1) Theoretical basis.

1.1 Stereographic projection of rock discontinuities.

The planar discontinuities in the rock mass (bedding planes, joints and faults) can be graphically represented and analyzed through hemispherical projection methods, whereby three-dimensional oriented planes can be drawn in two dimensions. Orientations of the rock planes can be recorded using a couple of numbers, dip direction and angle of dip. They represent the orientation, expressed as trend and plunge, of the line of maximum dip of an inclined plane. Plunge is the acute angle measured in a vertical plane between the given line and the horizontal; it varies from -90° to +90°. Trend is the azimuth, measured in clockwise sense, from the north of the vertical plane containing the line of given plunge; it varies from 0° to 360°. Plane orientations can usually be displayed through polar or equatorial projections. Among the most used projections, there are the polar equal-area nets (Schmidt's projection) and the equatorial equal-angle nets (Wulff's projection).
- Schmidt's projection

It's used to represent the rock plane through its pole, that's the normal of the plane itself. This projection preserves the ratio between the areas. Thus it's useful to process statistically the orientation of the rock planes and identify the main discontinuities sets and their mean orientation.
• Wulff's projection.

It can be used to represent the mean orientation of the main sets of rock planes. It preserves the ratio between the angles and, consequently, it's useful to display and analyze potential instabilities of rock wedges.

Equatorial equal-angle projection (a plane with orientation 345°/45° and its pole are displayed)
In case of linear survey, a sampling bias has to be taken in account as a function of the acute angle between the sampling line and the normal to the measured discontinuity planes. In fact the number of measured discontinuities reaches the maximum value when the sampling line is perpendicularly to the planes and becomes null when the sampling line is parallel. Consequently a weighting factor must be applied to the number of discontinuities of each discontinuity set. This factor \( w \) can be calculated by the following formula:

\[
w = 1 / |\cos(\alpha_n - \alpha_s)\cos\beta_n \cos\beta_s + \sin\beta_n \sin\beta_s| 
\]

where:
\[
\alpha_n = \text{trend of the normal to the discontinuity}; \\
\alpha_s = \text{trend of the sampling line}; \\
\beta_n = \text{plunge of the normal to the discontinuity}; \\
\beta_s = \text{plunge of the sampling line}.
\]

The correct number of discontinuities \( N_c \) is given by:

\[
N_c = w N_m
\]

where \( N_m \) is the number of measured discontinuities.

### 1.2 Spacing and frequency of rock discontinuities.

Spacing is the average distance between two adjacent rock discontinuities belonging to the same joint set, measured perpendicularly to the discontinuities themselves. Frequency is the reciprocal of spacing. The mean spacing is given by:

\[
S (\text{m}) = L / N
\]

where \( L \) is the length of the sampling line and \( N \) is the number of measured discontinuity intersections. Consequently the discontinuity frequency is:

\[
\lambda (\text{m}^{-1}) = 1 / S
\]
The spacing distribution is normally associated to a negative exponential distribution, which has a probability density function given by following expression:

\[ f(S) = \lambda e^{-\lambda S} \]

where the mean value and the standard deviation are both equal to \(1/\lambda\).

Mean values of the joint set spacings can be used to calculate the volumetric joint count (\(J_v\)), which is a measure for the number of joints intersecting a volume of rock mass. \(J_v\) can be estimated through the following formula:

\[ J_v = \frac{1}{S_1} + \frac{1}{S_2} + \frac{1}{S_3} + \ldots + \frac{1}{S_n} \]

where \(S_1, S_2, S_3, \ldots, S_n\) are the mean spacings of the discontinuity sets.

The volumetric joint count (\(J_v\)) is correlatable to the rock block volume through the following expression:

\[ V_b = \beta \times J_v^3 \frac{1}{\sin \gamma_1 \times \sin \gamma_2 \times \sin \gamma_3} \]

where \(\gamma_1, \gamma_1, \text{ and } \gamma_3\) are the intersecting angles of the main three joint sets.

The parameter \(\beta\) is the block shape factor, given as:

\[ \beta = \frac{(\alpha_2 + \alpha_2 \times \alpha_3 + \alpha_3)^3}{(\alpha_2 \times \alpha_3)^3} \]

with \(\alpha_2=S_2/S_1\) and \(\alpha_3=S_3/S_1\) (\(S_3>S_2>S_1\)).

Based on these two variables, it's possible to classify the shape of the blocks (Palmstrom, 1985):
1.3 Roughness of rock discontinuities.

The term 'roughness' indicates the non-regularity of the discontinuity surface, that's the deviation from the perfect planarity. From a practical point of view, roughness can be quantified using the Joint Roughness Coefficient (J.R.C.) parameter (Barton and Choubey, 1977). Roughness can be measured through the Shape Tracer and J.R.C. can be estimated using the Barton's J.R.C. Profiles.
J.R.C. varies from 0 (planar surface) to 20 (extremely irregular plane). J.R.C. can be theoretically calculated by the following expression:

$$J.R.C. = 32.2 + 32.47 \log_{10} Z$$

where $Z$ is given by:

$$Z = \sqrt{\frac{1}{n(dx)^2} \sum_{i=1}^{n} (y_{i+1} - y_i)}$$

with:

- $n$ = Number of scanning steps;
- $dx$ = Width of single step;
- $y$ = Height of the profile from the mean line.
As an alternative, J.R.C. can be gotten through a laboratory test (tilt test) suggested by Barton and Choubey, 1977. J.R.C. is given by the following formula:

$$J.R.C. = \frac{(\alpha - \varphi_r)}{\log_{10}\left(\frac{J.C.S.}{\sigma_n}\right)}$$

where:

- $\alpha$ (°) = angle of initial sliding
- $\varphi_r$ (°) = residual friction angle
- $\sigma_n$ (MPa) = normal lithostatic pressure
- J.C.S. (Mpa) = Joint Compressive Strength (Miller, 1965)

The residual friction angle can be approximately set equal to the angle of sliding of a perfectly planar rock surface. As an alternative, it can be estimated using the following formula:

$$\varphi_r = \varphi_b - 20 + 20\left(\frac{J.C.S_{a}}{J.C.S_{u}}\right)$$

where:

- $\varphi_b$ = basic friction angle of the discontinuity;
- $J.C.S_{u}$ = J.C.S. of unaltered discontinuity;
- $J.C.S_{a}$ = JCS of altered discontinuity.

The basic friction angle of the discontinuity is relative to a smooth and unaltered discontinuity surface as a function of the rock mineralogy and texture. In the following table are displayed some values of $\varphi_b$ for several lithologies.
1.4 Shear strength of rock discontinuities.

The shear strength of discontinuities is estimable using the empirical relation by Barton et al. (1985):

$$\tau = \sigma_n \tan \left[ JRC \log_{10} \left( \frac{JCS}{\sigma} \right) + \varphi_r \right]$$

The parameter J.C.S. (Joint Compressive Strength) can be calculated by measures carried out through a Schmidt's Hammer or, as an alternative, a concrete sclerometer. The instrument gives an index, correlatable to JCS through the following relation:

$$Log_{10} J.C.S.(MPa) = 0.00088 \gamma r + 1.01$$

where:
\( \gamma (kN/mc) = \) Rock unit weight;
\( r = \) Schmidt's Hammer index.
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As an alternative, JCS can be estimated using the following chart, which takes in account also the angle between the hammer and the horizontal.

Measurements must be performed both on unaltered and altered surfaces. The ratio gives an indication about the weathering degree of the joint surface. Using a concrete sclerometer, the hammer index must be corrected to take in account the different characteristics of the instruments. A correlation
between the Schmidt's hammer (ISH) and the concrete sclerometer (ICS) is suggested by Bagalà (1998):

\[ \text{ISH} = \frac{\text{ICS} - 22.1}{0.7} \]

1.5 Rock Quality Designation (R.Q.D.).

R.Q.D. (Deere, 1963) is defined as the percentage of the sampling line consisting of spacing values greater or equal than 10 cm.

\[ \text{R.Q.D.} = \frac{\sum \text{Spacing values} \geq 10 \text{ cm}}{\text{Total length of the sampling line}} \]

For a negative exponential distribution of the discontinuity frequency, R.Q.D. can be estimated through the following relation:

\[ \text{R.Q.D.} = 100(0.1\lambda + 1)e^{-0.1\lambda} \]

Deere suggests the following classification of the rock mass quality:

<table>
<thead>
<tr>
<th>R.Q.D. (%)</th>
<th>Quality</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 - 25</td>
<td>Very poor</td>
</tr>
<tr>
<td>26 - 50</td>
<td>Poor</td>
</tr>
<tr>
<td>51 - 75</td>
<td>Fair</td>
</tr>
<tr>
<td>76 - 90</td>
<td>Good</td>
</tr>
<tr>
<td>91 - 100</td>
<td>Very good</td>
</tr>
</tbody>
</table>

1.6 Uniaxial compressive strength of intact rock (Point Load test).

The uniaxial compressive strength of intact rock specimens can be estimated through Point Load tests. Using irregular shaped or cylindrical specimens, the Is50 parameter can be calculated through the Gremminer's formula:
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\[ Is(50)(\text{MPa}) = \left( \frac{0.138F}{DL} \right)^{0.75} \]

where:
- \( Is(50)(\text{MPa}) \) = Point load index relative to the reference diameter (50 mm);
- \( D(\text{mm}) \) = Distance between the instrument points;
- \( L(\text{mm}) \) = Specimen length along the failure surface;
- \( F(\text{N}) \) = Ultimate failure load.

The uniaxial compressive strength is correlated to \( Is_{50} \) through the following formula:

\[ \sigma_c(\text{MPa}) = 24Is_{50} \]

1.7 Instantaneous angle of shearing strength and cohesion of rock mass and discontinuities.

Hoek and Brown criterion.

The Coulomb criterion

\[ \tau = c + \sigma \tan \varphi; \]

where
- \( c \) = cohesion;
- \( \sigma \) = effective pressure;
- \( \varphi \) = angle of shear strength.

cannot be applied to the rock, where the correlation between shear strength and effective pressure is not linear. However it's possible to estimate instantaneous values of cohesion and angle of shear strength, relative to a specific value of effective pressure, through the empirical Hoek and Brown criterion.
The criterion is expressed as

\[ \sigma_1 = \sigma_3 + s \left[ m_b \frac{\sigma_3}{\sigma_c} + s \right]^a \]

where:
- \( s, a, m_b \) = Constants for a specific rock type;
- \( \sigma_c \) = Uniaxial compressive strength of the intact rock;
- \( \sigma_1, \sigma_3 \) = Major and minor principal stresses.

The \( s, a \) and \( m_b \) rock constants can be correlated to GSI (Geological Strength Index).

Three cases are distinguished based on the GSI value.

- **Undisturbed rock and G.S.I.>25:**
  \[
  m = \frac{m_i e^{\frac{GSI-100}{28}}}{GSI-100} \\
  s = e^{\frac{GSI-100}{9}} \\
  a = 0.5
  \]

- **Undisturbed rock and G.S.I.\leq25:**
  \[
  m = \frac{m_i e^{\frac{GSI-100}{28}}}{GSI-100} \\
  s = 0 \\
  a = 0.65 - \frac{GSI}{200}
  \]

- **Disturbed rock any value of G.S.I.**
  \[
  m_r = m_i e^{\frac{GSI-100}{14}}
  \]
\[ s_r = \frac{GSI - 100}{6} \]

\[ a = 0.5 \]

where:

\( m_i \) = variable depending on the rock mineralogy and petrographic characteristics, derivable from the following table:

<table>
<thead>
<tr>
<th>Rock type</th>
<th>Class</th>
<th>Group</th>
<th>Texture Coarse</th>
<th>Texture Medium</th>
<th>Texture Fine</th>
<th>Texture Very fine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sedimentary</td>
<td>Clastic</td>
<td>Conglomerates*</td>
<td>21 = 3</td>
<td>17 = 4</td>
<td>7 = 2</td>
<td>4 = 2</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Breccias</td>
<td>19 = 5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Non-</td>
<td>Carbonates</td>
<td>Crystalline Limestone</td>
<td>12 = 3</td>
<td>Sparitic Limestones</td>
<td>10 = 2</td>
</tr>
<tr>
<td></td>
<td>Clastic</td>
<td>Evaporites</td>
<td>Gypsum</td>
<td>8 = 2</td>
<td>Anhydrite</td>
<td>12 = 2</td>
</tr>
<tr>
<td></td>
<td>Organic</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Chalk</td>
</tr>
<tr>
<td>Metamorphic</td>
<td>Non-Foliated</td>
<td>Marble</td>
<td>9 = 3</td>
<td>Hornfels</td>
<td>19 = 4</td>
<td>Quartzites</td>
</tr>
<tr>
<td></td>
<td>Slightly Foliated</td>
<td>Migmatite</td>
<td>29 = 3</td>
<td>Amphibolites</td>
<td>26 = 6</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Foliated**</td>
<td>Gneiss</td>
<td>28 = 5</td>
<td>Schists</td>
<td>12 = 3</td>
<td>Phyllites</td>
</tr>
<tr>
<td>Igneous</td>
<td>Light</td>
<td>Granite</td>
<td>32 = 3</td>
<td>Diorite</td>
<td>25 = 5</td>
<td>Granodiorite</td>
</tr>
<tr>
<td></td>
<td>Dark</td>
<td>Gabro</td>
<td>27 = 3</td>
<td>Dolerite</td>
<td>16 = 5</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Nonte</td>
<td>20 = 5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hypabyssal</td>
<td>Porphrynes</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Diabase</td>
<td>Felsite</td>
<td>(15 = 5)</td>
<td>Dacite</td>
<td>(25 = 3)</td>
<td>Obsidian</td>
</tr>
<tr>
<td>Volcanic</td>
<td>Lava</td>
<td>Agglomerate</td>
<td>(19 = 3)</td>
<td>Breccia</td>
<td>(19 = 5)</td>
<td>Tuff</td>
</tr>
</tbody>
</table>
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Instantaneous cohesion \( (c_i) \) and angle of shear strength \( (\varphi_i) \) of the rock mass.

The parameters \( c_i \) and \( \varphi_i \) can be obtained through an implicit numerical technique. The calculation steps are the following:

- Using the Hoek and Brown criterion, \( \sigma_1 \) is calculated, making \( \sigma_3 \) variable from a value close to 0 to a maximum value approximately equal to 0.25 \( \sigma_c \). The incremental step of \( \sigma_3 \) \( (\Delta \sigma_3) \) is given by the ratio \( \Delta \sigma_3 = \sigma_c/10 \). To \( n \) steps \( \Delta \sigma_3 \) correspond \( n \) couple of \( \sigma_1, \sigma_3 \) values, through the Hoek and Brown formula, and \( n \) sets of values \( \delta \sigma_i/\delta \sigma_3, \sigma_n', \tau \), given by the Balmer relations:

\[
\sigma_n = \sigma_3 + \frac{\sigma_1 - \sigma_3}{\delta \sigma_1/\delta \sigma_3} + 1 ;
\]

\[
\tau = \left(\sigma_n - \sigma_3\right) \sqrt{\frac{\delta \sigma_1}{\delta \sigma_3}} ;
\]

\[
\frac{\delta \sigma_1}{\delta \sigma_3} = 1 + \frac{m_1 \sigma_c}{2(\sigma_1 - \sigma_3)} \quad \text{(GSI > 25, } a=0.5) \]

\[
\frac{\delta \sigma_1}{\delta \sigma_3} = 1 + am_1^a \left(\frac{\sigma_3}{\sigma_c}\right)^{a-1} \quad \text{(GSI < 25, } s=0) \]

By the linear regression formula:

\[
\varphi_i' = \arctan \left[ \frac{\sum \sigma_n \tau - \frac{\sum \sigma_n \sum \tau}{n}}{\sum \sigma_n^2 - \frac{(\sum \sigma_n)^2}{n}} \right],
\]
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\[ c'_i = \left( \sum \frac{\tau}{n} \right) - \left[ \left( \sum \frac{\sigma_n}{n} \right) \tan \phi'_i \right], \]

- Inside the calculated intervals of \( \sigma_n \) values (\( \Delta \sigma_n \)), the interval where falls the searched \( \sigma_n' \) value is identified. \( \Delta \sigma_n \) is associated to the intervals of cohesion and shear strength (\( \Delta c_i' \) and \( \Delta \phi_i' \)), whereby:

\[ c_i = \frac{\sigma_{abc}'}{\Delta \sigma_n} \Delta c_i', \]

\[ \phi_i = \frac{\sigma_{abc}'}{\Delta \sigma_n} \Delta \phi_i', \]

**Instantaneous cohesion** (\( c_i \)) and **angle of shear strength** (\( \phi_i \)) of the discontinuities.

The shear strength of the discontinuities, expressed as \( c_i \) and \( \phi_i \) values, can be estimated through the relations suggested by Barton. These the calculation steps:

\[ \tau = \sigma_n' \tan \left[ \varphi_b + JRCLog_{10} \left( \frac{JCS}{\sigma_n'} \right) \right]; \]

\[ \frac{\delta \tau}{\delta \sigma_n} = \tan \left[ \varphi_b + JRCLog_{10} \left( \frac{JCS}{\sigma_n'} \right) \right] - \frac{\pi JRC}{180 \ln 10} \left[ \left( \varphi_b + JRCLog_{10} \left( \frac{JCS}{\sigma_n'} \right) \right) \tan^2 \left[ \varphi_b + JRCLog_{10} \left( \frac{JCS}{\sigma_n'} \right) \right] + 1 \right]; \]

\[ \varphi_i = \arctan \left( \frac{\delta \tau}{\delta \sigma_n} \right); \]

\[ c_i = \tau - \sigma_n \tan \phi_i. \]
1.8 Rock mass classifications.

Rock mass classifications are simplified empirical schemes, which allow to estimate the quality of the rock mass through the calculation of a numerical index, correlatable to the strength characteristics of the rock.


The Bieniawski's rock mass classification takes in account five parameters relative to the condition of the rock mass and a corrective index as a function of the discontinuity orientation and of the considered problem. (tunnel, slope and foundation).

$$RMR = (A1 + A2 + A3 + A4 + A5) - Ic;$$

The five parameters are the following:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>$&lt;\text{Co}&gt;$ (Uniaxial compressive strength);</td>
</tr>
<tr>
<td>A2</td>
<td>$&lt;\text{RQD} %&gt;$ (Rock Quality Designation);</td>
</tr>
<tr>
<td>A3</td>
<td>$&lt;s&gt;$ (Discontinuity spacing);</td>
</tr>
<tr>
<td>A4</td>
<td>Discontinuity condition</td>
</tr>
<tr>
<td>A5</td>
<td>Ground water condition</td>
</tr>
<tr>
<td>Ic</td>
<td>Corrective index</td>
</tr>
</tbody>
</table>

A partial rating is assigned to each parameter and then an overall index is calculated, summing the ratings and applying the corrective index. They exist several version of this classification: 1976, 1979 and 1989.
The sum of the five partial ratings gives the Basic RMR (BRMR). The BRMR in the condition of dry discontinuities corresponds to the parameter GSI (Geological Strength Index):

\[ GSI = BRMR_{76} \text{ (when BRMR}>18) \]

The corrective index Ic is given by the following table:

<table>
<thead>
<tr>
<th>Rating</th>
<th>Inflow per 10m tunnel length (l/min)</th>
<th>Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>&lt; 25</td>
<td>IV</td>
</tr>
<tr>
<td>0-0.2</td>
<td>0.2 - 0.5</td>
<td>III</td>
</tr>
<tr>
<td>&gt;0.5</td>
<td></td>
<td>I</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Orientation of discontinuities</th>
<th>Rating</th>
<th>Very favourable</th>
<th>Favourable</th>
<th>Fair</th>
<th>Unfavourable</th>
<th>Very unfavourable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnels</td>
<td>0</td>
<td>0</td>
<td>-2</td>
<td>-5</td>
<td>-10</td>
<td>-12</td>
</tr>
<tr>
<td>Foundations</td>
<td>0</td>
<td>-2</td>
<td>-2</td>
<td>-7</td>
<td>-15</td>
<td>-25</td>
</tr>
<tr>
<td>Slopes</td>
<td>0</td>
<td>-5</td>
<td>-5</td>
<td>-25</td>
<td>-60</td>
<td>-80</td>
</tr>
</tbody>
</table>

Applying the Ic parameter to the BRMR the RMR index is calculated. The quality of the rock mass is gotten through the following table.

<table>
<thead>
<tr>
<th>RMR</th>
<th>CLASS</th>
<th>QUALITY</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-25</td>
<td>V</td>
<td>Very poor</td>
</tr>
<tr>
<td>25-50</td>
<td>IV</td>
<td>Poor</td>
</tr>
<tr>
<td>50-70</td>
<td>III</td>
<td>Fair</td>
</tr>
<tr>
<td>70-90</td>
<td>II</td>
<td>Good</td>
</tr>
<tr>
<td>90-100</td>
<td>I</td>
<td>Very good</td>
</tr>
</tbody>
</table>
**C.S.I.R. Rock Mass Rating: 1979**

The sum of the five partial ratings gives the Basic RMR (BRMR). The BRMR in the condition of dry discontinuities corresponds to the parameter GSI (Geological Strength Index):

\[ GSI = BRMR_{79} - 5 \] (when BRMR>23)

The corrective index Ic is given by the following table:

<table>
<thead>
<tr>
<th>Orientations of discontinuities</th>
<th>Very favourable</th>
<th>Favourable</th>
<th>Fair</th>
<th>Unfavourable</th>
<th>Very unfavourable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tunnel</td>
<td>0</td>
<td>-2</td>
<td>-5</td>
<td>-10</td>
<td>-12</td>
</tr>
<tr>
<td>Foundations</td>
<td>0</td>
<td>-2</td>
<td>-7</td>
<td>-15</td>
<td>-25</td>
</tr>
<tr>
<td>Slopes</td>
<td>0</td>
<td>-2</td>
<td>-7</td>
<td>-15</td>
<td>-25</td>
</tr>
</tbody>
</table>

Applying the Ic parameter to the BRMR the RMR index is calculated. The quality of the rock mass is gotten through the following table.

In the 1989 CSIR classification the parameters A1, A2 and A3 are given through the following graphics:

A1:

A2:
A3:

The A4 parameter is estimated through the sum of five partial indexes:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range of Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density length (m)</td>
<td>&lt; 1, 1.5, 3, 10, 20, &gt; 20</td>
</tr>
<tr>
<td>Aperture (mm)</td>
<td>None, &lt; 0.1, 0.1 - 1.0, 1.5, &gt; 5</td>
</tr>
<tr>
<td>Roughness</td>
<td>Very rough, Rough, Slightly rough, Smooth, Stickiness</td>
</tr>
<tr>
<td>Raking (gouge mm)</td>
<td>None, &lt; 5 hard filling, &gt; 5 hard filling, &lt; 5 soft filling, &gt; 5 soft filling</td>
</tr>
<tr>
<td>Weathing</td>
<td>Unweathered, Slightly weathered, Moderately weathered, Highly weathered, Decomposed</td>
</tr>
</tbody>
</table>

Finally the A5 parameter is calculated as in the 1979 classification. The sum of the five partial ratings gives the Basic RMR (BRMR). The BRMR in the condition of dry discontinuities corresponds to the parameter GSI (Geological Strength Index):

\[ \text{GSI} = \text{BRMR}_{29} - 5 \quad \text{(when BRMR}>23) \]

The corrective index Ic is given by the following table:

<table>
<thead>
<tr>
<th>Corrective index Ic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>0</td>
</tr>
<tr>
<td>&lt; 5 hard filling</td>
<td>1</td>
</tr>
<tr>
<td>&gt; 5 hard filling</td>
<td>2</td>
</tr>
<tr>
<td>&lt; 5 soft filling</td>
<td>3</td>
</tr>
<tr>
<td>&gt; 5 soft filling</td>
<td>4</td>
</tr>
<tr>
<td>Unweathered</td>
<td>5</td>
</tr>
<tr>
<td>Slightly weathered</td>
<td>6</td>
</tr>
<tr>
<td>Moderately weathered</td>
<td>7</td>
</tr>
<tr>
<td>Highly weathered</td>
<td>8</td>
</tr>
<tr>
<td>Decomposed</td>
<td>9</td>
</tr>
</tbody>
</table>
Applying the Ic parameter to the BRMR the RMR index is calculated. The quality of the rock mass is gotten through the following table.

<table>
<thead>
<tr>
<th>RMR</th>
<th>0-20</th>
<th>21-40</th>
<th>41-60</th>
<th>61-80</th>
<th>81-100</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLASS</td>
<td>V</td>
<td>IV</td>
<td>III</td>
<td>II</td>
<td>I</td>
</tr>
<tr>
<td>QUALITY</td>
<td>Very poor</td>
<td>Poor</td>
<td>Fair</td>
<td>Good</td>
<td>Very good</td>
</tr>
</tbody>
</table>

The geomechanical parameters of the rock mass are directly correlatable to the BRMR index through the following expressions:

\[
\varphi(°) = 5 + \frac{\text{BRMR}}{2}
\]
\[
c(\text{MPa}) = 0.005 \times \text{BRMR}^{\frac{\text{BRMR}}{10}}
\]
\[
E(\text{GPa}) = 10^{\frac{40}{\text{BRMR}}}
\]

dove:
- \(\varphi(°)\) = Angle of shear strength of the rock mass;
- \(c(\text{MPa})\) = Cohesion of the rock mass;
- \(E(\text{GPa})\) = Young modulus of the rock mass.

The RMR index can be correlated to the Q index (N.G.I. Q-System) and to RSR (Rock Structure Rating) through the following correlations:

\[
\text{RMR} = 9\ln Q + 44
\]
\[
\text{RMR} = \frac{\text{RSR} - 12.4}{0.77}
\]
C.S.I.R. Rock Mass Rating by the Sen's formula

Sen et al. (2003) suggested a numerical correlation between RMR and some parameters of the rock mass:

\[ RMR = 0.2RQD + 15\log(sp) + 0.075\sigma_c - 2.9\log(qw) + 34 + A_4 - A_6 \]

where
- \( RQD \) = Rock Quality Designation;
- \( sp(m) \) = mean discontinuity spacing;
- \( \sigma_c \) (MPa) = uniaxial compressive strength of the intact rock;
- \( qw (l/s) \) = hydraulic discharge along a 10 m length of the front;
- \( A_4 \) and \( A_6 \) = parameters \( A_4 \) and \( A_6 \) by 1989 C.S.I.R.

Barton (N.G.I. Q-System).

The Q index is calculated by the following formula:

\[ Q = \frac{RQD J_n}{J_n SRF} \]

The parameters at the second member have the following meaning.

- RQD % (Rock Quality Designation).
- \( J_n \) (Joint Set Number).

It depends on the number of the joint sets identifiable in the rock mass.
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<table>
<thead>
<tr>
<th>2. JOINT SET NUMBER</th>
<th>$J_p$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Massive, no or few joints</td>
<td>0.5 - 1.0</td>
</tr>
<tr>
<td>B. One joint set</td>
<td>2</td>
</tr>
<tr>
<td>C. One joint set plus random</td>
<td>3</td>
</tr>
<tr>
<td>D. Two joint sets</td>
<td>4</td>
</tr>
<tr>
<td>E. Two joint sets plus random</td>
<td>6</td>
</tr>
<tr>
<td>F. Three joint sets</td>
<td>9</td>
</tr>
<tr>
<td>G. Three joint sets plus random</td>
<td>12</td>
</tr>
<tr>
<td>H. Four or more joint sets, random,</td>
<td></td>
</tr>
<tr>
<td>heavily jointed, 'sugar cube', etc.</td>
<td>15</td>
</tr>
<tr>
<td>J. Crushed rock, earthlike</td>
<td>20</td>
</tr>
</tbody>
</table>

- Jr (Joint Roughness Number).

It depends on the discontinuity roughness.

<table>
<thead>
<tr>
<th>3. JOINT ROUGHNESS NUMBER</th>
<th>$J_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>a. Rock wall contact</td>
<td></td>
</tr>
<tr>
<td>b. Rock wall contact before 10 cm shear</td>
<td></td>
</tr>
<tr>
<td>A. Discontinuous joints</td>
<td>4</td>
</tr>
<tr>
<td>B. Rough and irregular, undulating</td>
<td>3</td>
</tr>
<tr>
<td>C. Smooth undulating</td>
<td>2</td>
</tr>
<tr>
<td>D. Slickensided undulating</td>
<td>1.5</td>
</tr>
<tr>
<td>E. Rough or irregular, planar</td>
<td>1.5</td>
</tr>
<tr>
<td>F. Smooth, planar</td>
<td>1.0</td>
</tr>
<tr>
<td>G. Slickensided, planar</td>
<td>0.5</td>
</tr>
<tr>
<td>c. No rock wall contact when sheared</td>
<td></td>
</tr>
<tr>
<td>H. Zones containing clay minerals thick</td>
<td>1.0</td>
</tr>
<tr>
<td>enough to prevent rock wall contact</td>
<td>(nominal)</td>
</tr>
<tr>
<td>J. Sandy, gravelly or crushed zone thick</td>
<td>1.0</td>
</tr>
<tr>
<td>enough to prevent rock wall contact</td>
<td>(nominal)</td>
</tr>
</tbody>
</table>
- Ja (Joint Alteration Number).

It depends on the alteration degree of the rock discontinuities and the characteristics of the filling.

<table>
<thead>
<tr>
<th>4. JOINT ALTERATION NUMBER</th>
<th>( J_a )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a. ) Rock wall contact</td>
<td></td>
</tr>
<tr>
<td>A. Slightly healed, hard, non-softening, impermeable filling</td>
<td>0.75</td>
</tr>
<tr>
<td>B. Unaltered joint walls, surface staining only</td>
<td>1.0</td>
</tr>
<tr>
<td>C. Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.</td>
<td>2.0</td>
</tr>
<tr>
<td>D. Silty- or sandy-clay coatings, small clay- traction (non-softening)</td>
<td>3.0</td>
</tr>
<tr>
<td>E. Softening or low-friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum, and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1 - 2 mm or less)</td>
<td>4.0</td>
</tr>
</tbody>
</table>

| \( b. \) Rock wall contact before 10 cm shear |         |
| F. Sandy particles, clay-free, disintegrating rock etc. | 4.0     |
| G. Strongly over-consolidated, non-softening clay mineral fillings (continuous < 5 mm thick) | 6.0     |
| H. Medium or low over-consolidation, softening clay mineral fillings (continuous < 5 mm thick) | 8.0     |
| J. Swelling clay fillings, i.e. montmorillonite, (continuous < 5 mm thick). Values of \( J_a \) depend on percent of swelling clay-size particles, and access to water. | 8.0 - 12.0 |

| \( c. \) No rock wall contact when sheared |         |
| K. Zones or bands of disintegrated or crushed | 6.0     |
| L. rock and clay (see G, H and J for clay | 8.0     |
| M. conditions) | 8.0 - 12.0 |
| N. Zones or bands of silty- or sandy-clay, small clay fraction, non-softening | 8.0     |
| O. Thick continuous zones or bands of clay | 10.0 - 13.0 |
| P. & R. (see G, H and J for clay conditions) | 6.0 - 24.0 |
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- Jw (Joint Water Number).

It depends on the ground water conditions.

- S.R.F (Stress Reduction Factor).

It depends on stress condition of the rock mass.

5. JOINT WATER REDUCTION

<table>
<thead>
<tr>
<th>Description</th>
<th>Jw</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Dry excavation or minor inflow i.e. &lt; 5 l/m locally</td>
<td>1.0</td>
</tr>
<tr>
<td>B. Medium inflow or pressure, occasional outwash of joint fillings</td>
<td>0.66</td>
</tr>
<tr>
<td>C. Large inflow or high pressure in competent rock with unfilled joints</td>
<td>0.5</td>
</tr>
<tr>
<td>D. Large inflow or high pressure</td>
<td>0.33</td>
</tr>
<tr>
<td>E. Exceptionally high inflow or pressure at blasting, decaying with time</td>
<td>0.2 - 0.1</td>
</tr>
<tr>
<td>F. Exceptionally high inflow or pressure</td>
<td>0.1 - 0.05</td>
</tr>
</tbody>
</table>

6. STRESS REDUCTION FACTOR

<table>
<thead>
<tr>
<th>Description</th>
<th>SRF</th>
</tr>
</thead>
<tbody>
<tr>
<td>a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated</td>
<td></td>
</tr>
<tr>
<td>A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)</td>
<td>10.0</td>
</tr>
<tr>
<td>B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth &lt; 50 m)</td>
<td>5.0</td>
</tr>
<tr>
<td>C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth &gt; 50 m)</td>
<td>2.5</td>
</tr>
<tr>
<td>D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)</td>
<td>7.5</td>
</tr>
<tr>
<td>E. Single shear zone in competent rock (clay free), (depth of excavation &lt; 50 m)</td>
<td>6.0</td>
</tr>
<tr>
<td>F. Single shear zone in competent rock (clay free), (depth of excavation &gt; 50 m)</td>
<td>2.5</td>
</tr>
<tr>
<td>G. Loose open joints, heavily jointed or 'sugar cube', (any depth)</td>
<td>5.0</td>
</tr>
</tbody>
</table>
The three ratios, in the formula to determine Q, have a specific physical meaning:

- **RQD/Jn**: It defines the rock mass structure and gives an approximate estimate of the block size.
- **Jr/Ja**: It takes into account the mechanical behavior of the rock mass.
- **Jw/SRF**: It expresses the effective stress condition acting on the rock mass.

The Q-system index (varying from 0.001 to 1000) is composed by 9 intervals, to which correspond as many classes of quality of the rock mass. The intervals are expressed in logarithmic scale.
Wickham (Rock Structure Rating).

It's based on the estimate of the RSR index (Rock Structure Rating), so defined:

$$\text{RSR} = A + B + C.$$ 

Where A, B and C are three partial indexes given through the following schemes.

- **Parameter A**: General characteristics of the rock.

<table>
<thead>
<tr>
<th>Basic Rock Type</th>
<th>Geological Structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hard</td>
<td>Medium</td>
</tr>
<tr>
<td>Igneous</td>
<td>1</td>
</tr>
<tr>
<td>Metamorphic</td>
<td>1</td>
</tr>
<tr>
<td>Sedimentary</td>
<td>2</td>
</tr>
<tr>
<td>Type 1</td>
<td>30</td>
</tr>
<tr>
<td>Type 2</td>
<td>27</td>
</tr>
<tr>
<td>Type 3</td>
<td>24</td>
</tr>
<tr>
<td>Type 4</td>
<td>19</td>
</tr>
</tbody>
</table>
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- Parameter B: Block size and direction of drive.

| Average joint spacing                      | Strike ⊥ to Axis | Strike || to Axis |
|--------------------------------------------|------------------|--------------|
|                                            | Both             |              |
|                                            | With Dip | Against Dip | Dipping | Vertical | Dipping | Vertical |
| 1. Very closely jointed, < 2 in            | 9        | 11          | 13       | 10       | 12       | 9        | 11       |
| 2. Closely jointed, 2-6 in                 | 13       | 16          | 19       | 15       | 17       | 14       | 14       |
| 3. Moderately jointed, 6-12 in             | 23       | 24          | 28       | 19       | 22       | 23       | 19       |
| 4. Moderately to blocky, 1-2 ft            | 30       | 32          | 36       | 25       | 28       | 30       | 28       |
| 5. Blocky to massive, 2-4 ft               | 36       | 38          | 40       | 33       | 35       | 36       | 24       |
| 6. Massive, > 4 ft                         | 40       | 43          | 45       | 37       | 40       | 40       | 38       |

To identify a rock mass class, it's necessary to correlate RSR index to RMR or Q:

\[ RSR = 0.77 \text{ RMR} + 12.4 \]

\[ RSR = 13.3 \log Q + 46 \]

- Parameter C: Physical characteristics of the discontinuities and hydraulic condition.

<table>
<thead>
<tr>
<th>Anticipated water inflow gpm/1000 ft of tunnel</th>
<th>Sum of Parameters A + B</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>13 - 44</td>
</tr>
<tr>
<td></td>
<td>Good</td>
</tr>
<tr>
<td>None</td>
<td>22</td>
</tr>
<tr>
<td>Slight, &lt; 200 gpm</td>
<td>19</td>
</tr>
<tr>
<td>Moderate, 200-1000 gpm</td>
<td>15</td>
</tr>
<tr>
<td>Heavy, &gt; 1000 gpm</td>
<td>10</td>
</tr>
</tbody>
</table>

Superscript a: Dip; flat: 0°-20°, dipping: 20-50°, and vertical: 50-90°

Superscript b: Joint condition: good = tight or cemented; fair = slightly weathered or altered; poor = severely weathered, altered or open
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Romana (S.M.R. - Slope Mass Rating).

The classification represents the application of the 1979 Bieniawski's classification to the case of rock slope stability. The SMR index (Slope Mass Rating) is given by the following relation:

\[ SMR = A_1 + A_2 + A_3 + A_4 + A_5 + (F_1 \times F_2 \times F_3) + F_4 \]

The A1-A5 indexes are relative to the Bieniawski's classification.

<table>
<thead>
<tr>
<th>PARAMETERS</th>
<th>RANGE OF VALUES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength of intact rock material</td>
<td>&lt;10</td>
</tr>
<tr>
<td>Joint spacing (m)</td>
<td>&gt;2.0</td>
</tr>
<tr>
<td>Ground water</td>
<td>Completely dry</td>
</tr>
<tr>
<td>Rating</td>
<td>15</td>
</tr>
</tbody>
</table>

The sum of the five partial indexes gives the Basic RMR (BRMR). The SMR index has to be calculated by the following formula:

\[ SMR = BRMR + (F_1 \times F_2 \times F_3) + F_4 \]

The variables F1, F2 and F3 depend on the orientation of the most unfavorable joint in the rock mass as a function of the slope orientation. F1 is given by the following expression:

\[ F_1 = [1 - \sin(|\alpha_j - \alpha_f|)]^2 \]

where \( \alpha_j \) and \( \alpha_f \) are, respectively, the dip direction of the most unfavorable joint and the dip direction of the slope.

F2 is obtained by the formula:
$F_2 = tg^2 \beta_j$

where $\beta_j$ if the angle of dip of the most unfavorable joint. When $F_2 > 1$, is must impose $F_2 = 1$.

$F_3$ is a correction to apply to the BRMR value as a function of the difference between the dip angle of the most unfavorable joint and the dip angle of the slope ($\beta_j - \beta_f$). It practically corresponds to the Bieniawski's correction:

<table>
<thead>
<tr>
<th>Joint dip direction - Slope dip direction</th>
<th>Very favourable ($&lt;10^\circ$)</th>
<th>Favourable ($10^\circ$-$20^\circ$)</th>
<th>Fair ($&gt;20^\circ$)</th>
<th>Unfavourable ($&gt;20^\circ$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rating</td>
<td>0</td>
<td>-5</td>
<td>-25</td>
<td>-50</td>
</tr>
</tbody>
</table>

$F_4$ is a correction to apply as a function of the excavation method:

<table>
<thead>
<tr>
<th>Excavation method</th>
<th>Natural slope</th>
<th>Presplitting</th>
<th>Smooth blasting</th>
<th>Blast or mechanical</th>
<th>Deficient blasting</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rating</td>
<td>-15</td>
<td>10</td>
<td>8</td>
<td>0</td>
<td>-8</td>
</tr>
</tbody>
</table>

The SMR index is correlated to the quality of the rock mass and to the stability condition of the rock slope:

<table>
<thead>
<tr>
<th>SMR</th>
<th>0-20</th>
<th>21-40</th>
<th>41-80</th>
<th>81-100</th>
</tr>
</thead>
<tbody>
<tr>
<td>CLASS</td>
<td>V</td>
<td>IV</td>
<td>III</td>
<td>I</td>
</tr>
</tbody>
</table>

Recommendation about the slope support is also given:
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Geological Strength Index (G.S.I.).

It's a fundamental parameter to envisage the mechanical behavior of the rock mass (see paragraph 1.7). It can be gotten by a correlation to the RMR index or, as an alternative, can be evaluated through the procedure suggested by Sonmez & Ulusay (1999).

GSI is estimated as a function of two variables, SR (Structure Rating) and SCR (Surface Condition Rating). SR is given by the following scheme:

\[
SR = \text{volumetric joint count} \quad J_v
\]

where SR depends on the volumetric joint count \((J_v)\) of the rock mass.
SCR can be obtained by the sum of three partial factors, as a function of the roughness and weathering degree of the joints and of the thickness and characteristics of the filling.

\[
SCR = R_r + R_w + R_f
\]

1.9 Tunnels.

Rock-support analysis.

The rock-support analysis is here presented using the Hoek and Brown simplified method. This procedure is based on some basic prerequisites.

- **Tunnel geometry**: the tunnel has a circular-section of initial radius \( r_i \) and a length such that the problem could be considered two-dimensional.
- **In situ stress field**: in situ horizontal and vertical stresses have the same magnitude \( p_0 \).
- **Support pressure**: the installed supports exert a uniform radial pressure \( p_i \) on the tunnel walls.
- **Characteristics of the undisturbed rock mass**: the rock mass has, in undisturbed condition, a linear-elastic behavior, characterized by a Young modulus \( E \) and a Poisson's ratio \( \nu \); the failure criterion is given by:

\[
\sigma_1 = \sigma_3 + \left( m\sigma_3^2 + s\sigma_3^2 \right)^{0.5}
\]

- **Characteristics of the disturbed rock mass**: the rock mass has, in disturbed condition, a perfectly plastic behavior and satisfies the following failure criterion:
Volumetric strains: the elastic-behavior regions are governed by the variables $E$ and $\nu$; at failure, the rock mass is exposed to a volume increase and the strains are calculated using the theory of plasticity.

Time-dependent behavior: both the undisturbed and disturbed rock masses don't exhibit a time-dependent behavior.

Extent of the plastic zone: the plastic-behavior zone reaches a radius $r$, which depends on the in situ pressure, the support pressure and the rock mass characteristics.

The scheme of calculation is listed below (Hoek e Brown, 1982).

**Input data:**

- $\sigma_c$: Uniaxial compressive strength of the intact rock;
- $m$, $s$: Constants of the undisturbed rock mass;
- $E$: Young modulus of the undisturbed rock mass;
- $\nu$: Poisson's ratio of the rock;
- $m_r$, $s_r$: Constants of the disturbed rock mass;
- $\gamma_r$: Unit weight of the rock;
- $p_0$: In situ pressure;
- $r_i$: Tunnel radius.

**Calculation sequence.**

The calculation sequence has to be repeated, using a $p_i$ (support pressure) value varying from 0 to $p_0$.

- $M = \frac{1}{2} \left[ \left( \frac{m}{4} \right)^2 + m \frac{p_0}{\sigma_c} + s \right]^{0.5} - \frac{m}{8}$
\[
D = \frac{-m}{m + 4\left[\frac{m}{\sigma_c}(p_0 - M\sigma_c) + s\right]^{0.5}}
\]

- \[
N = 2\left[\frac{p_0 - M\sigma_c}{m\sigma_c} + \frac{s}{m^2}\right]^{0.5}
\]
- \(p_i > p_0 - M\sigma_c\) strain around the tunnel is elastic.
- \(\frac{u_i}{r_i} = \frac{1 + \nu}{E}(p_0 - p_i)\)
- \(p_i \leq p_0 - M\sigma_c\) plastic failure around the tunnel.
- \(\frac{u_e}{r_e} = \frac{(1 + \nu)}{E}M\sigma_c\)
- \(\frac{r_e}{r_i} = e^{N^{-2}\left[\frac{p_i}{m\sigma_c} - \frac{s}{m^2}\right]^{0.5}}\)

Per \(\frac{r_e}{r_i} < \sqrt{3}\) : \(R = 2D\ln\frac{r_e}{r_i}\)
Per \(\frac{r_e}{r_i} > \sqrt{3}\) : \(R = 1.1D\)

\[
e_{av} = 2\frac{\frac{r_e}{r_i}}{\frac{r_e}{r_i}}\left(\frac{r_e}{r_i}\right)^2
\]

\[
A = 2\frac{\frac{u_e}{r_i} - e_{av}}{\left(\frac{r_e}{r_i}\right)^2}
\]
\[
\frac{u_i}{r_i} = 1 - \left( \frac{1 - \varepsilon_{av}}{1 + A} \right)^{0.5}
\]

- For the roof of the tunnel, plot \( \frac{u_i}{r_i} \) as a function of \( \frac{p_i + \gamma_c (r_e - r_i)}{p_0} \)

- For the sidewalls of the tunnel, plot \( \frac{u_i}{r_i} \) as a function of \( \frac{p_i}{p_0} \)

- For the floor of the tunnel, plot \( \frac{u_i}{r_i} \) as a function of \( \frac{p_i - \gamma_c (r_e - r_i)}{p_0} \)

The variable \( p_0 - M\sigma_c \) represents the critical pressure, that's the pressure the tunnel support has to contrast to prevent the making of a plastic zone around the tunnel.

**Rock-support design.**

The design of the tunnel support (concrete or shotcrete lining, steel set and rock bolt) has to be executed, proceeding by trial, first calculating the support stiffness and the maximum support pressure and then plotting the support curve on the graphic pressure vs strain. The method makes possible to combine two supports and to process a compound support curve. The support will be considered well designed, when the support curve will intersect, in the chart, the pressure-vs-strain curves relative to the roof, the sidewalls and the floor of the tunnel.
Concrete or shotcrete lining: calculation of the support stiffness and the maximum support pressure.

Input data:
- $E_c$(MPa) = Modulus of elasticity of concrete or shotcrete;
- $v_c$ = Poisson's ratio of concrete or shotcrete;
- $t_c$(m) = Thickness of concrete or shotcrete;
- $r_i$(m) = Tunnel radius;
- $\sigma_{cc}$(MPa) = Uniaxial compressive strength of concrete or shotcrete.

Stiffness: $k_i = \frac{E_c v_i^2 - (r_i - t_i)^2}{(1 + v_c)(1 - 2v_c)r_i^2 + (r_i - t_i)^2}$

Maximum pressure: $P_{sc, \text{max}} = \frac{1}{2} \sigma_{cc} \left[ 1 - \frac{(r_i - t_i)^2}{r_i^2} \right]$

Steel set: calculation of the support stiffness and the maximum support pressure.

Input data:
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W(m)= Flange width of the steel set;
X(m)= Depth of the section of the steel set;
A_s(m^2)= Cross-sectional area of the steel set;
I_s(m)= Moment of inerzcia of the steel section;
E_s(Mpa)= Modulus of elasticity of the steel section;
σ_y(MPa)= Yield strength of steel;
r_i(m)= Tunnel radius;
S(m)= Steel set spacing along the tunnel;
θ (rad)= Angle between the blocking points;
t_b(m)= Thickness of the blocks;
E_b(MPa)= Modulus of elasticity of the blocks.

Stiffness:
\[
\frac{1}{k_s} = \frac{S_r}{E_s A_s} + \frac{S_r^3}{2E_s I_s} \left[ \frac{\theta \left( \theta + \sin \theta \cos \theta \right)}{2\sin^2 \theta} - 1 \right] + \frac{2S\theta r_b}{E_b W^2}
\]

Maximum pressure:
\[
p_{ax,\text{max}} = \frac{3A_s I_s \sigma_{xy}}{2S\theta \left( 3I_s + XA_s \left[ r_i - \left( t_b + \frac{1}{2}X \right) \right] \left( 1 - \cos \theta \right) \right)}
\]

- Bolt: calculation of the support stiffness and the maximum support pressure.

Input data:
l(m)= Free bolt length;
d_b(m)= Bolt diameter;
E_b(MPa)= Modulus of elasticity of the bolts;
Q(MPa)= Load-deformation constant for anchor and head;
T_{ax}(MN)= Ultimate failure load from pull-out test;
r_i(m)= Tunnel radius;
s_c(m)= Circumferential bolt spacing;
s_l(m)= Longitudinal bolt spacing.

Stiffness:
\[
\frac{1}{k_b} = \frac{S_r}{r_i} \left[ \frac{4I_s}{\pi d_b E_b} + Q \right]
\]
Maximum pressure:  \( p_{sb \text{ max}} = \frac{T_{bf}}{s_c s_i} \)

- Calculation of the support curve in a single support system.

Input data:
\( k = \) Stiffness support;  
\( p_{smax} = \) Maximum support pressure;  
\( u_{i0} = \) Initial tunnel deformation before support installation.

The support curve is obtained, making the pressures \( (p_i) \) varying from 0 to \( p_{smax} \) in the following relation:

\[
u_i = \left( \frac{u_{i0}}{r_i} + \frac{p_i}{k} \right) r_i
\]

- Calculation of the support curve in a compound support system (the supports are installed at the same time).

Input data:
\( k_1 = \) Stiffness of the support 1;  
\( p_{smax1} = \) Maximum pressure of support 1;  
\( k_2 = \) Stiffness of the support 2;  
\( p_{smax2} = \) Maximum pressure of support 2;  
\( u_{i0} = \) Initial tunnel deformation before support installation.

The support curve is obtained, making the pressures \( (p_i) \) varying from 0 to \( p_{smax} \) in the following relations:

\[
\begin{align*}
  u_{max 1} & = \frac{r_i p_{s max 1}}{k_1} \\
  u_{max 2} & = \frac{r_i p_{s max 2}}{k_2} \\
  u_{i2} & = \frac{r_i p_i}{(k_1 + k_2)}
\end{align*}
\]
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For $u_{12} < u_{\text{max}1} < u_{\text{max}2}$: $u_i = \left( \frac{u_0}{r_i} + \frac{p_i}{(k_1 + k_2)} \right) r_i$

For $u_{12} > u_{\text{max}1} < u_{\text{max}2}$: $p_{\text{max}12} = \frac{u_{\text{max}1} (k_1 + k_2)}{r_i}$

For $u_{12} < u_{\text{max}2} < u_{\text{max}1}$: $p_{\text{max}12} = \frac{u_{\text{max}2} (k_1 + k_2)}{r_i}$
1.10 Rock slope stability.

Markland test
This test allows to get an indication of the stability of the wedges inside of the rock mass as a function of their spatial orientation and of the average shear strength along their discontinuities. The discontinuity shear strength is quantified through an average angle of shear strength. The considers five possible conditions.
1. Potentially unstable wedge.

This condition happens when the wedge is oriented in the slope direction and the angle of shear strength is lesser than the angle of dip of the sliding line.
2. Stable wedge.

This condition happens when the wedge is oriented in the slope direction and the angle of shear strength is greater than the angle of dip of the sliding line or when the wedge is anti-dip slope oriented.
3. Uncertain instability.

This condition happens when the wedge is oriented in the slope direction and the angle of shear strength is approximately equal to the angle of dip of the sliding line (±2°).
4. Block toppling.

This condition happens when the slope and one of the discontinuity are approximately vertical and have a similar dip direction.
5. Roof stability.

The presence of unstable blocks in the roof of a tunnel is put in evidence by the intersection of three or more discontinuities which form a closed shape.

Once identified the potential slope instabilities, the Markland test makes possible to quantify the correction to apply in the Bieniawski’s rock mass classification, on base of the following scheme:
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<table>
<thead>
<tr>
<th>Very favourable</th>
<th>= no unstable wedges</th>
</tr>
</thead>
<tbody>
<tr>
<td>Favourable</td>
<td>= uncertain instabilities</td>
</tr>
<tr>
<td>Fair</td>
<td>= one unstable wedge</td>
</tr>
<tr>
<td>Unfavourable</td>
<td>= two unstable wedges</td>
</tr>
<tr>
<td>Very unfavourable</td>
<td>= three or more unstable wedges</td>
</tr>
</tbody>
</table>

Planar stability.

In case of potential sliding along a single discontinuity, the slope stability can be performed through a two-dimensional scheme.

Two-dimensional scheme (after Giani, 1988)

The safety factor can be described by the following expression:

\[
SF = \frac{[W(1 \pm k_c)\cos \beta - U - V \sin \beta + R \sin(\beta + \alpha)] \tan \phi + cA}{W \sin \beta - R \cos(\beta + \alpha) + V \cos \beta + k_h W \cos \beta}
\]

where:

- W = weight of the sliding wedge;
- V = volume of the sliding wedge;
- A = area of the sliding plane;
- \( \alpha \) = angle of dip of the discontinuity;
- \( \phi \) = angle of shear strength along the discontinuity;
- c = cohesion along the discontinuity;
- U = groundwater pressure along the discontinuity;
- H = height of the slope;
- z = depth of the tension crack;
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\( V \) = groundwater pressure along the tension crack;
\( \gamma_w \) = unit weight of water;
\( R \) = magnitude of the external force, if present;
\( \beta \) = angle from the horizontal plane of \( R \);
\( k_v \) = vertical seismic coefficient;
\( k_h \) = horizontal seismic coefficient.

The variables \( c \) and \( \varphi \) are calculated, using the procedure seen in the paragraph 1.7.

The groundwater flow inside the slope can be taken in account, in the slope stability calculation, by supposing one of the four following scenarios:

1. The rock mass is heavily fractured and a real piezometric line is formed: \( U = \gamma_w V \cos \alpha \).
2. The rock mass is heavily fractured and there is a saturated tension crack: \( U = \gamma_w V \cos \alpha, \ V = 0.5 \gamma_w z^2 \);
3. The rock mass is usually drained and only a temporary flow along the discontinuity and the tension crack occurs during rainy events: \( U = 0.5 \gamma_w z (H-z) \cosec \alpha, \ V = 0.5 \gamma_w z^2 \);
4. The rock mass is usually drained and only a temporary flow along the discontinuity occurs during rainy events, but occasionally the
flow can be stuck at the slope toe: $U = 0.5\gamma_w(H-z)^2\csc\alpha + \gamma_w z H \csc\alpha$.

**Three-dimensional stability.**

The simplest scheme of three-dimensional instability is referred to the case of a tetrahedral wedge having one or two free surfaces. For the calculation of the safety factor, the weight of the wedge, having two free surfaces, can be decomposed in two components:

- $T_{12}$ acting along the intersection line of the two joints;
- $N_{12}$ normal to this line.

The last one must be balanced by a tangential reaction $T_N$ and by a normal reaction $N$, acting on both the faces.

The reaction $N$ determines the maximum mobilised resistance to the sliding and the safety factor can be defined as follows:
\[ F_S = \frac{AT_r \frac{N}{A}}{\sqrt{\left(\frac{T_{i2}}{2}\right)^2 + T_N^2}} \]

where:
- \( A \) = area of each joint;
- \( T_r \) = selected shear strength criterion.

To reach equilibrium along the normal direction to the intersection line must be:

\[ 2N \sin \frac{i}{2} + 2T_N \cos \frac{i}{2} = N_{i2} = W \cos b_{i2} \]

where:
- \( i \) = angle between the joints A and B;
- \( b_{i2} \) = angle of dip, in respect to the horizontal plane, of the intersection line.

From a static point of view, the problem is undetermined, because different combinations of \( N \) and \( T_N \) can give different safety factors. By assuming \( T_N = 0 \), the maximum safety factor, among the possible ones, is calculable (method of the rigid wedge).

To calculate the safety factor, it's necessary, first of all, that the orientation of the joints, in respect to the slope, be able to allow the sliding. Moreover it needs that the wedge be in contact to the underlying rock mass, that is the normals to the wedge faces have to be directed downward.

The safety factor can be calculated as follows:

\[ 1) \quad F_S = \frac{A_1T_{r1} \frac{N_1}{A_1} + A_2T_{r2} \frac{N_2}{A_2}}{T_{i2}} \]

where:
- \( A_1 \) = area of the joint 1
A2 = area of the joint 2
TR1 (N1/A1) = available shear strength along the joint 1 as a function of the normal stress N1/A1;
TR2 (N2/A2) = available shear strength along the joint 2 as a function of the normal stress
T12 = component of the wedge weight acting along the intersection of the planes 1 and 2.

The expression 1) is valid only in case both N1 and N2 are greater than zero and the wedge moves itself along the intersection between the planes 1 and 2. In case of N1>0 and N2<0 or N1<0 and N2>0 the sliding happens along the dip angle respectively of the plane 1 and of the plane 2 and not along the intersection line. Thus the safety factor has to be expressed in the following ways:

\[ FS = \frac{A_1 TR_1 N_1}{A_1 T_1} \] (N1>0 and N2<0)

where

\[ T_1 = \text{component of the wedge weight acting along the dip angle of the plane 1} \]

\[ FS = \frac{A_2 TR_2 N_2}{A_2 T_2} \] (N2>0 and N1<0)

where

\[ T_2 = \text{component of the wedge weight acting along the dip angle of the plane 2} \]

Finally, in case of N1<0 and N2<0 (wedge which is uplifted in respect to the underlying rock mass, due to a toppling or to a very high water pressure) it's impossible to define a safety factor and thus it assumes a instability without numerically quantify it.