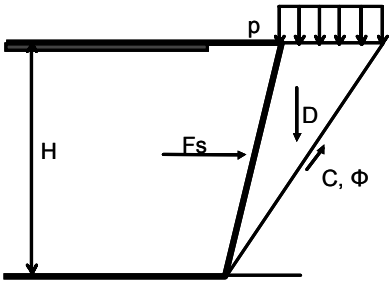
Failure Mode	Model of Analysis
	FE Method 
	Face Stability  $\eta = \frac{\text{resisting forces}}{\text{forces producing failure}}$

Figure 7: Description of Behavior Type 3: Progressive failure induced by stresses

6.5.4 Behavior Type 4: Failure induced ahead of the tunnel face

Rock mass behavior: Behavior type 4 is characterized by weak, incompetent rock mass, below groundwater level. Groundwater can cause ingress of water and rock mass, resulting in rock mass piping and failure.

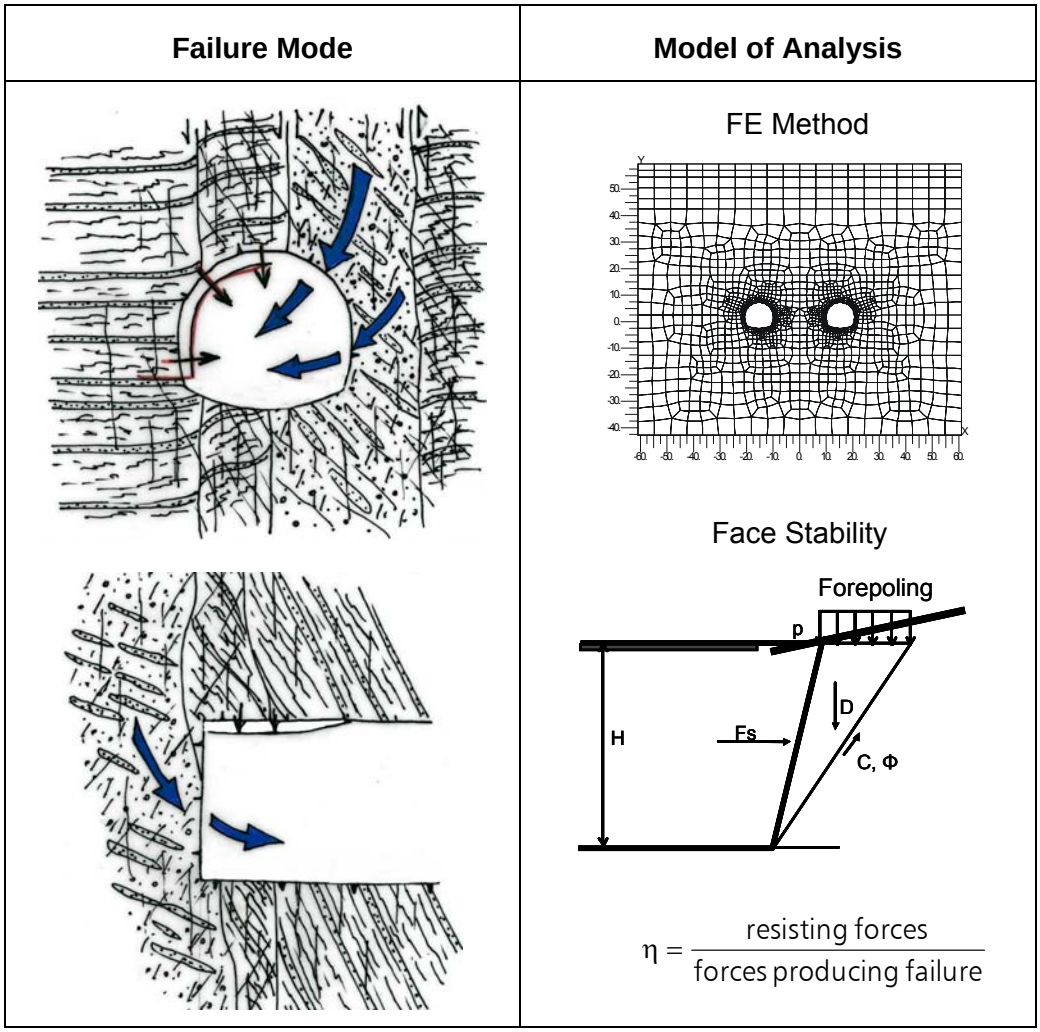


Figure 8: Description of Behavior Type 4: Failure induced ahead of the tunnel face

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Appendix 1+2: Geotechnical Profiles with Layout of Support Measures

1a Geotechnical Profile of South Bound

1b Geotechnical Profile of North Bound

Appendix 3: Review of Technical Memorandum DST / Geotechnical Design Part 2